JTG

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

JTG 3362—2018（EN）

Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts

公路钢筋混凝土及预应力混凝土桥涵设计规范

（英文版）

Issue date：July 18，2018 Effective date：November 1，2018

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

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

Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts

公路钢筋混凝土及预应力混凝土桥涵设计规范

(英文版)

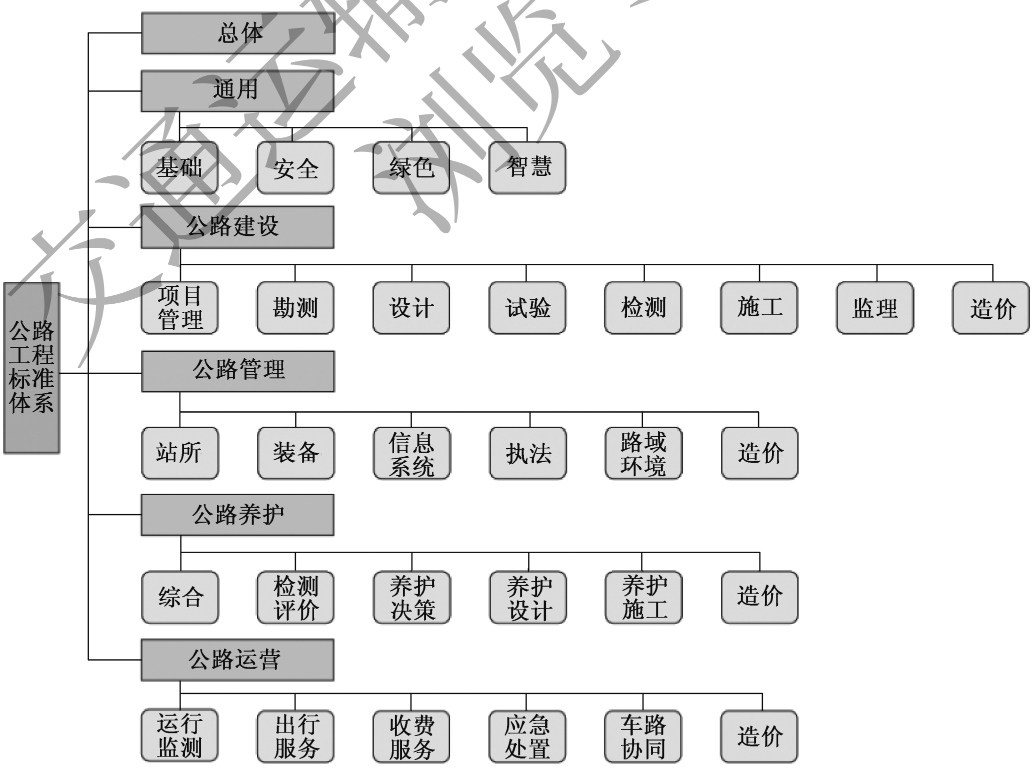
JTG 3362—2018 (EN)

Chief editing organization for Chinese version: CCCC Highway Consultants Co., Ltd.
Chief editing organization for English version: Fuzhou University
Issuing authority: Ministry of Transport of the People's Republic of China
Effective date: November 1, 2018

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。



在公路标准体系当中，包含了多项桥梁相关的设计、施工、养护标准，有效支撑了中国公路桥梁的快速发展。其中桥梁设计规范目前有：《公路桥涵设计通用规范》《公路钢筋混凝土及预应力混凝土桥涵设计规范》《公路圬工桥涵设计规范》《公路钢结构桥梁设计规范》《公路钢混组合桥梁设计与施工规范》《公路斜拉桥设计规范》《公路悬索桥设计规范》《公路钢管混凝土拱桥设计规范》《公路装配式混凝土桥梁设计规范》《公路桥梁抗风设计规范》《公路桥梁抗撞设计规范》《公路桥梁景观设计规范》等。截至 2022 年底，中国已建成公路桥梁 103.3 万座、8576.5 万延米，其中，混凝土结构桥梁占比 90% 以上。《公路钢筋混凝土及预应力混凝土桥涵设计规范》作为公路桥涵结构设计极其重要的规范，在中国交通建设行业得到了非常广泛的应用。

第一部《公路钢筋混凝土及预应力混凝土桥涵设计规范》(JTJ 023—85)于 1985 年颁布实施。尔后，经历了 2004 年的第一次修订(JTG D62—2004)和 2018 年的第二次修订(JTG 3362—2018)。经过近四十年的技术发展，建立了内容较为完整的公路混凝土桥涵结构设计技术体系。本次编译的《公路钢筋混凝土及预应力混凝土桥涵设计规范》(JTG 3362—2018)中文版于 2018 年 7 月修订发布，并于 2018 年 11 月 1 日实施。

《公路钢筋混凝土及预应力混凝土桥涵设计规范》(JTG 3362—2018)采用以概率理论为基础、按分项系数表达的极限状态设计方法；充实了箱梁抗倾覆设计方法、空间效应分析模型、应力扰动区设计方法、体外预应力设计方法等技术要求，对推动桥梁建设技术进步、提升公路混凝土桥梁品质发挥了非常重要的作用。

本规范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，为其他国家类似建设条件的公路桥涵建设提供参考借鉴。

本规范英文版的编译工作由中华人民共和国交通运输部委托福州大学主持完成，并由中华人民共和国交通运输部公路局组织审定。

本规范英文的内容与现行中文版一致，如出现异议时，以中文版为准。

感谢中文版主编袁洪先生在本规范英文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:福建省福州市福州大学城乌龙江北大道2号,邮政编码:350108,电子邮箱:baochunchen@ fzu. edu. cn),以便修订时研用。

英文版主编单位:福州大学

英文版主编:陈宝春

英文版参编人员:陈昭晖,刘君平,苏家战

英文版主审:Khaled Sennah

英文版参与审查人员:张慧彧,夏勇,姚海冬,董延峰

The People's Republic of China

Ministry of Transport

Public Notice

No.59

Public Notice on Issuing the
Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts

The *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362—2018) is hereby issued as one of the industry standards for highway engineering to become effective on November 1, 2018. The former edition of the specifications JTG D62—2004 and its English version JTG D62—2004 (EN) named *Code for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* shall be superseded from the same date.

The general administration and final interpretation of *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362—2018) belong to the Ministry of Transport; while particular interpretation for application and routine administration of the *Specifications* shall be provided by CCCC Highway Consultants Co., Ltd.

Comments, suggestions and inquiries are welcome and should be addressed to CCCC Highway Consultants Co., Ltd. (No. 83, Deshengmenwai Street, Xicheng District, Beijing 100088, China). The feedback will be considered in future revisions.

It is hereby announced.

Ministry of Transport of the People's Republic of China

July 16, 2018

General Office of Ministry of Transport   Printed on July 16, 2018

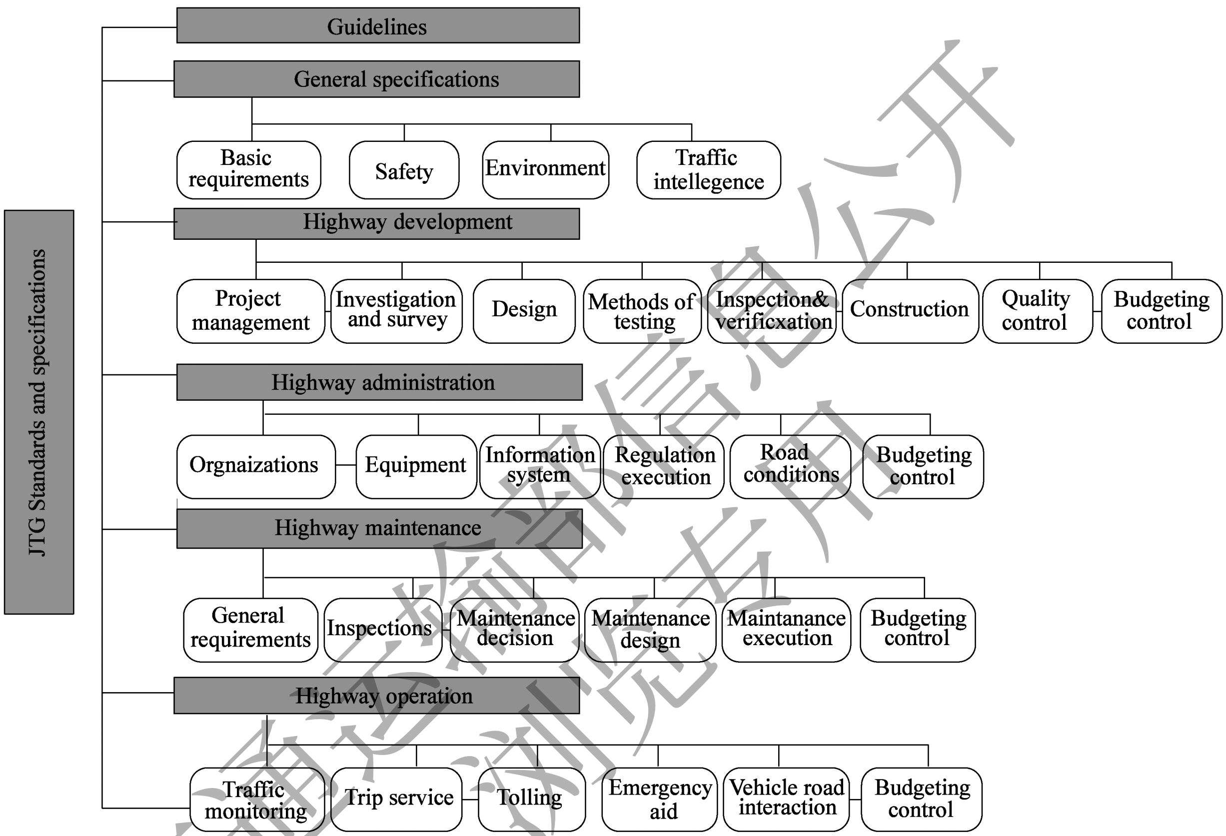
Introduction to English Version

Standards reflect the achievement of civilization, provide common languages for technical communications and improve global connectivity. In recent years, the Chinese government has been proactively implementing standardization to stimulate innovation, coordination, greenness and opening up for shared development in China and worldwide. To align with the Belt One Road Initiative for mutual development, the Ministry of Transport of the People's Republic of China organized the compilation and publication of international version of Chinese transportation industry standards and specifications to meet the increasing demands for international cooperation in transportation, enhance global connectivity, promote knowledge dissemination and sharing of experience.

JTG is the designation referring to the standards and specifications of highway transportation industry, issued by the Ministry of Transport of the People's Republic of China. This system emcompasses the entire lifecycle of highway engineering projects, from planning and construction to maintenance and operation management. It includes standards for the facilities, technologies, management, and services required throughout these processes, as well as standards related to safety, environmental protection, and economic evaluation.

In the highway standard system, it includes a number of standards for design, construction and maintenance of bridges, which have effectively supported the rapid development of highway bridges in China. The current bridge design specifications include: *General Specifications for Design of Highway Bridges and Culverts*, *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts*, *Code for Design of Highway Masonry Bridges and Culverts*, *Specifications for Design of Highway Cable-stayed Bridge*, *Specifications for Design of Highway Suspension Bridge*, *Specifications for Design of Highway Concrete-filled Steel Tubular Arch Bridges*, *Specifications for Design of Highway Precast Concrete Bridges*, *Wind-resistant Design Specification for Highway Bridges*, *Specifications for Collision Design of Highway Bridges*, *Specifications for Landscape Design of Highway*

Bridges, etc. As of the end of 2022, 1.033 million highway bridges with the total length of 85.765 million meters have been built in China, among which concrete bridges accounts for more than 90%. *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* is one of the fundamental standards for structural design of highway bridges and culverts, and has been widely applied to the transportation industry of China.



The first edition of *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTJ 023-85) was issued in 1985 which underwent the first revision in 2004 (JTG D62—2004) and the second revision in 2018 (JTG 3362—2018). After nearly 40 years of technical development, a complete technical system for the design of highway concrete bridges and culverts has been established. The Chinese version of *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362—2018) was revised and issued in July 2018 and put into effect on November 1, 2018.

Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts (JTG 3362—2018) has adopted the limit state design methodology, which is based on the probability theory and expressed with partial factors. Technical requirements for overturning resistance design of box girder bridges, analysis models for spatial effects, design methods for disturbed regions, design methods for bridges with external tendons and others were supplemented. These developments have played an important role in promoting technical progress in bridge construction and improving quality of highway concrete bridges.

The release of the English version of the *Specifications* aims to share the engineering experience and technical achievements from China and provide references for other countries to build highway bridges and culverts with similar construction conditions.

The editing of the English version was conducted by Fuzhou University under the authorization of the Ministry of Transport of the People's Republic of China and approved by the Highway Department, the Ministry of Transport of the People's Republic of China.

The contents and numbering of the chapters, sections, clauses and sub-clauses in the English version are consistent with those in the Chinese version. In the event of any ambiguity or discrepancy between the English version and the Chinese version of the *Specifications*, the Chinese version shall prevail.

Gratitude is given here to Mr. Yuan Hong, the chief editor of the Chinese version, for the valuable guidance and comments during the review of the English version.

Feedbacks are welcome and will be taken into account in future editions. Please address them to the chief editing organization for English version in writing (Address: No. 2, Wulongjiang North Avenue, Fuzhou University Town, Fuzhou, Fujian, China, Postal Code: 350108, E-mail: baochunchen@fzu.edu.cn).

Chief editing organization for English version: Fuzhou University

Chief editor for English version: Chen Baochun

Associate editors for English version:
Chen Zhaohui, Liu Junping, Su Jiazhan

Review editor for English version: Khaled Sennah

Associate review editors for English version:
Zhang Huiyu, Xia Yong, Yao Haidong, Dong Yanfeng

Foreword to Chinese Version

The revision of *Code for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG D62—2004), pursuant to the "Notice of Planning for Compilation and Amendment Program of Highway Engineering Standards in 2011" (No. 115) issued by the Ministry of Transport of the People's Republic of China in 2006, was carried out by the chief editing organization for Chinese version, CCCC Highway Consultants Co., Ltd.

The revision has been approved and issued as *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (*JTG* 3362—2018) for implementation.

During the revision, the editorial team conducted extensive special studies and research works, reviewed the updated technical development and design experience in China, and referred to relevant domestic and international standards. Upon the completion of the first draft of the *Specifications*, the drafted specification was circulated for comments from relevant experts and organizations involved in design, construction, maintenance, and administration, based on which several rounds of discussion, consultation, and updating were conducted before being finalized for approval.

The main revisions made in this version include the following contents: strength grades of reinforcement used in concrete bridges and culverts were updated; basic requirements for the design of bridge structures were added; design requirements for durability of concrete bridges and culverts were emphasized; requirements for checking of overturning resistance of concrete box girder bridges, practical refined analysis methods for complex bridges, design methods for bridges with external tendons, and design methods for disturbed regions of concrete bridges were supplemented; calculation methods for resistance on cross-sections of circular compression members were updated; calculation formulas for determining effective length factors of compression members with different boundary conditions were added; calculation methods for determining crack width in reinforced concrete structures or Type B prestressed concrete structures were adjusted; and requirements for detailing design were supplemented.

Feedbacks are welcome and will be taken into account in future editions. Please address them to the chief editing organization for Chinese version (Mr. Li Huichi, Address: CCCC Highway Consultants Co., Ltd., No. 85, Deshengmenwai Street, Xicheng District, Beijing 100088, China; Fax No.: 010-82017041; Email: sssohpdi@163.com).

Chief editing organization for Chinese version:

CCCC Highway Consultants Co., Ltd.

Associate editing organizations for Chinese version:

Tongji University
Southeast University
Research Institute of Highway, Ministry of Transport
Dalian University of Technology
CCCC Highway Bridges National Engineering Research Center Co., Ltd.

Chief editor for Chinese version: Yuan Hong

Associate editors for Chinese version:

Zhao Junli	Xu Guoping	Xu Dong	Liu Zhao
Lv Jianming	Gong Jinxin	Feng Min	Li Huichi
Li Wenjie			

Chief review editor for Chinese version: Shen Yonglin

Other review editors for Chinese version:

Yu Peiyu	Yang Yaoquan	Feng Pengcheng	Xi Guangheng
Zhang Shaoqing	Li Huaifeng	Ma Jianzhong	Liu Junqi
Liu Dongxu	Dai Benliang	Xu Hongguang	Han Dazhang
Li Zheng	Shi Fanghua	Zhong Mingquan	Tian Bo
Liang Linong	Bao Qiwei	Qin Dahang	Xu Yue

Contents

1 General Provisions

1.0.1 The *Specifications* is developed for regulating the design of reinforced and prestressed concrete highway bridges and culverts to ensure the quality of the projects.

1.0.2 The *Specifications* is applicable to the design of reinforced and prestressed concrete bridge and culvert structures to be built on classified highways, and is not applicable to the design of bridge and culvert structures made of special concrete.

1.0.3 The limit state design methodology, which is based on the probability theory and expressed with partial factors, is adopted in the *Specifications* for design purposes.

1.0.4 In addition to the *Specifications*, the design of reinforced and prestressed concrete bridge and culvert structures shall also comply with the provisions in current relevant national and industry standards.

2 Terms and Symbols

2.1 Terms

2.1.1 Reinforcing steel

Non-prestressed reinforcement in concrete structural members.

2.1.2 Prestressing steel

Materials used for applying prestressing forces in concrete structural members, including steel wires, steel strands, and steel bars.

2.1.3 Reinforced concrete structure

A concrete structure that contains load-carrying reinforcing steel.

2.1.4 Prestressed concrete structure

A concrete structure that contains prestressing steel that is used to develop prestressing forces by tensioning or other methods.

2.1.5 Limit state

A state beyond which a whole structure or a part of the structure no longer satisfies a certain functional requirement for which it was designed.

2.1.6 Design situation

A set of design conditions that represent the corresponding conditions occurring during a certain time period in the entire process from construction to operation of the structure, for which the design will demonstrate that the relevant limit states are not exceeded.

2.1.7 Characteristic material strength

A basic representative value of material strength adopted in the design of structural members, which is determined using the fractile value with 95 percent level of confidence through statistical analysis of the material strength data obtained by the specified standard test methods for standard specimens.

2.1.8 Partial factor

The factors adopted in the design formulas to ensure the designed structure meet the specified reliability when the probabilistic limit state design method is used. The factors are classified into two types: the partial factor for action and the partial factor for resistance (materials).

2.1.9 Design material strength

The value obtained from the characteristic material strength divided by a partial factor.

2.1.10 Safety level

The level designated to a bridge or culvert in the design according to the severity of consequences of its failure.

2.1.11 Importance factor of structure

A factor which relates to the safety level and is used for adjusting the effects of action combinations in order to achieve a specified reliability of the structure.

2.1.12 Nominal geometric parameter

The basic representative value of a geometrical parameter adopted for the design of a structure or a structural member, which may be taken from the value specified in the design documents.

2.1.13 Design ultimate load-carrying capacity/Design resistance

The ultimate load-carrying capacity of a structure or member calculated using the design value of material strength for its ultimate limit state design.

2.1.14 Cracking moment

The theoretical bending moment when the initial crack appears on a member.

2.1.15 Construction load

Temporary loads applied on a structure or a structural member during construction, which are considered in a structural design for transient situations, including self-weight of the structure, loads of formwork attached to the structure or structural member, construction materials, equipment, personnel, etc.

2.1.16 Durability design

The design to achieve durability of structures or members in the design life, including selection of materials, application of construction measures and implementation of additional protection.

2.1.17 Disturbed region

The region in concrete structures where the sectional strain distribution does not conform to the assumption that plane sections remain plane, is also referred to as D region.

2.1.18 Strut-and-tie model

A truss model that reflects the force transfer path and force effects in the disturbed regions of a concrete structure.

2.1.19 Bursting force

Transverse tensile forces in the post-tensioned anchorage zone, resulting from the spreading of concentrated anchor forces.

2.1.20 Spalling force

Tensile forces in concrete around base plates, resulting from the compressive deformation ahead of anchorage due to concentrated anchor force in the post-tensioned anchorage zone.

2.1.21 Thickness of concrete cover

Minimum distance between the surface of reinforcement and the surface of the concrete of a member.

2.1.22 Development length

The required length for reinforcement (a reinforcing bar or prestressing tendon) to achieve the design bearing stress by the effect of bonding between its surface and concrete or by the confining effect of its bent end.

2.2 Symbols

2.2.1 Symbols of Material Properties

C30—Concrete with the characteristic cube compressive strength of 30 MPa

E_c, G_c—Modulus of elasticity of concrete, shear modulus of concrete, respectively

E_s, E_p—Modulus of elasticity of reinforcing steel and prestressing steel, respectively

$f_{ce,d}$—Design equivalent compressive strength of concrete strut in strut-and-tie model

f_{ck}, f_{cd}—Characteristic and design axial compressive strength of concrete, respectively

f'_{ck}, f'_{tk}— Characteristic axial compressive strength and tensile strength of concrete at the construction stage (the transient situation), respectively

f_{cu}—Compressive strength of 150 mm concrete cube

f'_{cu}—Compressive strength of 150 mm concrete cube at the construction stage

$f_{cu,k}$—Characteristic compressive strength of 150 mm concrete cube

f_{pk}, f_{pd}—Characteristic and design tensile strength of prestressing steel, respectively

f_{sk}, f_{sd}—Characteristic and design tensile strength of reinforcing steel, respectively

f'_{sd}, f'_{pd}—Design compressive strength of reinforcing steel and prestressing steel, respectively

f_{tk}, f_{td}—Characteristic and design axial tensile strength of concrete, respectively

2.2.2 Symbols of Actions and Action Effects

F_{ld}—Design concentrated reaction force or local compressive force

M_{1Gd}, M_{2Gd}—Design bending moment in composite flexural member due to self-weight at the first phase and the second phase (Translator's note: non-composite phase and composite phase), respectively

M_{1Qd}—Design bending moment in composite flexural member due to other additional actions (Translator's note: except the self-weight of the structure) at the first phase

M_{2Qd}—Design bending moment in composite flexural member due to variable actions at the second phase

M_{cr}—Cracking moment on the cross-section of the flexural member

M_d—Design bending moment

M_k—Bending moment calculated under the combination of the characteristic actions

M_s, M_l—Bending moment calculated under the frequent combination and the quasi-permanent combination of actions, respectively

N_d—Design axial force

N_p—Resultant force in prestressing steel and reinforcing steel in post-tensioned members

N_{p0}—Resultant force in prestressing steel and reinforcing steel when normal stress in concrete equals zero

T_d—Design torsional moment

V_d—Design shear force

V_{cs}—Design shear resistance of the inclined section composed of the resistances of concrete and stirrups

V_{sb}—Design shear resistance of bent bars intersecting with the inclined section

V_{pb}—Design shear resistance of curved tendons intersecting with the inclined section

W_{fk}—Maximum crack width calculated in flexural member

σ_{cc}—Normal compressive stress in concrete on the cracked section of a member at the service stage

σ_{con}, σ'_{con}—Maximum jacking stress for prestressing steel in the tension zone and compression zone, respectively (which are stresses ahead of anchorages inside beams for post-tensioned members)

σ_{kc}, σ_{kt}—Normal compressive stress and tensile stress in concrete calculated with the characteristic actions, respectively

σ_l, σ'_l—Loss of prestress of prestressing steel in corresponding phases in the tension zone and compression zone of members, respectively

σ_{p0}, σ'_{p0}—Stress of longitudinal prestressing steel when the normal stress of concrete at the point of the resultant force in longitudinal prestressing steel in compression zone and tension zone equals zero, respectively

σ_{pc}—Normal precompression stress of concrete due to prestressing force

σ_{pe}, σ'_{pe}—Effective prestress of longitudinal prestressing steel in compression zone and tension zones, respectively

σ_{pt}—Normal tensile stressin concrete due to prestress

σ_s, σ_p—Stress or stress increment of longitudinal reinforcing steel and prestressing steel in the analysis of cross-sectional resistance, respectively

σ_{ss}—Stress of longitudinal reinforcing steel in cracked section induced by the frequent combination of actions

σ_{st}, σ_{lt}—Normal tensile stress of concrete at the tension edge of the member section under frequent combinations of actions and quasi-permanent combinations of actions, respectively

σ_{tp}, σ_{cp}—Principal tensile and compressive stress of concrete in a member, respectively

τ—Shear stress of concrete in concrete structure or member

2.2.3 Symbols of Geometrical Parameters

A—Gross area of section of a member

A_0, A_n—Area of transformed section and net section of a member, respectively

A_{cor}—Area of concrete core confined by wire fabric, spiral reinforcement or stirrup

A_{cr}—Area of transformed section for cracked section

A_l, A_{ln}—Local compression area and local compression net area of concrete, respectively

A_p, A'_p— Cross-sectional areas of longitudinal prestressing steel in the tension zone and compression zone of a member, respectively

A_s—Cross-sectional area of longitudinal reinforcing steel in the tension zone of a member, or cross-sectional area of all longitudinal reinforcing steel of a member with circular section

A'_s— Cross-sectional area of longitudinal reinforcing steel in the compression zone of a member

A_{sb}, A_{pb}— Cross-sectional areas of bent bars and curved tendon in the same bent-up plane, respectively

A_{sv}—Total cross-sectional area of each leg of the stirrup in one section

B—Flexural stiffness of an equivalent section of a cracked member

B_0—Flexural stiffness of transformed section for the whole section

B_{cr}—Flexural stiffness of transformed section for the cracked section

I—Moment of inertia of the gross section

I_0, I_n—Moment of inertia of transformed section and net section, respectively

I_{cr}—Moment of inertia of transformed section for cracked section

S_0, S_n—Statical moment of area above (or below) calculated fiber of transformed section and net section with respect to gravity axis, respectively

W—Elastic resisting moment at the tensile edge of the gross section

W_0, W_n—Elastic resisting moment at tensile edges of transformed section and net section, respectively

a, a'—Distance from the resultant force in reinforcing steel or prestressing steel in tension and compression zones to the proximal edge of the section, respectively

a_s, a_p—Distance from the resultant force in reinforcing steel or prestressing steel in the tension zone to the edge of the tension zone, respectively

a'_s, a'_p—Distance from the resultant force in reinforcing steel or prestressing steel in the compression zone to the edge of the compression zone, respectively

b—Width of rectangular section, web width of T-section or I-section

b_f, b'_f—(Effective) flange width in the tension zone and compression zone of T-section or I-section, respectively

h_f, h'_f—Flange depth in tension zone and compression zone of T-section or I-section, respectively

c—Thickness of the concrete cover

d—Nominal diameter of reinforcement

e, e'—Distance from the axial force to the resultant force in longitudinal reinforcement in tension zone and compression zone, respectively

e_0—Eccentricity of axial force with respect to the gravity axis of the section

e_s, e_p—Distance from the axial force to the resultant force in longitudinal reinforcing steel and prestressing steel in the tension zone, respectively

e'_s, e'_p—Distance from the axial force to the resultant force in longitudinal reinforcing steel and prestressing steel in the compression zone, respectively

e_{p0}, e_{pn}—Eccentricity of resultant force in prestressing steel and reinforcing steel with respect to the gravity axis of the transformed section and net section, respectively

l—Effective span of the flexural member or length between nodes of the compression member

l_0—Effective length of the compression member

l_n—Clear span of the flexural member

r—Radius of circular section

s_v, s_p—Spacing of stirrups and vertical prestressing steels, respectively

y_0, y_n—Distance from the center of gravity ofthe transformed section and that of net section to the location of the calculated fiber, respectively

y_p, y'_p—Distance from the resultant force in prestressing steel in tension zone and compression zone of a member to the gravity axis of the transformed section, respectively

y_{pn}, y'_{pn}— Distance from the resultant force in prestressing steel in the tension zone and compression zone of a member to the gravity axis of the net section, respectively

y_s, y'_s—Distances from the centroid of reinforcing steel in the tension zone and compression zone of a member to the gravity axis of the transformed section, respectively

y_{sn}, y'_{sn}—Distances from the centroid of reinforcing steel in the tension zone and compression zone of a member to the gravity axis of the net section, respectively

x—Depth of compression zone

z— Internallever arm, i.e., distance from the resultant force in longitudinal tension reinforcement to that of concrete in the compression zone

2.2.4 Symbols of Calculation Coefficients and Others

k_{qf}—Factor of safety against overturning in the transverse direction

α_{ES}, α_{EP}—Ratio of modulus of elasticity of reinforcing steel and prestressing steel to the modulus of elasticity of concrete, respectively

β_{cor}—Increase coefficient of local compression capacity when confinement reinforcement is arranged

β_a—Reduction coefficient for effective wall thickness of box section in calculation of its torsional resistance

β_t—Reduction coefficient of torsional resistance of concrete in shear-torsion member

γ—Influence coefficient of plasticity of concrete in the tension zone

γ_0—Importance factor of structure for bridge and culvert

η— Amplification factor for eccentricity of axial force in eccentrically loaded compression member

η_θ—Factor for long-term deflection

ρ—Reinforcement ratio for longitudinal tension reinforcement or longitudinal reinforcement ratio

ρ_{sv}—Reinforcement ratio for stirrup

ρ_{te}—Effective reinforcement ratio for longitudinal tension reinforcement

φ—Stability factor of axially loaded compression member

3 Materials

3.1 Concrete

3.1.1 The concrete strength class shall be determined based on the characteristic compressive strength of 150 mm cubes.

3.1.2 The concrete strength class of load-carrying members in highway bridges and culverts shall conform to the following requirements:

1 The concrete strength class shall not be lower than C25 for reinforced concrete members, and shall not be lower than C30 when reinforcing steel with characteristic strength of 400 MPa or above is used.

2 The concrete strength class shall not be lower than C40 for prestressed concrete members.

3.1.3 The characteristic axial compressive strength f_{ck} and the characteristic axial tensile strength f_{tk} of concrete shall comply with the provisions in Table 3.1.3.

Table 3.1.3 Characteristic strength of concrete

Strengthclass	C25	C30	C35	C40	C45	C50	C55	C60	C65	C70	C75	C80
f_{ck} (MPa)	16.7	20.1	23.4	26.8	29.6	32.4	35.5	38.5	41.5	44.5	47.4	50.2
f_{tk} (MPa)	1.78	2.01	2.20	2.51	2.51	2.65	2.74	2.85	2.93	3.00	3.05	3.10

3.1.4 The design axial compressive strength f_{cd} and the desgin axial tensile strength f_{td} of concrete shall comply with the provisions in Table 3.1.4.

Table 3.1.4　Designstrength of concrete

Strengthclass	C25	C30	C35	C40	C45	C50	C55	C60	C65	C70	C75	C80
f_{cd} (MPa)	11.5	13.8	16.1	18.4	20.5	22.4	24.4	26.5	28.5	30.5	32.4	34.6
f_{td} (MPa)	1.23	1.39	1.52	1.65	1.74	1.83	1.89	1.96	2.02	2.07	2.10	2.14

3.1.5　The modulus of elasticity of concrete under compression or tension, E_c, should be taken from Table 3.1.5. It may also be determined from reliable test data.

Table 3.1.5　Modulus of elasticity of concrete

Strengthclass	C25	C30	C35	C40	C45	C50	C55	C60	C65	C70	C75	C80
E_c ($\times 10^4$ MPa)	2.80	3.00	3.15	3.25	3.35	3.45	3.55	3.60	3.65	3.70	3.75	3.80

Notes: For pumped concrete with air-entraining agent and high sand rate, the modulus of elasticity, E_c, for C50-C80 in this table should be multiplied by a reduction factor of 0.95 if test data are not available.

3.1.6　The shear modulus of concrete, G_c, may be taken as 0.4 times E_c in Table 3.1.5, and the Poisson's ratio of concrete, υ_c, may be taken as 0.2.

3.2　Reinforcement

3.2.1　The reinforcement for concrete structures of a highway bridge or culvert shall conform to the following requirements:

1　In reinforced concrete and prestressed concrete members, types of HPB300, HRB400, HRB500, HRBF400 or RRB400 should be selected for reinforcing steel. In prestressed concrete members, deformed bars among those mentioned before shall be selected for stirrups. Cold-rolled deformed bars may be used for wire fabrics required by detailing.

2　In prestressed concrete members, steel strands and steel wires shall be adopted for prestressing tendons; prestressing threaded bars may be adopted for medium- and small-sized members, or used as vertical and transverse prestressing tendons.

3.2.2　The characteristic tensile strengths of reinforcing steel f_{sk} and prestressing steel f_{pk} shall comply with the provisions in Table 3.2.2-1 and Table 3.2.2-2, respectively.

Table 3.2.2-1　Characteristic tensile strength of reinforcing steel

Type of reinforcing steel	Symbol	Nominal diameter d (mm)	f_{sk} (MPa)
HPB300	ϕ	6 ~ 22	300

continued

Type of reinforcing steel	Symbol	Nominal diameter d (mm)	f_{sk} (MPa)
HRB400 HRBF400 RRB400	ϕ ϕ^F ϕ^R	6 ~ 50	400
HRB500	ϕ	6 ~ 50	500

Table 3.2.2-2 Characteristic tensile strength of prestressing steel

Type of prestressing steel		Symbol	Nominal diameter$d$ (mm)	f_{pk} (MPa)
Steel strand	1 × 7	ϕ^S	9.5, 12.7, 15.2, 17.8	1720, 1860, 1960
			21.6	1860
Stress-relieved wire	Plain wire Deformed wire	ϕ^P ϕ^H	5	1570, 1770, 1860
			7	1570
			9	1470, 1570
Prestressing threaded bar		ϕ^T	18, 25, 32, 40, 50	785, 930, 1080

Notes: The usage of steel strands with a characteristic tensile strength of 1960 MPa as prestressing steel shall be based on reliable engineering experience or fully verified by test results.

3.2.3 Design tensile strength f_{sd} and design compressive strength f'_{sd} of reinforcing steel shall be taken from Table 3.2.3-1. Design tensile strength f_{pd} and design compressive strength f'_{pd} of prestressing steel shall be taken from Table 3.2.3-2.

Table 3.2.3-1 Design tensile strength and design compressive strength of reinforcing steels

Type of reinforcing steel	f_{sd} (MPa)	f'_{sd} (MPa)
HPB300	250	250
HRB400, HRBF400, RRB400	330	330
HRB500	415	400

Notes: 1. For a reinforcing steel with a tensile strength higher than 330 MPa, its design tensile strength shall be taken as 330 MPa when the reinforcing steel is used in a reinforced concrete member under an axial tensile force or a tensile force with small eccentricity. It shall also be taken as 330 MPa in calculating the shear resistance of the inclined section, torsional resistance and punching shear resistance for transverse reinforcing steel, such as stirrups or confinement reinforcement perpendicular to the longitudinal primary reinforcement.

2. When different types of reinforcing steel are arranged in a member, the design strength of each steel shall be used.

Table 3.2.3-2 Design tensile strength and design compressive strength of prestressing steel

Type of prestressing steel	f_{pk} (MPa)	f_{pd} (MPa)	f'_{pd} (MPa)
Steel strand 1 × 7	1720	1170	390
	1860	1260	
	1960	1330	

continued

Type of prestressing steel	f_{pk} (MPa)	f_{pd} (MPa)	f'_{pd} (MPa)
Stress-relieved wire	1470	1000	410
	1570	1070	
	1770	1200	
	1860	1260	
Prestressing threaded bar	785	650	400
	930	770	
	1080	900	

3.2.4 The modulus of elasticity E_s of reinforcing steel and the modulus of elasticity E_p of prestressing steel should be taken from Table 3.2.4. E_s and E_p may be determined by test data if they are available and reliable.

Table 3.2.4 Modulus of elasticity of reinforcement

Type of reinforcing steel	Modulus of elasticity, E_s ($\times 10^5$ MPa)	Type of prestressing steel	Modulus of elasticity, E_p ($\times 10^5$ MPa)
HPB300	2.10	Steel strand	1.95
HRB400, HRB500 HRBF400, RRB400	2.00	Stress-relieved wire	2.05
		Prestressing threaded bar	2.00

4 Basis of Structural Design

4.1 General

4.1.1 Reinforced and prestressed concrete highway bridges and culverts shall be designed for the following two limit states:

1 Ultimate limit state:

A limit state that a structure and/or its members reach the ultimate load-carrying capacity or the deformation/displacement unfit to keep on loading.

2 Serviceability limit state:

A limit state corresponding to the normal use of a structure and/or its members.

4.1.2 The design for concrete bridges and culverts shall include the following contents:

1 Conceptual design.

2 Detailing design of the structures and members.

3 Analysis of actions and action effects.

4 Limit state checks for structures and members.

5 Special design of structures and members to satisfy specific requirements.

4.1.3 Standardized spans should be adopted for bridges with spans no longer than 50 m.

4.1.4 The span of reinforced concrete beam bridges should comply with the following requirements:

1 The span of prefabricated reinforced concrete slab bridges should not exceed 10 m.

2 The span of cast-in-place reinforced concrete slab bridges should not exceed 10 m for simply-supported slabs, and not exceed 16 m for continuous slabs.

3 The span of prefabricated reinforced concrete T-beam bridges should not exceed 16 m.

4 The span of cast-in-place reinforced concrete box girder bridges should not exceed 20 m for simply-supported structure, and not exceed 25 m for continuous structure.

4.1.5 The span of prestressed concrete beam bridges should comply with the following requirements:

1 The span of prefabricated prestressed concrete voided slab bridges should not exceed 20 m.

2 The span of cast-in-place prestressed concrete slab bridges should not exceed 20 m for simply-supported slabs, and not exceed 25 m for continuous slabs.

3 The span of prefabricated prestressed concrete T-beam bridges should not exceed 50 m.

4 The span of prefabricated prestressed concrete composite box girder bridges should not exceed 40 m.

4.1.6 For bridges with a span larger than 100 m, the concrete girders should be designed as fully prestressed concrete members.

4.1.7 The effect of action on structures shall be analyzed using the theory of elasticity, and should satisfy the following requirements:

1 Internal forces of structural members upon the completion of the bridge shall be accumulated over the designer's assumed construction sequence.

2 Stresses of structural members upon the completion of the bridge shall be determined by accumulating the calculated stresses at each construction stage, using the corresponding net section or transformed section.

3 In computing the effects of vehicular loads, the effects of their transverse positioning shall be taken into account. The effects may be analyzed by a refined finite element model or determined by reliable engineering judgment.

4 Complex concrete bridge structures, such as curved bridges, wide bridges, skewed bridges, or bridges with varying widths or with a fork, may be analyzed using solid finite element models or using practical refined analysis models described in Appendix A.

4.1.8 In a permanent situation, the structural system of a beam bridge shall not be changed, and the following provisions shall also be complied with in its design:

1 Under the fundamental combination of actions, the bearings for compression-only should always remain in compression.

2 When the combination of actions with characteristic values is used (in accordance with Clause 7.1.1), the action effect of a simply-supported or continuous beam with monolithic sections shall comply with the provision in Eq. (4.1.8):

$$\frac{\sum S_{\mathrm{bk,i}}}{\sum S_{\mathrm{sk,i}}} \geqslant k_{\mathrm{qf}} \tag{4.1.8}$$

where:

k_{qf}—factor of safety against overturning in the transverse direction of the bridge, which is taken as $k_{\mathrm{qf}} = 2.5$;

$\sum S_{\mathrm{bk,i}}$—design action effect that contributes to the stability of the superstructure;

$\sum S_{\mathrm{sk,i}}$—design action effect to destabilize the superstructure.

4.1.9 Disturbed regions (D-regions) in structural members may be analyzed by a strut-and-tie model described in Appendix B, a solid finite element model, or simplified formulas based on the specific situation.

4.1.10 Requirements for inspection, monitoring, maintenance or replacement of concrete structures in highway bridges at the service stage should be proposed in the design whenever necessary, and corresponding accesses, spaces or devices should be provided in the design.

4.2 Analysis of Slabs

4.2.1 Slabs supported on four sides with a ratio of long side to short side larger than or equal to 2 may be calculated as a one-way slab with the span of the short side; otherwise, the slab shall be analyzed as a two-way slab.

4.2.2 For simply-supported slabs, the effective span shall be taken as the horizontal distance between the centers of two supports. For slabs interconnected with ribs of beams, the effective span may be taken as the clear distance between two ribs plus the slab thickness for calculating the bending moment, but it shall not be larger than the distance between the centers of the two ribs. For this situation, the bending moment may be calculated according to the following simplified formulas:

1 Bending moment at supports

$$M = -0.7M_0 \tag{4.2.2-1}$$

2 Bending moment at mid-span

1) When the ratio of slab thickness to rib depth is larger than or equal to 1/4:

$$M = +0.7M_0 \tag{4.2.2-2}$$

2) When the ratio of slab thickness to rib depth is smaller than 1/4:

$$M = +0.5M_0 \tag{4.2.2-3}$$

where:

M_0—bending moment at mid-span of the simply-supported slab with a span length equal to effective span.

For slabs interconnected with ribs of beams, the effective span may be taken as the clear distance between the two ribs, and the shear force is calculated according to the simply-supported slab with a span length equal to the effective span.

4.2.3 When calculating force in an one-way monolithic deck slab, the distribution width of the wheel load in the deck slab should be calculated based on the following provisions:

1 Load distribution width parallel to the span length

$$b = b_1 + 2h \tag{4.2.3-1}$$

2 Load distribution width perpendicular to the span length

1) When a single wheel is located at the mid-span of the slab:

$$a = (a_1 + 2h) + d + \frac{l}{3} \geqslant \frac{2}{3}l \tag{4.2.3-2}$$

2) When multiple wheels are located at the mid-span of the slab, and the load distribution widths for different wheels calculated by Eq. (4.2.3-2) overlap, the following formula

applies:

$$a = (a_1 + 2h) + d + \frac{l}{3} \geqslant \frac{2}{3}l + d \tag{4.2.3-3}$$

3) When wheels are located on the support of the slab:

$$a = (a_1 + 2h) + t \tag{4.2.3-4}$$

4) When wheels are located near the support of the slab with a distance from the support of x:

$$a = (a_1 + 2h) + t + 2x \tag{4.2.3-5}$$

However, it should not be larger than the distribution width when wheels are located at the mid-span of the slab.

5) When the total distribution width calculated according to this provision is larger than the width of the slab, the width of the slab is used as the load distribution width.

6) For precast slabs that are not connected to each other, the distribution width of the wheel in the slab shall not be larger than the width of the precast slab.

In the above formulas:

l—effective span of the slab;

h—thickness of deck overlay;

t—thickness of the slab at mid-span;

d—distance between the centers of two outmost wheels for the situation with multiple wheels;

a_1, b_1—dimensions of the wheel patch contacted with the deck slab perpendicular to and parallel to the span length, respectively.

4.2.4 When the angle between a line normal to the axis of a support and the longitudinal axis of the bridge, i.e. skew angle, is not larger than 15°, the skewed slab of a monolithic skewed slab bridge may be calculated as a right slab. When $l/b \leqslant 1.3$, its effective span may be taken as the perpendicular distance between the two support axes. When $l/b > 1.3$, its effective span may be taken as the skew span length. Herein, l is the skew span, b is the width of the slab normal to the longitudinal axis of the bridge.

Prefabricated slabs in a prefabricated skewed slab bridge with shear keys may be calculated as right slabs with a width equal to the perpendicular distance between two slab edges and an effective span equal to the skew span length.

4.2.5 When $l_c \leqslant 2.5$m, the wheel load distribution width in the direction perpendicular to the span of the cantilever slab may be calculated according to the following provisions:

$$a = (a_1 + 2h) + 2l_c \tag{4.2.5}$$

where:

a—wheel load distribution width in the direction perpendicular to the span length of the cantilever slab;

a_1—wheel contact dimension with the slab in the direction perpendicular to the span length of the cantilever slab;

l_c—horizontal distance between the outer edge of the web and the end of the spreading line, which spreads from the outer edge of the wheel through the deck overlay with an angle of 45° (Figure 4.2.5);

h—thickness of deck overlay.

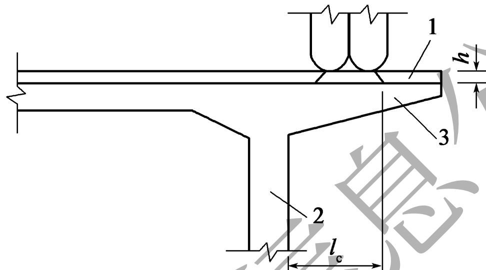


Figure 4.2.5 Distribution of wheel load on cantilever slab

1-Deck overlay; 2-Web; 3-Cantilever slab

4.2.6 For deck slabs with haunches while interconnected with beam ribs, the effective thickness of the slabs may be calculated according to Eq. (4.2.6) for checking cross-sections of the slabs within the haunches or the ribs (Figure 4.2.6):

$$h_c = h'_f + s \cdot \tan\alpha \tag{4.2.6}$$

where:

h_c—effective depth of any investigated section inside the starting point of the haunch to the central line of the rib;

h'_f—thickness of the slab excluding the haunch;

s—horizontal distance from the starting point of the haunch to the investigated section, the section being located between the starting point of the haunch and the axis of the rib;

α—angle between the bottom surface of the haunch and the soffit of the cantilever slab, $\tan\alpha$ is taken as 1/3 when $\tan\alpha > 1/3$.

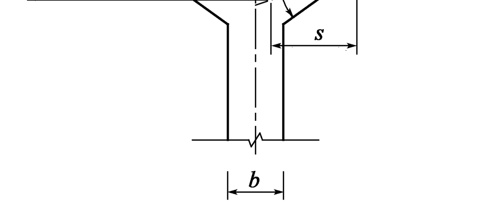


Figure 4.2.6 Effective thickness of slab at haunch

4.3 Analysis of Beams

4.3.1 For the analysis of statically indeterminate structures, the flexural stiffness of members shall be taken in accordance with the following provisions:

Members permitted to crack $0.8E_cI$

Members expected not to crack E_cI

where I is the moment of inertia of the gross section.

4.3.2 In calculating cross-sectional resistances and stresses, effective widths shall be used for compression flanges of T-beams, I-beams and box beams.

4.3.3 Effective width b'_f of the compression flanges of T-beams and I-beams shall be taken in accordance with the following provisions:

1 For interior beams, the minimum of the following three values shall be taken as the effective width:

1) For simply-supported beams, it is taken as 1/3 the effective span. For continuous beams, it is taken as 0.2 times the effective span in the range of positive moment of each interior span, 0.27 times the effective span in the range of positive moment in an end span, and 0.07 times the sum of the effective spans of the two adjacent spans around the support in the range of negative moment at each interior support.

2) Average spacing between two adjacent beams.

3) $(b + 2b_h + 12h'_f)$, where b is the web width, b_h is the haunch width, h'_f is the depth of the overhang flange in the compression zone. When $h_h/b_h < 1/3$, b_h shall be substituted by $3h_h$, where h_h is the depth at the end of the haunch.

2 For exterior beams, it shall be taken as the sum of 1/2 the effective flange width of the adjacent interior beam, 1/2 the web width, and the smaller of 6 times the average thickness of the outer cantilever slab and the physical width of the outer cantilever slab.

4.3.4 The effective width b_{mi} of top and bottom flanges on each side of the web in box beams may be calculated according to the following provisions:

1 For cross-sections at interior portions of a span in a simply-supported beam, each span of a

continuous beam or each interior span of a cantilever beam

$$b_{mi} = \rho_f b_i \quad (4.3.4\text{-}1)$$

$$\rho_f = -6.44\,(b_i/l_i)^4 + 10.10\,(b_i/l_i)^3 - 3.56\,(b_i/l_i)^2 - 1.44(b_i/l_i) + 1.08 \quad (4.3.4\text{-}2)$$

2 For cross-sections at supports of a simply-supported beam, end supports and interior supports of a continuous beam, and cantilever portions of a cantilever beam

$$b_{mi} = \rho_s b_i \quad (4.3.4\text{-}3)$$

$$\rho_s = 21.86\,(b_i/l_i)^4 - 38.01\,(b_i/l_i)^3 + 24.57\,(b_i/l_i)^2 - 7.67(b_i/l_i) + 1.27 \quad (4.3.4\text{-}4)$$

where:

b_{mi}—effective width of the top or bottom flange on each side of the web, $i = 1, 2, 3, \cdots$ (Figure 4.3.4);

b_i—physical width of the top or bottom flange on each side of the web, $i = 1, 2, 3, \cdots$ (Figure 4.3.4);

ρ_f—effective flange width coefficient for cross-sections at interior portions of a span in a simply-supported beam, each span of a continuous beam or each interior span of a cantilever beam;

ρ_s—effective flange width coefficient for cross-sections at supports in a simply-supported beam, end supports and interior supports of a continuous beam, and cantilever portions of a cantilever beam;

l_i—theoretical span length determined according to Table 4.3.4.

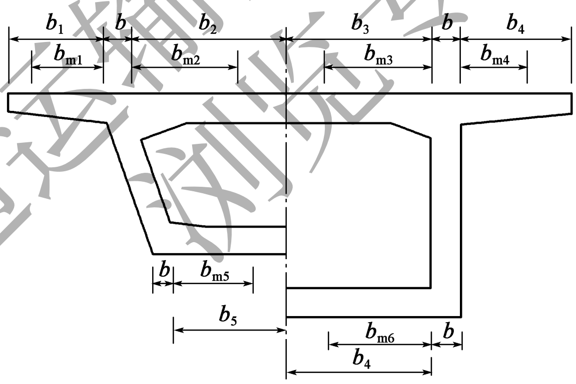
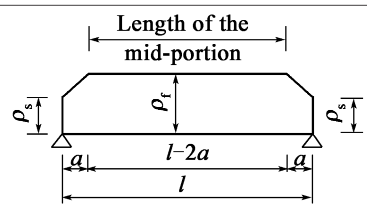


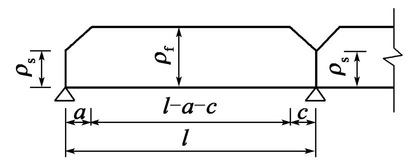
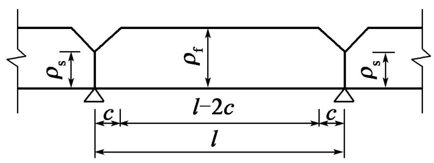
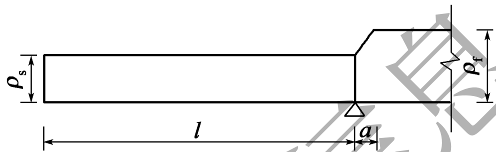
Figure 4.3.4 Effective flange width of box beam

When thebeam depth $h \geqslant b_i/0.3$, the effective flange width shall be equal to the physical flange width.

Table 4.3.4 Applied location ofρ_s, $\rho_f$and theoretical span l_i

Structural system		Theoretical span l_i
Simply-supported beams		$l_i = l$

continued

Structural system			Theoretical span l_i
Continuous beams	End span		For end support or interior portion of a span: $l_i = 0.8l$
	Interior span		Forinterior portion of a span: $l_i = 0.6l$; for interior supports: l_i is equal to 0.2 times the sum of two adjacent spans
Cantilever beams			$l_i = 1.5l$

Notes: 1. The a is equal to the physical flange width b_i corresponding to the effective flange width b_{mi} to be determined, while a shall not be larger than $0.25l$.

2. l is the effective span of the beam.

3. $c = 0.1l$.

4. Within the transition portion of the span with a length of a or c, the effective flange width may be determined as the value between $\rho_s b_i$ and $\rho_f b_i$ by linear interpolation.

4.3.5 In the calculation of the negative moments at interior support of a continuous beam, the reduction effect of the bearing width on the bending moment may be considered. The bending moment after reduction is calculated according to the following formulas (Figure 4.3.5), but it shall not be smaller than 0.9 times the moment before reduction.

$$M_e = M - M' \tag{4.3.5-1}$$

$$M' = \frac{1}{8}qa^2 \tag{4.3.5-2}$$

where:

M_e—negative moment at support after reduction;

M—negative moment at support calculated by theoretical formula or method;

M'—reduced bending moment;

q—load intensity of reaction force R distributed along the gravity axis G-G of the beam at an upward spread angle of 45° from two sides of the support, $q = R/a$;

a—distribution length of reaction force between two intersection points on the gravity axis G-G when spreading upward at an angle of 45° from the two sides of the support (circular support may be converted into square one with the side length equal to 0.8 times the diameter).

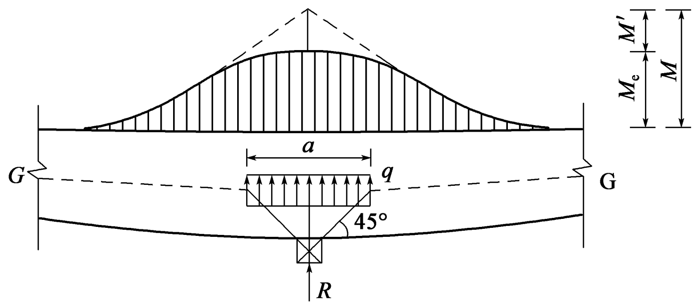


Figure 4.3.5 Calculation diagram of reduction for the bending moment at interior support

4.3.6 For continuous beams with variable depth or with constant depth strengthened by haunches at supports, the variation of the inertia moments shall be considered in calculating the effect of action. When the ratio of inertia moment of the support section to that of the midspan section is equal to or smaller than 2, its influence may not be considered.

4.3.7 In calculating the stress of the section at the interior support, the cross-section of the beam close to the diaphragm may be used if there is a diaphragm at the interior support of the continuous beam.

4.3.8 In calculating the action effect on continuous beams and other statically indeterminate structures, the influence of temperature, concrete shrinkage and creep, and nonuniform settlement of foundation shall be considered according to the specific situation. For prestressed concrete continuous beams and other statically indeterminate structures, secondary effects due to prestressing force shall also be considered.

4.3.9 In calculating the creep of concrete, it may be assumed that creep has a linear relationship with the concrete stress. In the absence of data and suitable calculation methods for the actual local conditions, the creep coefficient of concrete may be calculated in accordance with Appendix C.

The shrinkage strain of concrete may be calculated in accordance with Appendix C.

4.3.10 Stresses on cross-sections of beams induced by positive temperature gradient when exposed to sunlight and negative temperature gradient when cooling may be calculated in accordance with Appendix D. The curve of vertical temperature gradient when exposed to sunlight may be taken from *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015).

4.4 Analysis of Arches

4.4.1 In the design calculation of the arch in an arch bridge, the combination effect of the arch and the spandrel structures may not be considered. If the combination effects are considered, the spandrel structures shall be designed to conform to the conditions presupposed in the calculation. Provisions for arch calculation in this section are all applicable for the mechanical analysis of the arch itself without consideration of the combination effect of the arch and the spandrel structures.

The reduction coefficient for the positive moment in each cross-section of the arch induced by lane load should be taken from Table 4.4.1.

Table 4.4.1 Reduction coefficientfor positive moments

Section	Span length$L$ (m)		
	$L \leqslant 60$	$60 < L < 100$	$L \geqslant 100$
Arch crown, 1/4 span	0.7	Linear interpolation	1.0
Arch springing	0.9	Linear interpolation	1.0
Other section	Linear interpolation		

4.4.2 The arch axis shall be optimally selected during the design of the arch bridge to reduce the eccentricity of axial force under the combination of actions. For a long-span arch bridge, if the deviation of the arch axis from the thrust line of the self-weight in some sections is too large, or the eccentricity of axial force is large under actions of structural gravity as well as the resulted elastic shortening, actions of temperature drop and concrete shrinkage, the arch axis shall be adjusted appropriately. The bending moments induced by deviation of the arch axis from the thrust line of the self-weight shall also be considered.

4.4.3 For a slab arch with box section, the nonuniform distribution of live loads in the transverse direction shall be considered if the spandrel piers are bent columns. For a slab arch with spandrel wall piers, the live loads may be considered to be evenly distributed in the full width of the arch rings when the live loads in the transverse direction of the bridge is not beyond the arch ring.

4.4.4 Live loads on deck arch bridges with braced ribs may be distributed to the arch ribs through the cap beams and spandrel columns.

4.4.5 The cap beams of the spandrel bent piers in the transverse direction of the bridge may be calculated in accordance with the provisions specified in Section 8.4.

4.4.6 During the arch erection and other construction stages for an arch bridge, the sectional strength and stability of the arch at each stage shall be checked.

4.4.7 The arch shall be checked for its in-plane stability according to Clause 5.3.1. Here, the design compression force of the arch, N_d, may be calculated by Eq. (4.4.7):

$$N_d = H_d / \cos\varphi_m \tag{4.4.7}$$

where:

H_d—design compression force of an arch;

φ_m—angle between the horizontal line and the link line between the arch crown and the arch springing.

At the construction stage, the partial factor for the action effect induced by self-weight of the members shall be 1.2; if there are other additional load actions during construction, the partial factor shall be 1.4. For checking the in-plane stability of the arch in the service stage, the partial factor for the action effect may be taken from *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015).

In the in-plane stability analysis, the effective length of the arch may be taken in accordance with the following provisions:

$0.58L_a$ for three-hinged arch;

$0.54L_a$ for two-hinged arch;

$0.36L_a$ for hingeless arch.

L_a is the length of the arch axis.

4.4.8 For an slab arch with a width less than 1/20 of its effective span, its lateral (out-of-plane) stability shall be checked. In analyzing the lateral stability of a braced rib arch, the structure may be regarded as a straight built-up column with a length equal to that of the arch axis, and its effective length and slenderness are determined according to the arch articulation. The averaged axial force of the arch may be calculated by Eq. (4.4.7).

4.4.9 The action effects in the arch springing section induced by wind or centrifugal force may be approximately calculated according to the following assumptions:

1. To calculate the moment M_1 at the beam end, the arch is analogized as a horizontal straight beam fixed at both ends, whose span is equal to the effective span of the arch, and the wind or centrifugal force is applied on the whole beam uniformly.

2. To calculate the moment M_2 at the fixed end, the arch is analogized as a vertical cantilever beam fixed at the bottom, whose span is equal to the effective rise of the arch. The cantilever beam is subjected to the uniform force obtained from the wind load acting on the half span, as well as a concentrated force at its free end, which is obtained from the centrifugal force acting on the half span.

3. The bending moment of the arch, M, is the sum of the projection of the above two moments

on the arch springing section perpendicular to the curve plane:

$$M = M_1 \cos\varphi + M_2 \sin\varphi \quad (4.4.9)$$

where:

φ—angle between the span line and the tangent line of the arch axis at the arch springing.

4.4.10 For a long-span arch bridge, four cross-sections shall be checked, including the cross-section at the arch crown, the 3/8 span, the 1/4 span and the arch springing; for medium and small span arch bridge, the cross-section on the 1/4 and 3/8 span may not be checked; for super-long span arch bridge, besides the four sections described above, additional control sections shall also be selected for checks according to the sectional reinforcement conditions. The resistance of the cross-section shall be checked in accordance with the provisions specified in Section 5.3, and the effective length of the members may be taken in accordance with the provisions in Clause 4.4.7.

4.4.11 A multi-span hingless arch bridge shall be analyzed as a continuous arch structure. When the ratio of the thrust stiffness of the pier to that of the arch is larger than 37, the structure may be analyzed as a single-span arch bridge.

4.4.12 Two-hinged arch may be selected as the support system for a truss arch. The nodes of the truss arch are considered as fixed nodes in analysis. When the structure is analyzed by the simplified method, the nodes may be taken as pinned ones, but the sectional strength of the bottom chords shall have an extra no less than 20%.

The structural self-weight of a truss arch carried by its arch discs may be taken as uniform distribution along the entire span in analysis. But in the construction of the truss arch, if the other members are assembled after the closure of the bottom chords, then all the structural self-weight before the closure shall be carried by the bottom chords. The bridge deck may be considered to carry live load on the bridge together with the top chords.

For the top chords and the web members (vertical and diagonal bracings) connected with the top chords through the nodes, the bending moment induced by the local load on the bridge deck shall be considered.

Transverse distribution of live load shall be considered for truss arches.

Curves close to the thrust line of the self-weight should be taken as the arch axis of the truss arch, such as the catenary with a small arch axis coefficient m or the quadratic parabola.

4.4.13 Movable bearings shall be arranged at both ends of the top chord of the rigid frame arch. The bridge deck may participate in carrying the live load together with the rigid-frame arch discs.

Transverse distribution of live loads shall be considered for rigid-frame arches.

4.4.14 If the ratio of the flexural stiffness of the arch rib section to that of the tie section is smaller than 1/100 in tied arches, the arch rib may be regarded as a flexible arch resisting only the axial compression. If the above ratio is higher than 100, the tie may be regarded as an axial tension member. The joint of the above members may be regarded as hinged.

If the above ratio is between 1/100 and 100, the joint of the tie and the arch rib shall be regarded as fixed, and then, the load-induced bending moment shall be distributed among them according to their flexural rigidities.

4.5 Requirements for Durability Design

4.5.1 The design life of concrete structures and members in highway bridges and culverts shall comply with the provisions in *Technical Standard of Highway Engineering* (JTG B01-2014).

4.5.2 The environmental category for concrete structures and members in highway bridges and culverts shall be determined in accordance with the provisions in Table 4.5.2 based on the type of exposure of their surface to the environment.

Table 4.5.2 Environmental category for concrete structures and members in highway bridges and culverts

Category of environment	Condition
Class Ⅰ-General	Only affected by concrete carbonation
Class Ⅱ- Freeze-thaw	Affected by repeated Freeze-thaw
Class Ⅲ-Coastal or marine chloride	Affected by chlorine in the marine environment
Class Ⅳ-Deicing salt and other chloride	Affected by chlorine salts such as deicing salts
Class Ⅴ-Salt crystallization	Affected by the expansion of sulfate crystals in concrete pores
Class Ⅵ-Chemical corrosive	Corroded by strong acidic or alkaline chemicals
Class Ⅶ-Abrasion	Under the actions of wind, water flow, or water inclusions, such as friction, cutting, and impact.

4.5.3 The concrete strength class for use in each environment category shall not be lower than that specified in Table 4.5.3.

Table 4.5.3 Requirement for lowest concrete strength class

Category of member	Beam, slab, tower, arch, superstructure of culvert		Pier, abutment, substructure of culvert		Pile cap, foundation	
Design life (Year)	100	50, 30	100	50, 30	100	50, 30
Class Ⅰ-General	C35	C30	C30	C25	C25	C25
Class Ⅱ-Freeze-thaw	C40	C35	C35	C30	C30	C25
Class Ⅲ-Coastal or marine chloride	C40	C35	C35	C30	C30	C25
Class Ⅳ-Deicing salt and other chloride	C40	C35	C35	C30	C30	C25
Class Ⅴ-Salt crystallization	C40	C35	C35	C30	C30	C25
Class Ⅵ-Chemical corrosive	C40	C35	C35	C30	C30	C25
Class Ⅶ-Abrasion	C40	C35	C35	C30	C30	C25

4.5.4 The following technical measures for durability shall be taken for concrete structures and members in highway bridges and culverts:

1 The thickness of the concrete cover for reinforcing steel shall satisfy the provisions in Clause 9.1.1.

2 Prestressing system in the prestressed concrete structures shall include multiple protection measures according to the specific situation.

3 For the concrete structure in which permeability is required to be low, the permeability grade of concrete shall comply with the provisions of relevant standards.

4 In the moist environments of cold regions, concrete shall satisfy the requirements for frost resistance. The frost resistance grade of concrete shall comply with the provisions of relevant standards.

5 The structural form and detailing of bridges and culverts shall be conducive to proper drainage and ventilation, while condensation of moisture and accumulation of adverse substances in the structures can be avoided.

5 Design for Ultimate Limit State in Persistent Situations

5.1 General

5.1.1 In the design of highway bridges and culverts under persistent situations, the resistance and stability of the structural members shall be calculated according to the requirement of the ultimate limit state, and the resistance against overturning and sliding of the structure shall also be checked if necessary.

5.1.2 When expressed in the form of internal force, the ultimate limit state of members in bridges and culverts shall be checked by the following equations:

$$\gamma_0 S \leqslant R \tag{5.1.2-1}$$

$$R = R(f_d, \alpha_d) \tag{5.1.2-2}$$

where:

γ_0—importance factor of the structure according to the design safety level of highways bridges and culverts. It is taken to be 1.1, 1.0, and 0.9 for the design safety levels of 1, 2 and 3, respectively. The design safety level of highway bridges and culverts shall meet the requirements of *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015);

S—design value for the effect of the combination of actions, in which the impact force shall be included in the vehicular load. According to the provisions of *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015), the fundamental combination of actions shall be adopted in structural calculations for persistent design situations;

R—design resistance of a member;

$R(\cdot)$—resistance function of a member;

f_d—design material strength;

f_a—design geometric parameter. In the absence of reliable data, the characteristic geometric parameter, a_k, may be used, which is the specified value in design documents.

5.1.3 Flexural resistance of cross-section shall be calculated based on the following basic assumptions:

1 The plane sections remain plane after bending.

2 The tensile strength of concrete is ignored.

3 The stress in longitudinal reinforcement equals to the product of strain and modulus of elasticity, and the value shall meet the following requirements:

$$-f'_{sd} \leqslant \sigma_{si} \leqslant f_{sd} \tag{5.1.3-1}$$

$$-(f'_{pd} - \sigma_{p0i}) \leqslant \sigma_{pi} \leqslant f_{pd} \tag{5.1.3-2}$$

where:

σ_{si}, σ_{pi}—stress in the longitudinal reinforcing steel or prestressing steel of the *i*th layer, which shall be calculated in accordance with the Eqs. (5.1.5-1) and (5.1.5-2), respectively, in which the positive value indicates tensile stress and the negative value indicates compressive stress;

f_{sd}, f'_{sd}—design tensile strength and design compressive strength of longitudinal reinforcing steel, respectively, which shall be taken from Table 3.2.3-1;

f_{pd}, f'_{pd}—design tensile strength and design compressive strength of longitudinal prestressing steel, respectively, which shall be taken from Table 3.2.3-2;

σ_{p0i}—stress in the longitudinal prestressing steel of the *i*th layer when the normal stress in concrete at the centroid of the prestressing steel is zero, which shall be calculated in accordance with Clause 6.1.6 in the *Specifications*.

5.1.4 Calculations of compressive stress for flexural members and eccentrically loaded compression members shall comply with the following provisions:

1 The compressive stress block of concrete in the compression zone at the cross-section shall be simplified as an equivalent rectangular stress block;

2 Ratio of the depth of rectangular stress block to the physical depth of compression zone, β, shall be taken from Table 5.1.4;

3 The compressive strength in the rectangular stress block shall be taken from the design axial

compressive strength of concrete.

Table 5.1.4 Coefficient β

Concrete strength class	C50 and below	C55	C60	C65	C70	C75	C80
β	0.80	0.79	0.78	0.77	0.76	0.75	0.74

5.1.5 The stress of internal longitudinal reinforcement shall be determined according to the following provisions:

For reinforcing steel,

$$\sigma_{si} = \varepsilon_{cu} E_p \left(\frac{\beta h_{0i}}{x} - 1 \right) \tag{5.1.5-1}$$

For prestressing steel,

$$\sigma_{pi} = \varepsilon_{cu} E_p \left(\frac{\beta h_{0i}}{x} - 1 \right) + \sigma_{p0i} \tag{5.1.5-2}$$

where:

x—depth of the rectangular stress block of the compression zone;

h_{0i}—distance from the centroid of longitudinal steel of the ith layer to the edge of the compressive side (for eccentrically loaded compression members, the edge at the side with larger compressive stress is considered);

E_s, E_p—elastic modulus of reinforcing steel and prestressing steel, respectively;

b—ratio of the depth of rectangular stress block to the physical depth of compression zone, which is taken from Table 5.1.4;

ε_{cu}—ultimate compressive strain of concrete under nonuniform compression. When the concrete strength class is C50 or below, $\varepsilon_{cu} = 0.0033$; when the concrete strength class is C80, $\varepsilon_{cu} = 0.003$. If the concrete strength class is between C50 and C80, ε_{cu} can be calculated by linear interpolation.

The stress of internal longitudinal steel calculated by Eq. (5.1.5-1) or Eq. (5.1.5-2) shall comply with the provision of Item 3 in Clause 5.1.3 in the *Specifications*.

5.1.6 In calculating the flexural resistance of the cross-section and inclined section within the anchorage zone at the ends of pretensioned concrete structures, the design tensile strength of prestressing steel in the anchorage zone is taken as zero at the starting point of the anchorage and f_{pd} at the end of the anchorage. The tensile strength between the two points shall be calculated by linear interpolation. The development length of the prestressing steel, l_a, shall be adopted from Table 5.1.6.

Table 5.1.6 Development length l_a (mm) of prestressing steel

Types of prestressing steel	Concrete strength class					
	C40	C45	C50	C55	C60	≥C65
Steel strand 1×7, f_{pd} = 1260 MPa	130 d	125 d	120 d	115 d	110 d	105 d
Deformed wire, f_{pd} = 1200 MPa	95 d	90 d	85 d	83 d	80 d	80 d

Note: 1. For members constructed by the method of abrupt release of the prestressing steel, the starting point of the development length in the calculation is $0.25l_{tr}$ from the member ends, where l_{tr} is the transfer length taken from Table 6.1.8.

2. If the design tensile strength of the prestressing steel, f_{pd}, is different from the value in the table, the development length shall be increased or decreased proportionally to the strength value in the table.

3. The d is the nominal diameter of the prestressing steel.

5.2 Flexural Members

5.2.1 The relative balanced depth of the compression zone of flexural members, ξ_b, shall be taken from Table 5.2.1.

Table 5.2.1 Relative balanced depth of compression zone ξ_b

Types of steel	Concrete strength class			
	C50 or below	C55, C60	C65, C70	C75, C80
HPB300	0.58	0.56	0.54	—
HRB400, HRBF400, RRB400	0.53	0.51	0.49	—
HRB500	0.49	0.47	0.46	—
Steel strand, steel wire	0.40	0.38	0.36	0.35
Prestressing threaded bar	0.40	0.38	0.36	—

Note: 1. For flexural members with different types of reinforcement in the tension zone of the section, the smallest value ξ_b among all ξ_b with respect to all types of reinforcement used shall be selected.

2. $\xi_b = x_b/h_0$, x_b is the depth of the rectangular compressive stress block under the balanced condition (longitudinal tension reinforcement and concrete in the compression zone reach their design strength at the same time).

5.2.2 For flexural members arranged only with longitudinal internal reinforcements and with rectangular sections or T-sections whose flanges are located on the tension sides, their flexural resistances on cross-section shall be calculated in accordance with the following provisions (Figure 5.2.2):

$$\gamma_0 M_d \leq f_{cd} b x\left(h_0 - \frac{x}{2}\right) + f'_{sd} A'_s (h_0 - a'_s) + (f'_{pd} - \sigma'_{p0}) A'_p (h_0 - a'_p) \qquad (5.2.2\text{-}1)$$

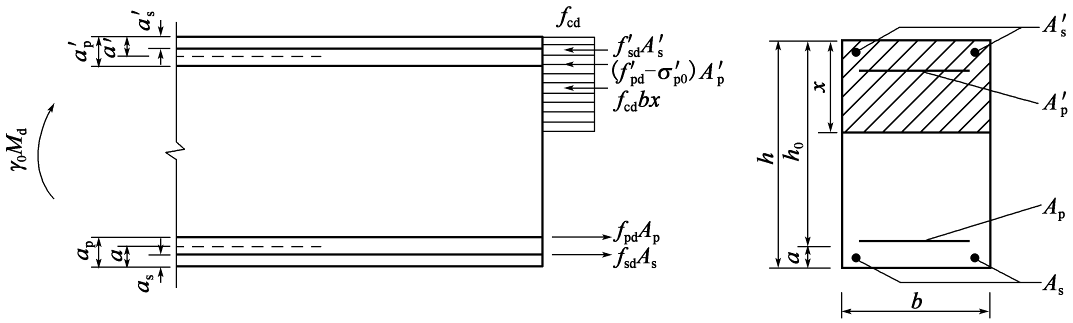


Figure 5.2.2 Flexural resistance on cross-section of rectangular members

Depth of the compression zone, x, shall be calculated by the following equation:

$$f_{sd}A_s + f_{pd}A_p = f_{cd}bx + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_p \tag{5.2.2-2}$$

Depth of the compression zone, x, shall meet the following requirements:

$$x \leqslant \xi_b h_0 \tag{5.2.2-3}$$

When the compression zone is arranged with longitudinal reinforcing steel and prestressing steel, and the prestressing steel is in compression, *i. e.*, $(f'_{pd} - \sigma'_{p0})$ is positive:

$$x \geqslant 2a' \tag{5.2.2-4}$$

When the compression zone is arranged only with longitudinal reinforcing steel, or with both the reinforcing steel and the prestressing steel while the prestressing steel is in tension, *i. e.*, $(f'_{pd} - \sigma'_{p0})$ is negative:

$$x \geqslant 2a'_s \tag{5.2.2-5}$$

where:

γ_0—importance factor of structure for bridge and culvert, which is taken from Clause 5.1.2 in the *Specifications*;

M_d—design bending moment, which is calculated according to the provisions in Clause 5.1.2 of the *Specifications*;

f_{cd}—design axial compressive strength of concrete, which is taken from Table 3.1.4 of the *Specifications*;

f_{sd}, f'_{sd}—design tensile strength and design compressive strength of longitudinal reinforcing steel, respectively, which is taken from Table 3.2.3-1 of the *Specifications*;

f_{pd}, f'_{pd}—design tensile strength and design compressive strength of longitudinal prestressing steel, respectively, which is taken from Table 3.2.3-2 of the *Specifications*;

A_{sd}, A'_{sd}— cross-sectional area of longitudinal reinforcing steel in the tension zone and the compression zone, respectively;

A_p, A'_p— cross-sectional area of longitudinal prestressing steel in the tension zone and the compression zone, respectively;

b—width of rectangular section or web width of T-section;

h_0—effective depth of the section, $h_0 = h - a$, where h is the total depth of the section;

a, a'—distance from the resultant force in reinforcing steel or prestressing steel in the tension zone or compression zone to the edge of the tension zone or compression zone, respectively;

a'_s, a'_p—distance from the resultant force in reinforcing steel and prestressing steel in the compression zone to the edge of compression zone, respectively;

σ'_{p0}—stress of prestressing steel when the normal stress of concrete at the point of the resultant force in the prestressing steel in the compression zone equals to zero. The prestressing steel stress in pretensioned members is calculated according to Eq. (6.1.5-2) of the *Specifications*, and that in post-tensioned members is calculated according to Eq. (6.1.5-5) of the *Specifications*.

5.2.3 For T-section or I-section flexural members arranged only with longitudinal internal reinforcements and having flanges located at the compression zones, their flexural resistances on cross-section shall be calculated in accordance with the following provisions:

1 If the following requirement is satisfied:

$$f_{sd}A_s + f_{pd}A_p \leqslant f_{cd}b'_f h'_f + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_p \tag{5.2.3-1}$$

The flexural resistance shall be calculated according to the relevant equations in Clause 5.2.2 of the *Specifications* by considering a rectangular section with a width of b'_f [Figure 5.2.3a)].

2 If the requirement of Eq. (5.2.3-1) is not satisfied, the calculation shall be carried out according to the following provisions [Figure 5.2.3b)]:

$$\gamma_0 M_d \leqslant f_{cd}\left[bx\left(h_0 - \frac{x}{2}\right) + (b'_f - b)h'_f\left(h_0 - \frac{h'_f}{2}\right)\right] + f'_{sd}A'_s(h_0 - a'_s) + (f'_{pd} - \sigma'_{p0})A'_p(h_0 - a'_p) \tag{5.2.3-2}$$

The depth of compression zone, x, shall be calculated using the following equation, and shall meet the requirements of Eq. (5.2.2-3), and Eq. (5.2.2-4) or Eq. (5.2.2-5).

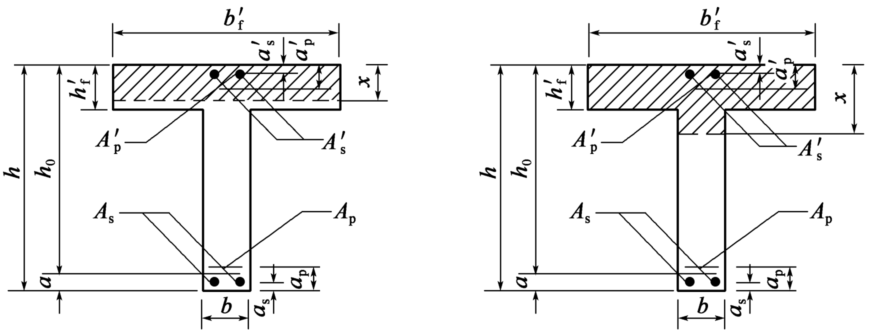
$$f_{sd}A_s + f_{pd}A_p = f_{cd}[bx + (b'_f - b)h'_f] + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_p \tag{5.2.3-3}$$

where:

h'_f—depth of compression flange of T-section or I-section;

b'_f—effective width of compression flange of T-section or I-section, which shall be determined according to Clause 4.3.3 of the *Specifications*.

For box members arranged only with longitudinal internal reinforcement, their flexural resistances on cross-section may be calculated with reference to this clause.



a)Calculation for rectangular section when $x \leqslant h'_f$ b)Calculation for T-section when $x > h'_f$

Figure 5.2.3 Calculation diagram for resistanceon cross-section of T-section flexural member

(the directions of internal forces are the same as those in Figure 5.2.2)

5.2.4 When the longitudinal reinforcement in the compression zone is taken into account in the calculation while the conditions in Eq. (5.2.2-4) or Eq. (5.2.2-5) of the *Specifications* are not satisfied, the flexural resistances on cross-section of the members arranged with only longitudinal internal reinforcement shall comply with the following provisions (Figure 5.2.2):

1 When longitudinal reinforcing steel and prestressing steel are arranged in the compression zone and the prestressing steel is in compression:

$$\gamma_0 M_d \leqslant f_{pd} A_p (h - a_p - a) + f_{sd} A_s (h - a_s - a') \tag{5.2.4-1}$$

2 When only longitudinal reinforcing steel is arranged in the compression zone, or both the reinforcing steel and the prestressing steel are arranged while the prestressing steel is in tension:

$$\gamma_0 M_d \leqslant f_{pd} A_p (h - a_p - a'_s) + f_{sd} A_s (h - a_s - a'_s) - (f'_{pd} - \sigma'_{p0}) A'_p (a'_p - a'_s) \tag{5.2.4-2}$$

where:

a_s, a_p—distance from the resultant force in reinforcing steel and prestressing steel (both in the tension zone) to the bottom edge of the tension zone, respectively.

5.2.5 For T-section flexural beams arranged with longitudinal external tendons, their flexural resistances on cross-section shall be calculated in accordance with the following provisions:

1 Flanges in tension:

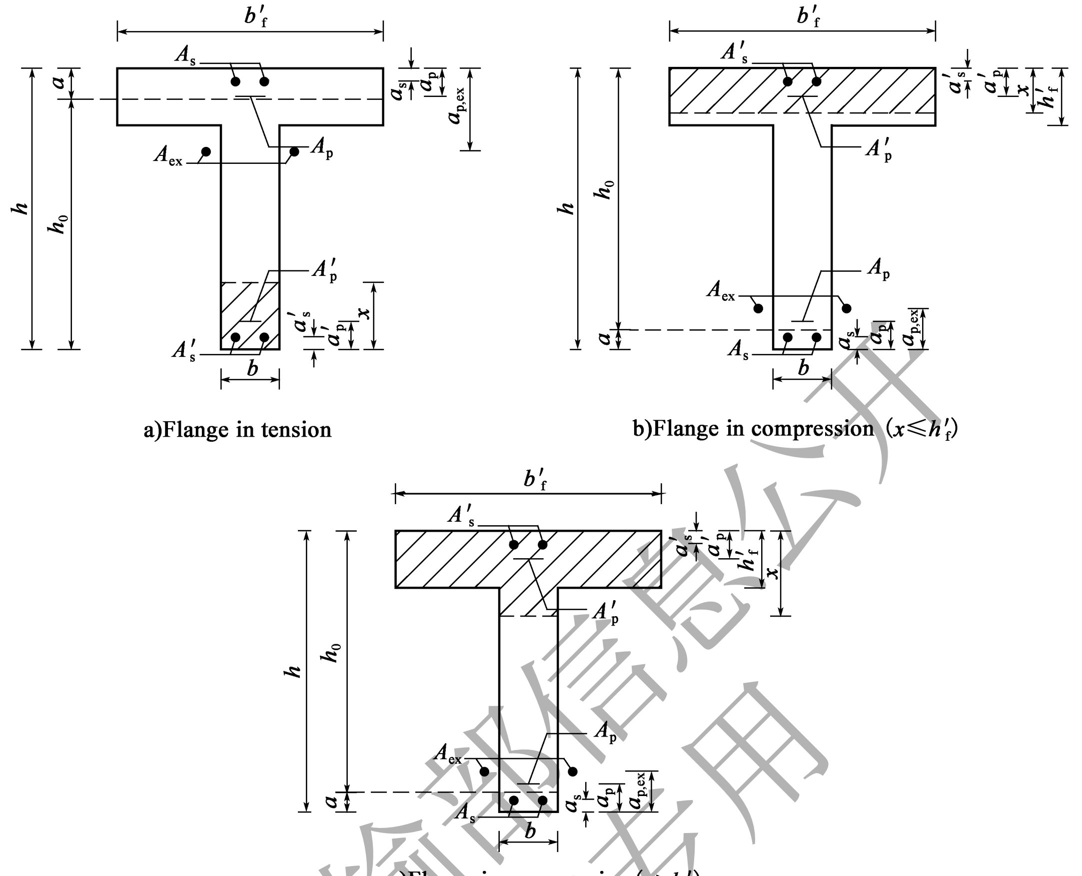
$$\gamma_0 M_d \leqslant f_{cd} bx \left(h_0 - \frac{x}{2} \right) + f'_{sd} A'_s (h_0 - a'_s) + (f'_{pd} - \sigma'_{p0}) A'_p (h_0 - a'_p) \tag{5.2.5-1}$$

$$f_{sd} A_s + f_{pd} A_p + \sigma_{pe,ex} A_{ex} = f_{cd} bx + f'_{sd} A'_s + (f'_{pd} - \sigma'_{p0}) A'_p \tag{5.2.5-2}$$

where:

$\sigma_{pe,ex}$—effective stress in the external tendons after subtracting loss of prestress in service stage, which shall be calculated according to Clauses 6.1.6 of the *Specifications*;

A_{ex}—cross-sectional area of the external tendons.



a)Flange in tension

b)Flange in compression ($x \leqslant h'_f$)

c)Flange in compression ($x > h'_f$)

Figure 5.2.5 Calculation of flexural resistance on cross-section of T-section members arranged with external tendons

a—distance from the resultant force in reinforcing steel, internal and external tendons in the tension zone to the bottom edge of the tension zone

2 Flange in compression:

1) If $f_{sd}A_s + f_{pd}A_p + \sigma_{pe,ex}A_{ex} \leqslant f_{cd}b'_f h'_f + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_p$,

$$\gamma_0 M_d \leqslant f_{cd} b'_f x\left(h_0 - \frac{x}{2}\right) + f'_{sd}A'_s(h_0 - a'_s) + (f'_{pd} - \sigma'_{p0})A'_p(h_0 - a'_p) \qquad (5.2.5\text{-}3)$$

$$f_{sd}A_s + f_{pd}A_p + \sigma_{pe,ex}A_{ex} = f_{cd}b'_f x + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_p \qquad (5.2.5\text{-}4)$$

where:

h'_f—depth of compression flange of T-section;

b'_f—effective width of compression flange of T-section, which is taken from the provisions in Clause 4.3.3 of the *Specifications*.

2) If $f_{sd}A_s + f_{pd}A_p + \sigma_{pe,ex}A_{ex} > f_{cd}b'_f h'_f + f'_{sd}A'_s + (f'_{pd}\sigma'_{p0})A'_p$,

$$\gamma_0 M_d \leqslant f_{cd}\left[bx\left(h_0 - \frac{x}{2}\right) + (b'_f - b)h'_f\left(h_0 - \frac{h'_f}{2}\right)\right] + f'_{sd}A'_s(h_0 - a'_s) +$$

$$(f'_{pd} - \sigma'_{p0})A'_p(h_0 - a'_p) \tag{5.2.5-5}$$

$$f_{sd}A_s + f_{pd}A_p + \sigma_{pe,ex}A_{ex} = f_{cd}[bx + (b'_f - b)h'_f] + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_p \tag{5.2.5-6}$$

The depth of compression zone, x, calculated by Eq. (5.2.5-2), Eq. (5.2.5-4) or Eq. (5.2.5-6), shall comply with the requirements in Eq. (5.2.2-3), Eq. (5.2.2-4) or Eq. (5.2.2-5) of the *Specifications*, respectively.

Flexural resistance on cross-section of box members arranged with longitudinal external tendons may be calculated in accordance with this clause.

5.2.6 When the longitudinal reinforcement in the compression zone is taken into account in the calculation while the conditions in Eq. (5.2.2-4) and Eq. (5.2.2-5) of the *Specifications* are not satisfied, the flexural resistances on cross-section of the members arranged with longitudinal external tendons shall comply with the following provisions (Figure 5.2.5):

1 When longitudinal reinforcing steel and prestressing steel are arranged in the compression zone and the internal prestressing steel is in compression:

$$\gamma_0 M_d \leqslant f_{pd}A_p(h - a_p - a'_s) + f_{sd}A_s(h - a_s - a'_s) + \sigma_{pe,ex}A_{ex}(h - a_{p,ex} - a') \tag{5.2.6-1}$$

2 When only longitudinal reinforcing steel is arranged in the compression zone, or both the reinforcing steel and the internal prestressing steel are arranged while the prestressing steel is in tension:

$$\gamma_0 M_d \leqslant f_{pd}A_p(h - a_p - a'_s) + f_{sd}A_s(h - a_s - a'_s) + \sigma_{pe,ex}A_{ex}(h - a_{p,ex} - a') - (f'_{pd} - \sigma'_{p0})A'_p(a'_p - a'_s) \tag{5.2.6-2}$$

where:

$a_{p,ex}$—distance from the resultant force in external tendons to the bottom edge of the tension zone.

5.2.7 In calculating flexural resistance on cross-section of a flexural member, if the requirement of Eq. (5.2.2-3) is not satisfied, the depth of the cross-sectional compression zone of concrete, x, may be re-computed, in which the longitudinal reinforcement arranged by the detailing requirements may not be taken into account.

5.2.8 In calculating the shear resistance on inclined section of a flexural member, the investigated position shall be adopted according to the following provisions:

1 For simply supported beams or beam portions of continuous beams close to end supports

1) Sections at a distance of $h/2$ from the centerline of support [Section 1-1 in Figure 5.2.8a)];

2) Sections at the bent-up position of a bent bar in the tension zone [Section 2-2 and Section 3-3 in Figure 5.2.8a)];

3) Sections where the longitudinal reinforcement anchored in the tension zone starts to bear no forces [Section 4-4 in Figure 5.2.8a)];

4) Sections with the change of quantity or stirrup spacing [Section 5-5 in Figure 5.2.8a)];

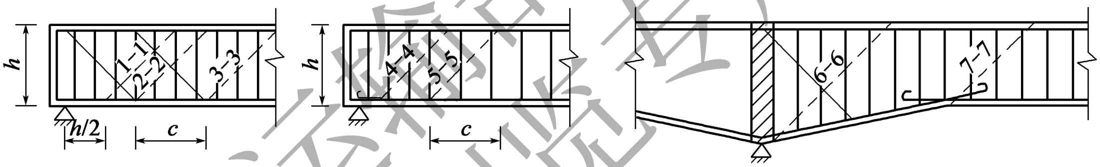
5) Sections at a location where the web width changes.

2 For cantilever beam or beam portions of continuous beam close to interior supports

1) Sections at the edge of the diaphragm on support [Section 6-6 in Figure 5.2.8b)];

2) Sections where the depth of the beam with variable depth changes abruptly [Section 7-7 in Figure 5.2.8b)];

3) Sections required to be checked as those in simply supported beams.



a) Simply supported beams or beam portions of continuous beams close to end supports

b) Cantilever beam or beam portions of continuous beam close to interior supports

Figure 5.2.8 Schematic diagram of checking position for shear resistance on the inclined section

5.2.9 When rectangular, T-section and I-section flexural members are arranged with vertical prestressing steel, stirrups and bent bars, their shear resistances on inclined sections shall be calculated in accordance with the following provisions (Figure 5.2.9):

$$\gamma_0 V_d \leqslant V_{cs} + V_{sb} + V_{pb} + V_{pb,ex} \tag{5.2.9-1}$$

$$V_{cs} = 0.45 \times 10^{-3} \alpha_1 \alpha_2 \alpha_3 b h_0 \sqrt{(2 + 0.6P)\sqrt{f_{cu,k}}(\rho_{sv} f_{sv} + 0.6\rho_{pv} f_{pv})} \tag{5.2.9-2}$$

$$V_{sb} = 0.75 \times 10^{-3} f_{sd} \sum A_{sb} \sin\theta_s \tag{5.2.9-3}$$

$$V_{pb} = 0.75 \times 10^{-3} f_{pd} \sum A_{pb} \sin\theta_p \tag{5.2.9-4}$$

$$V_{pb,ex} = 0.75 \times 10^{-3} \sum \sigma_{pe,ex} A_{ex} \sin\theta_{ex} \tag{5.2.9-5}$$

where:

V_d—design shear force (kN), which is taken from the cross-section corresponding to the inclined shear compression zone;

V_{cs}—design shear resistance in inclined section shared by concrete and stirrups (kN);

V_{sb}—design shear resistance of bent bar intersecting with the inclined section (kN);

V_{pb}—design shear resistance of internal curved tendon intersecting with the inclined section (kN);

$V_{pb,ex}$—design shear resistance of external curved tendon intersecting with the inclined section (kN);

α_1—influence coefficient due to bending moment with opposite sign. In calculating the shear resistance of the simply supported beam or beam portions of continuous beam close to supports or end supports, $\alpha_1 = 1.0$; in calculating the shear resistance of cantilever beam or beam portions of continuous beam close to interior supports, $\alpha_1 = 0.9$;

α_2—adjustment coefficient for the use of prestress. It is 1.25 for prestressed concrete flexural members. But it is 1.0 for the case that the direction of the sectional bending moment caused by the resultant force in prestressing steel is the same as that of the external bending moment, or for partially prestressed concrete flexural members in which cracks are permitted. It is also 1.0 for reinforced concrete flexural members;

α_3—influence coefficient due to the presence of a flange in compression. For rectangular section, $\alpha_3 = 1.0$; for T-section or I-section, $\alpha_3 = 1.1$;

b—width of the cross-section corresponding to the diagonal shear compression zone (mm), *i.e.*, width of rectangular section, or width of the web of T-section and I-section;

h_0—effective sectional depth, which is taken as the distance from the resultant force in longitudinal tension reinforcement to the extreme compression fiber in the cross-section corresponding to diagonal shear compression zone;

P—reinforcement percentage for longitudinal tension reinforcement inside the inclined section, $P = 100\rho$, $\rho = (A_p + A_s)/bh_0$; when $P > 2.5$, takes $P = 2.5$;

$f_{cu,k}$—characteristic compressive strength of 150 mm concrete cubes (MPa);

ρ_{sv}, ρ_{pv}—reinforcement ratio for stirrup and vertical prestressing steel in inclined section, respectively, $\rho_{sv} = A_{sv}/(s_v b)$, $\rho_{pv} = A_{pv}/(s_p b)$.

f_{sv}, f_{pv}—design tensile strength of stirrup and vertical prestressing steel, respectively, which is taken from Table 3.2.3-1 and Table 3.2.3-2 of the *Specifications*;

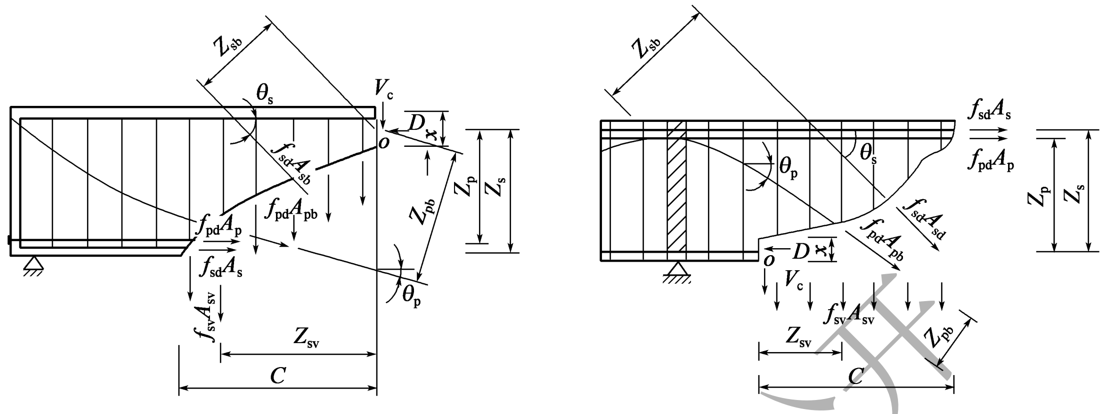
A_{sv}, A_{pv}—total cross-sectional area of stirrup and vertical prestressing steel within the inclined section, respectively (mm^2);

s_v, s_p—spacing of stirrups and vertical prestressing steels within the inclined section, respectively (mm);

$\sigma_{pe,ex}$—effective stress of external tendon after subtracting loss of prestress in service stage, which is calculated by Clause 6.1.6 of the *Specifications*;

A_{sb}, A_{pb}, A_{ex}—cross-sectional area of bent bars, internal curved tendons and external curved tendons at the same bent-up plane within the inclined section (mm^2), respectively;

θ_s, θ_p, θ_e—angle between the horizontal line and the tangent line of bent bars, internal curved tendons, and external curved tendons, respectively (in degrees).



a) Simply supported beams or beam portions of continuous beams close to end supports

b) Cantilever beam or beam portions of continuous beam close to interior supports

Figure 5.2.9 Calculation diagram for shear resistance on inclined sections

The shear resistance on inclined section of flexural members with box sections may be calculated in accordance with the provisions of this clause.

5.2.10 In checking the resistance on inclined section, the length of the horizontal projection of the inclined section (Figure 5.2.9), C, shall be calculated by Eq. (5.2.10):

$$C = 0.6mh_0 \tag{5.2.10}$$

where:

m—generalized shear span-to-depth ratio, which is calculated using M_d and V_d of the cross-section corresponding to the diagonal shear compression zone, $m = M_d/(V_d h_0)$; if $m > 3.0$, then $m = 3.0$ is taken;

h_0—effective sectional depth, which is taken as the distance from the resultant force in longitudinal tension reinforcement to the extreme compression fiber in the cross-section corresponding to diagonal shear compression zone;

M_d—design bending moment corresponding to V_d in Clause 5.2.9 of the *Specifications*.

5.2.11 For flexural members of rectangular sections, T-sections, or I-sections, their sections resisting shear shall comply with the following requirements:

$$\gamma_0 V_d \leqslant 0.51 \times 10^{-3} \sqrt{f_{cu,k}}\, bh_0 \tag{5.2.11}$$

where:

V_d—design shear force, which is the most unfavorable value of the checked inclined section;

$f_{cu,k}$—characteristic compressive strength of 150 mm concrete cubes (MPa);

b—width of rectangular section (mm) or the web width of T-section or I-section. The minimum width in the range of the checked inclined section is taken for it (mm);

h_0—effective depth (mm), *i. e.*, the distance from the resultant force in longitudinal tension reinforcement to the extreme compression fiber, which is taken as the minimum effective depth in the range of the checked inclined section.

For continuous beams with variable depth (haunched), besides the cross-sectional dimensions of the beam portions near the end supports, the cross-sectional dimensions where the cross-sections changed abruptly shall also be checked.

5.2.12 For flexural members with rectangular sections, T-sections, or I-sections, if the following provisions are satisfied, the shear resistance on the inclined section may not be checked, and stirrups are designed based on the detailing requirements in Clause 9.3.12 of the *Specifications*.

$$\gamma_0 V_d \leqslant 0.50 \times 10^{-3} \alpha_2 f_{td} b h_0 \tag{5.2.12}$$

where:

f_{td}—design axial tensile strength of concrete (MPa), which is taken according to the provisions in Table 3.1.4 of the *Specifications*.

For slab-type flexural members without stirrup, the calculated value on the right-hand side of Eq. (5.2.12) may be multiplied by a factor of 1.25.

Note: The unit and meaning of V_d, b and h_0 in Eq. (5.2.12) refer to Clause 5.2.11 of the *Specifications*

5.2.13 For reinforced concrete flexural members with rectangular sections, T-sections, or I-sections, in the design of their reinforcement for shear resistance of inclined sections, stirrups and bent bars shall be calculated and arranged according to the following provisions:

1 The envelope of design shear force is drawn first. The most unfavorable design shear force (to be used for the design of shear reinforcement) shall be determined according to the following provisions: the design shear force of the section $h/2$ from support [Figure 5.2.13a)], V'_d, shall be adopted for simply supported beam or the beam portion of the continuous beam close to the end supports; the design shear force of the section at the edge of the diaphragm on the support [Figure 5.2.13b)], V'_d, shall be adopted for beam portion close to interior supports in a continuous or cantilever beam with constant depth; the design shear force at the turning section from constant depth to variable depth [Figure 5.2.13c)], V_d^0, shall be adopted for the beam portion close to interior supports in continuous or cantilever beam with variable depth (haunched). No less than 60% V'_d or V_d^0 shall be shared by concrete and stirrups, and no more than 40% shall be resisted by the bent bars, and these two parts in the envelope of design shear force are divided by a horizontal line.

2 The type and diameter of stirrups may be pre-chosen. Their spacing, s_v (mm), may be

calculated using the following equation:

$$s_{v} = \frac{0.2 \times 10^{-6} \alpha_1^2 \alpha_3^2 (2 + 0.6P) \sqrt{f_{cu,k}} A_{sv} b h_0^2}{(\xi \gamma_0 V_d)^2} \tag{5.2.13-1}$$

where:

V_d—most unfavorable design shear force used for the design of shear reinforcement (kN). In calculating stirrup spacing for the simply supported beam or the beam portion of a continuous beam close to end supports, the constant depth cantilever beam or beam portion of a continuous beam close to interior supports, let [Figure 5.2.13a), b)]; in calculating stirrup spacing for the variable depth (haunched) cantilever beam or beam portion of a continuous beam close to interior supports, let [Figure 5.2.13c)];

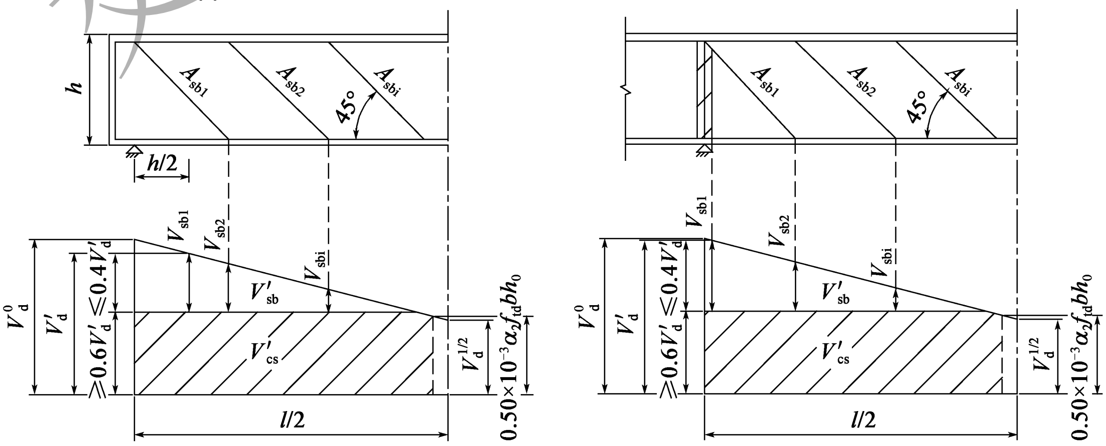
ξ—distribution coefficient to distribute the most unfavorable design shear force to concrete and stirrup in the design of shear reinforcement, $\xi \geqslant 0.6$ is taken;

h_0—effective depth of the section to resist the most unfavorable design shear force in the design of shear reinforcement (mm);

b—beam web width of the section to resist the most unfavorable design shear force in the design of shear reinforcement (mm), which is taken as the minimum web width for the beam with variable web width;

A_{sv}—total cross-sectional area of the stirrup in one section (mm^2).

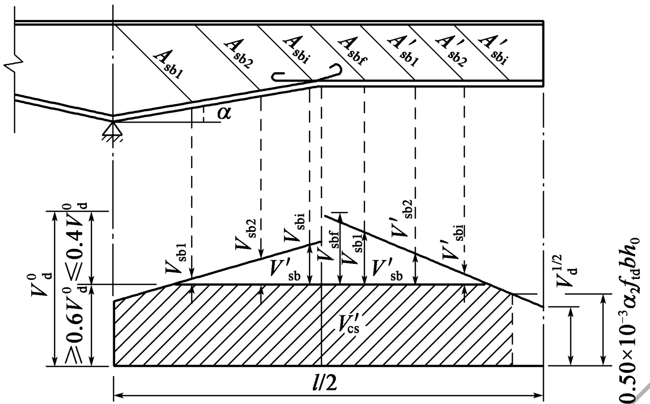
3 In calculating the area of the first row of the bent bar, A_{sb1}, the shear force V_{sb1} resisted by the bent bar at the section $h/2$ from support may be adopted [Figure 5.2.13a)] for a simply supported beam or the beam portion close to end support of a continuous beam. The shear force V_{sb1} resisted by the bent bar located at the diaphragm edge at support is adopted [Figure 5.2.13b)] for the beam portions close to interior support in a continuous or cantilever beam with constant depth. The shear force V_{sb1} resisted by the bent bar at the bottom bent position closest to the support is adopted [Figure 5.2.13c)] for the beam portion with variable cross-sections close to interior supports in a continuous or cantilever beam with variable cross-sections.



a) Simply supported beam or the beam portion of continuous beam close to the end support

b) Cantilever beam or beam portion of a continuous beam close to interior supports, both with constant depth (haunched)

Figure 5.2.13



c)Cantilever beam or beam portion of a continuous beam close to interior supports, both with variable depth (haunched)

Figure 5.2.13 Design and calculation of reinforcing steel for shear resistance of the inclined section

In the above figures:

V_d^0—most unfavorable design shear force caused by actions;

V'_d—the most unfavorable design shear force used in the design of shear reinforcement. The value at $h/2$ from support is adopted for simply supported beam or beam portion close to the end support in a continuous beam; the value on the diaphragm edge at support is adopted for the beam portion close to the interior supports in a continuous or cantilever beam with constant depth;

$V_d^{l/2}$—design shear force at mid-span;

V'_{cs}—total design shear force resisted by concrete and stirrup (shadow area in the diagram);

V'_{sb}—total design shear force resisted by the bent bar;

V_{sb1}, V_{sb2}, V_{sb3}—design shear force resisted by the bent bar in the simply supported beam, in the continuous beam or the cantilever beam with constant depth, in the beam portion with variable depth of continuous or cantilever beam with variable depth (haunched), respectively;

V_{sbf}—design shear force resisted by the bent bar at the turning section from constant depth to variable depth in a continuous or cantilever beam with variable depth (haunched);

V'_{sb1}, V'_{sb2}, V'_{sbi}—design shear force resisted by the bent bar at the beam portion with constant depth for a continuous beam or cantilever beam with variable depth (haunched);

A_{sb1}, A_{sb2}, A_{sbi}—cross-sectional area of the first, second and ith row bent bar from the support point, respectively, in the simply supported beam, in the continuous beam or the cantilever beam with a constant depth, in the beam portion with variable depth of continuous beam or cantilever beam with variable depth (haunched);

A_{sbf}—cross-sectional area of the bent bar across the turning section from constant depth to variable depth in continuous or cantilever beams with variable depths (haunched);

A'_{sb1}, A'_{sb2}, A'_{sbi}—cross-sectional area of the first, second and ith row bent bar from the turning section from constant depth to variable depth in continuous or cantilever beam with variable depth (haunched), respectively;

h—depth of beam with constant depth;

l—effective span of the beam;

α—angle between the horizontal line and the bottom profile of the beam portion with variable depth.

4 In calculating each row of the bent bar after the first row of the bent bar, *i. e.*, A_{sb2}, …, A_{sbi}, the shear force V_{sb2}, …, V_{sbi} resisted by the former row of the bent bar in the bottom bent position is adopted [Figure 5. 2. 13a), b)] for simply supported beam, the beam portion of continuous beam close to end supports, the beam portion close to interior supports in continuous or cantilever beam with constant depth; the shear force V_{sb2}, …, V_{sbi} resisted by each row of the bent bar at the bottom bent position is adopted [Figure 5.2.13c)] for the beam portion with variable depth close to interior supports in continuous or cantilever beam with variable depth (haunched).

5 In calculating A_{sbf} of the bent bar across the turning section from constant depth to variable depth in the continuous or cantilever beam with variable depth (haunched), the part of the peak shear force V_{sbf} resisted by the bent bar at the turning section is adopted [Figure 5. 2. 13c)]; in calculating A'_{sb1}, A'_{sb2}, A'_{sbi} of each row of the bent bar of the beam portion with a constant depth, the shear force V'_{sb1}, V'_{sb2}, V'_{sbi} resisted by each row of the bent bar at the top bent position is adopted [Figure 5.2.13c)].

6 The cross-sectional area of each row of bent bar is calculated by the following equation:

$$A_{sb} = \frac{\gamma_0 V_{sb}}{0.75 \times 10^{-3} f_{sd} \sin\theta_s} \tag{5.2.13-2}$$

where:

A_{sb}—total cross-sectional area of each row of the bent bar (mm^2), *i. e.*, A_{sb1}, A_{sb2}, A_{sbi} or A'_{sb1}, A'_{sb2}, A_{sbf} in Figure 5.2.13;

V_{sb}—design shear force resisted by each row of the bent bar (kN), *i. e.*, V_{sb1}, V_{sb2}, V_{sbi} or V'_{sb1}, V'_{sb2}, V_{sbf} in Figure 5.2.13.

Note: $f_{cu,k}$, f_{sv} and f_{sd} are in MPa.

5.2.14 For flexural members with rectangular sections, T-sections or I-section, the flexural resistance on the inclined section shall be checked by the following provisions (Figure 5.2.9):

$$\gamma_0 M_d \leqslant f_{sd} A_s Z_s + f_{pd} A_p Z_p + \sum f_{sd} A_{sb} Z_{sb} + \sum f_{pd} A_{pd} Z_{pd} + \sum f_{sv} A_{sv} Z_{sv} \tag{5.2.14-1}$$

Herein the length of the horizontal projection line of the most unfavorable inclined section is determined by trial calculation according to the following formula:

$$\gamma_0 M_d \leqslant \sum f_{sd} A_{sb} \sin\theta_s + \sum f_{pd} A_{pd} \sin\theta_p + \sum f_{sv} A_{sv} \tag{5.2.14-2}$$

where:

M_d—design bending moment, which is taken from the vertical section corresponding to the inclined shear compression zone;

V_d—design shear force corresponding to design bending moment M_d;

Z_s, Z_p—distance from the resultant force in tension reinforcing steel or tension prestressing steel to the central point O of the compression zone, respectively;

Z_{sb}, Z_{pb}—distance from the resultant force in bent bar or curved tendon which are in the same bent-up plane intersected with inclined section to the central point O of the compression zone, respectively;

Z_{sv}—horizontal distance from the resultant force in the stirrup of the same plane intersected with the inclined section to the compression zone at the inclined section.

The depth of the compression zone at the inclined section, x, is obtained from the equilibrium condition that the sum of all forces on the inclined section projected along the longitudinal axis of the member equals zero.

For the longitudinal reinforcement and stirrup of flexural members, when the requirements in Clause 9.1.4, Clause 9.3.8 and Clause 9.3.12 of the *Specifications* are satisfied, flexural resistance on the inclined section may not be checked.

5.3 Compression Members

5.3.1 For axially loaded reinforced concrete compression members with stirrups (or spirals, or transverse reinforcement welded on longitudinal reinforcement) (Figure 5.3.1), the compressive resistance on cross-section shall comply with the following provision:

$$\gamma_0 N_d \leqslant 0.9\varphi (f_{cd} A + f'_{sd} A'_s) \tag{5.3.1}$$

where:

N_d—design axial force;

φ—stability factor of axially loaded compression member, which is taken from Table 5.3.1 of the *Specifications*;

A—gross area of the section of a member, when the longitudinal reinforcement ratio is greater than 3%, A shall be substituted by $A_n = A - A'_s$;

A'_s—total cross-sectional area of longitudinal reinforcement.

Table 5.3.1 Stability factor of axially loaded reinforced concrete compression members

l_0/b	≤8	10	12	14	16	18	20	22	24	26	28
$l_0/2r$	≤7	8.5	10.5	12	14	15.5	17	19	21	22.5	24
l_0/i	≤28	35	42	48	55	62	69	76	83	90	97
φ	1.0	0.98	0.95	0.92	0.87	0.81	0.75	0.70	0.65	0.60	0.56
l_0/b	30	32	34	36	38	40	42	44	46	48	50
$l_0/2r$		28	29.5	31	33	34.5	36.5	38	40	41.5	43
l_0/i		111	118	125	132	139	146	153	160	167	174
φ	0.52	0.48	0.44	0.40	0.36	0.32	0.29	0.26	0.23	0.21	0.19

Note: In the table, l_0 is the effective length of the member; b is the minor length of the rectangular section; r is the radius of the circular section; i is the minimum radius of the gyration of the section.

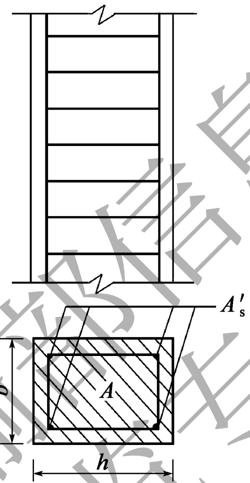


Figure 5.3.1 Axially loaded reinforced concrete compression member with stirrups

5.3.2 For an axially loaded reinforced concrete compression member with slenderness ratio $l_0/i \leq 48$, if it is designed with confinement reinforcement of spiral or welded ring type (as shown in Figure 5.3.2), the transformed sectional area of the confinement reinforcement, A_{so}, is not less than 25% of the total cross-sectional area of longitudinal reinforcement, and the spacing is not larger than 80 mm or $d_{cor}/5$, then the compressive resistance on the cross-section of the member shall comply with the following provision:

$$\gamma_0 N_d \leq 0.9(f_{cd}A_{cor} + f'_{sd}A'_s + kf_{sd}A_{so}) \tag{5.3.2-1}$$

$$A_{so} = \frac{\pi d_{cor} A_{so1}}{s} \tag{5.3.2-2}$$

where:

A_{cor}—cross-sectional area of concrete core;

A_{so}—transformed sectional area of confinement reinforcement;

d_{cor}—diameter of concrete core;

k—influence coefficient of confinement reinforcement, for concrete strength class equal to or lower than C50, $k = 2.0$; for concrete strength class between C50 and C80, $k = 2.0$

~1.7, which is obtained by linear interpolation;

A_{sol}—cross-sectional area of a single confinement reinforcement;

s—spiral or interval spacing of confinement reinforcement along the axial direction of the member.

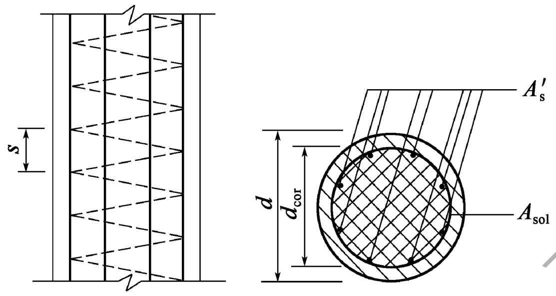


Figure 5.3.2 Axially loaded reinforced concrete compression member with spiral confinement reinforcement

If the transformed sectional area, spacing and slenderness ratio of confinement reinforcement do not match the conditions in this clause, or the compressive resistance on cross-section calculated by Eq. (5.3.2-1) is less than that calculated by Eq. (5.3.1), the confining effect of confinement reinforcement shall not be taken into account, and the compressive resistance on cross-section shall be calculated according to the provisions in Clause 5.3.1 of the *Specifications*.

The design compressive resistance calculated by Eq. (5.3.2-1) shall not be greater than 1.5 times that calculated by Eq. (5.3.1).

5.3.3 For eccentrically loaded compression members, the relative balanced depth of the compression zone, ξ_b, shall be used to determine if the eccentricity in the members is large or small, where ξ_b shall be determined by the following provisions:

1 For eccentrically loaded reinforced concrete compression members, ξ_b may be taken from Table 5.2.1 of the *Specifications*;

2 For eccentrically loaded prestressed concrete compression members, ξ_b may be calculated according to the following equations:

1) For prestressing threaded bars,

$$\xi_b = \frac{\beta}{1 + \dfrac{f_{pd} - \sigma_{p0}}{E_p \varepsilon_{cu}}} \qquad (5.3.3\text{-}1)$$

2) For steel wires or steel strands,

$$\xi_b = \frac{\beta}{1 + \frac{0.002}{\varepsilon_{cu}} + \frac{f_{pd} - \sigma_{p0}}{E_p \varepsilon_{cu}}} \tag{5.3.3-2}$$

where:

β—ratio of the depth of rectangular stress block to the physical depth of compression zone, which is taken from Table 5.1.4;

σ_{p0}—stress of prestressing steel when the normal stress of concrete is zero at point of the resultant force in longitudinal prestressing steel in the tension zone, which is calculated according to Eq. (6.1.5-2) or Eq. (6.1.5-5) of the *Specifications*;

ε_{cu}—ultimate compressive strain of concrete under nonuniform compression. For concrete strength class equal to C50 or below, $\varepsilon_{cu} = 0.0033$; for concrete strength class of C80, $\varepsilon_{cu} = 0.003$. For concrete strength class between C50 and C80, ε_{cu} can be calculated by linear interpolation;

f_{pd}—design tensile strength of prestressing steel;

E_p—modulus of elasticity of prestressing steel.

5.3.4 The compressive resistance of eccentrically loaded compression members with a rectangular section shall comply with the following provisions (Figure 5.3.4):

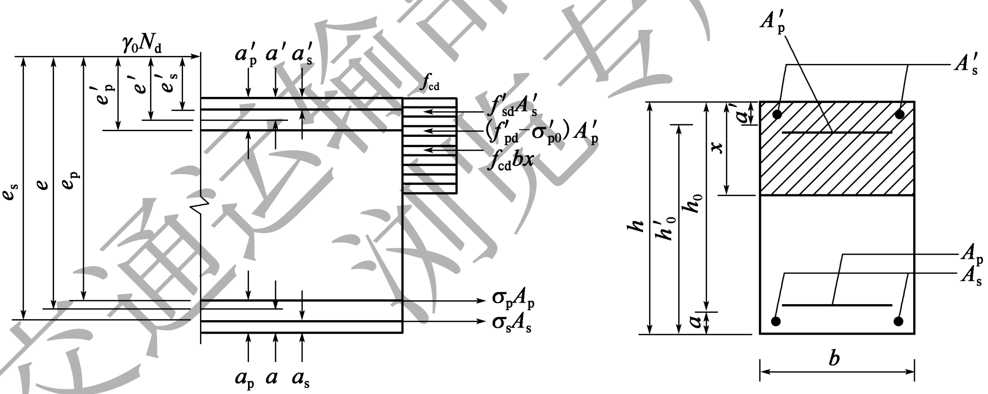


Figure 5.3.4 Calculation on compressive resistance of cross-section in eccentrically loaded compression members with rectangular section

$$\gamma_0 N_d \leq f_{cd}bx + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_P - \sigma_s A_s - \sigma_p A_p \tag{5.3.4-1}$$

$$\gamma_0 N_d e \leq f_{cd}bx\left(h_0 - \frac{x}{2}\right) + f'_{sd}A'_s(h_0 - a'_s) + (f'_{pd} - \sigma'_{p0})A'_P(h_0 - a'_p) \tag{5.3.4-2}$$

$$e = \eta e_0 + \frac{h}{2} - a \tag{5.3.4-3}$$

where:

e—distance from the axial force to the resultant force in longitudinal reinforcement (A_s or A_p) on the tensile side or on the minor compressive side;

e_0—eccentricity of axial force with respect to the gravity axis of cross-section, $e_0 = M_d/N_d$;

M_d—design bending moment corresponding to axial force;

h_0—distance from the extreme compression fiber of the major compressed side to the resultant force in longitudinal reinforcement on the tension side or on the minor compression side, $h_0 = h - a$;

η—amplification factor for eccentricity of axial force in an eccentrically loaded compression member, which is calculated according to the provisions in Clause 5. 3. 9 of the *Specifications*.

The stress of longitudinal reinforcementon the tensile side or minor compressive side of the section, σ_s or σ_p, shall be adopted according to the following cases:

If $\xi \leqslant \xi_b$, the member is in compression with a large eccentricity, then $\sigma_s = f_{sd}$, $\sigma_p = f_{pd}$, and the balanced depth of the compression zone, $\xi = x/h_0$;

If $\xi > \xi_b$, the member is in compression with a small eccentricity, then σ_s and σ_p are to be calculated according to provisions in Clause 5. 1. 5 of the *Specifications*.

If longitudinal compression reinforcement on the major compression side of the section is taken into account in the calculation of the compressive resistance, the depth of the compression zone shall meet the requirements of Eq. (5. 2. 2-4) and Eq. (5. 2. 2-5).

For a compression member with small eccentricity, when the axial force acts between the point of resultant force in longitudinal reinforcement of (A'_s and A'_p) and that of (A_s and A_p), its compressive resistance shall also comply with the following provisions:

$$\gamma_0 N_d e' \leqslant f_{cd} b h \left(h'_0 - \frac{h}{2}\right) + f'_{sd} A_s (h'_0 - a_s) + (f'_{pd} - \sigma_{p0}) A_p (h'_0 - a_p) \qquad (5.3.4\text{-}4)$$

$$e' = \frac{h}{2} - e_0 - a' \qquad (5.3.4\text{-}5)$$

where:

e'—distance from the axial force to the resultant force in longitudinal reinforcement of (A'_s and A'_p) on the major compression side of the cross-section. In the calculation, the influence of the amplification factor for eccentricity, η, on the eccentricity e_0 may be ignored;

h'_0—distance from the extreme compression fiber on the minor compression side of the cross-section to the resultant force in longitudinal reinforcement on the major compression side of the cross-section, $h'_0 = h - a'$;

For a compression member with small eccentricity and with a rectangular section reinforced symmetrically, the cross-sectional area of reinforcement may also be calculated by the following equations:

$$A_s = A'_s = \frac{\gamma_0 N_d e - \xi(1 - 0.5\xi) f_{cd} b h_0^2}{f'_{sd}(h_0 - a'_s)} \qquad (5.3.4\text{-}6)$$

Herein the balanced depth of the compression zone, ξ, may be calculated by the following equation:

$$\xi = \frac{\gamma_0 N_d - \xi_b f_{cd} b h_0}{\dfrac{\gamma_0 N_d e - 0.43 f_{cd} b h_0^2}{(\beta - \xi_b)(h_0 - a'_s)} + f_{cd} b h_0^2} + \xi_b \tag{5.3.4-6}$$

Note: For a compression member with small eccentricity, if the effective depth of the compression zone, x, is greater than h, *i. e.*, $x > h$, then h is used instead of x in calculating the compressive resistance of the member, but x is still used in calculating the reinforcement stress of σ_s and σ_p.

5.3.5 For an eccentrically loaded compression member with I-section or T-section, when the flange is located on the major compression side of the cross-section, the compressive resistance of the cross-section shall be calculated according to the following provisions:

1 If the depth of the compression zone $x \leqslant h'_f$, the compressive resistance shall be calculated as if the cross-section is a rectangular section with a width of b'_f;

2 If the depth of compression zone $x > h'_f$, the compressive resistance shall be calculated by the following formulas (Figure 5.3.5):

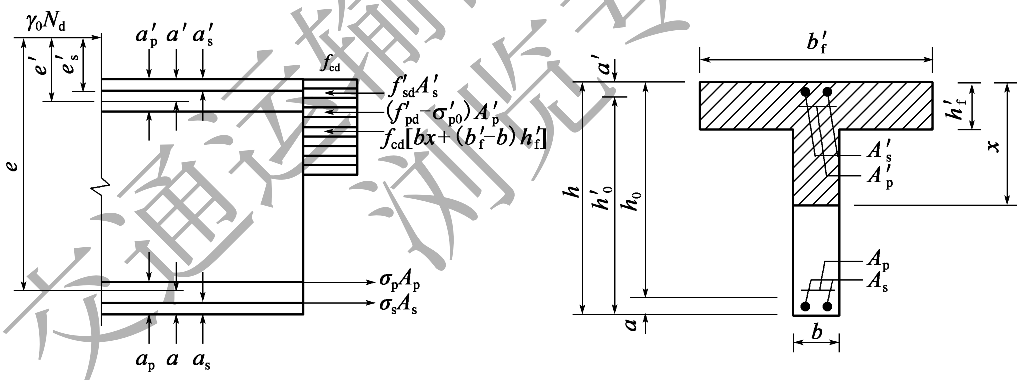


Figure 5.3.5 Calculation of compressive resistance of an eccentrically loaded compression member with a T-section

$$\gamma_0 N_d \leqslant f_{cd}[bx + (b'_f - b)h'_f] + f'_{sd}A'_s + (f'_{pd} - \sigma'_{p0})A'_P - \sigma_s A_s - \sigma_p A_p \tag{5.3.5-1}$$

$$\gamma_0 N_d e \leqslant f_{cd}\left[bx\left(h_0 - \frac{x}{2}\right) + (b'_f - b)h'_f\left(h_0 - \frac{h'_f}{2}\right)\right] + f'_{sd}A'_s(h_0 - a'_s) + (f'_{pd} - \sigma'_{p0})A'_P(h_0 - a'_p) \tag{5.3.5-2}$$

In determining the stresses of longitudinal reinforcement on the tension side or the minor compression side, σ_s or σ_p, as well as in designing the compression reinforcement on the major compression side of the section, the requirements for the depth of compression zone, x, shall comply with the provisions in Clause 5.3.4 of the *Specifications*.

For an I-section or T-sections member whose flange is located on the tension side or the minor compression side, if $x > h - h_f$, then the contribution of the compressive part of the flange shall be taken into account in the calculation of the compressive resistance.

In calculating the compressive resistance of a T-sections compression member with small eccentricity whose flange is located on the major compression side, if the axial force acts between the resultant force in longitudinal reinforcement of (A_s' and A_p') and that of (A_s and A_p), the following provisions shall also be complied with:

$$\gamma_0 N_d e' \leqslant f_{cd}\left[bh\left(h_0' - \frac{h}{2}\right) + (b_f' - b)h_f'\left(\frac{h_f'}{2} - a'\right)\right] + f_{sd}' A_s (h_0' - a_s) + (f_{pd}' - \sigma_{p0}) A_p (h_0' - a_p) \tag{5.3.5-3}$$

In calculating the compressive resistance of a T-sections compression member with small eccentricity whose flange is located on the minor compression side, the following provisions shall also be complied with:

$$\gamma_0 N_d e' \leqslant f_{cd}\left[bh\left(h_0' - \frac{h}{2}\right) + (b_f - b)h_f\left(h_0' - \frac{h_f}{2}\right)\right] + f_{sd}' A_s (h_0' - a_s) + (f_{pd}' - \sigma_{p0}) A_p (h_0' - a_p) \tag{5.3.5-4}$$

where:

b_f—width of flange on the minor compression side;

h_f—depth of flange on the minor compression side.

5.3.6 In calculating the compressive resistance of an eccentrically loaded compression member, the compressive resistance may be calculated by Eqs. (5.2.4-1) and (5.2.4-2) if the compressive reinforcement on the major compression side is taken into account, while the depth of compression zone does not meet the requirements of Eq. (5.2.2-4) or Eq. (5.2.2-5). Herein, M_d in the above formulas shall be substituted by $N_d e'$ or $N_d e_s'$, respectively, and the amplification factor for eccentricity, η, shall be taken into account in the calculation.

5.3.7 For an eccentrically loaded compression member with a rectangular section, I-section, or T-section, if longitudinal reinforcing steel is evenly arranged along the web depth and the number of each row of reinforcing steel is not less than four, its compressive resistance shall comply with the following provisions:

$$\gamma_0 N_d \leqslant f_{cd}[\xi b h_0 + (b_f' - b)h_f'] + f_{sd}' A_s' - \sigma_s A_s + N_{sw} \tag{5.3.7-1}$$

$$\gamma_0 N_d e \leqslant f_{cd}\left[\xi(1 - 0.5\xi) b h_0^2 + (b_f' - b)h_f'\left(h_0 - \frac{h_f'}{2}\right)\right] + f_{sd}' A_s'(h_0 - a_s') + M_{sw} \tag{5.3.7-2}$$

$$N_{sw} = \left(1 + \frac{\xi - \beta}{0.5\beta\omega}\right) f_{sw} A_{sw} \tag{5.3.7-3}$$

$$M_{sw} = \left[1 - \left(\frac{\xi - \beta}{\beta\omega}\right)^2\right] f_{sw} A_{sw} h_{sw} \quad (5.3.7\text{-}4)$$

where:

A_{sw}—total cross-sectional area of longitudinal reinforcement that evenly arranged along the web depth;

f_{sw}—design strength of longitudinal reinforcement that evenly arranged along the web depth;

N_{sw}—axial force resisted by longitudinal reinforcement that evenly arranged along the web depth, if $\xi = \frac{x}{h} > \beta$, then $N_{sw} = f_{sw} A_{sw}$;

M_{sw}—moment of internal force of longitudinal reinforcement that evenly arranged along the web depth with respect to the centroid of longitudinal reinforcement (A_s) at tension side or minor compression side, if $\xi > \beta$, then $M_{sw} = 0.5 f_{sw} A_{sw} h_{sw}$;

h_{sw}—depth of region evenly arranged with longitudinal reinforcement along the web, $h_{sw} = h_0 - a'_s$;

ω—ratio of the depth of region evenly arranged with longitudinal reinforcement along the web to the effective depth of the cross-section, and $\omega = h_{sw}/h_0$.

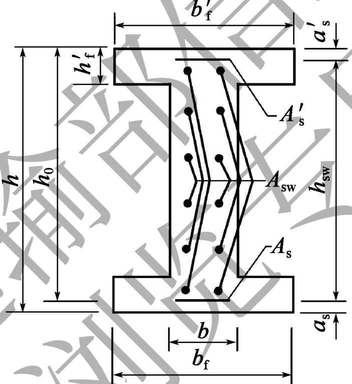


Figure 5.3.7 Compressive resistance of an eccentrically loaded compression member of I-section with longitudinal reinforcing steel evenly arranged along the web depth

For the reinforcement stress σ_s at the tension side or minor compression side in Eq. (5.3.7-1), if $\xi \leqslant \xi_b$, $\sigma_s = f_{sd}$; if $\xi > \xi_b$, it is calculated using Eq. (5.2.5-1).

If compression reinforcement (A'_s) on the major compression side is taken into account in the calculation, the depth of compression zone shall meet the requirement of $x \geqslant 2a'_s$; If the requirement is not met, its compressive resistance shall comply with the following provisions:

$$\gamma_0 N_d e' \leqslant f_{sd} A_s (h_0 - a'_s) + M'_{sw} \quad (5.3.7\text{-}5)$$

$$M'_{sw} = 0.5 f_{sw} A_{sw} h_{sw} \quad (5.3.7\text{-}6)$$

For an eccentrically loaded compression member with a T-section or I-section, if $x \leqslant h'_f$, the compressive resistance shall be calculated as if the section is a rectangular section with a width of b'_f. For an I-section member, if $x > h - h'_f$, the contribution of the compressive part of the flange at the minor compression side shall be taken into account in the calculation.

Note: if the computed ξ is greater than h/h_0, *i. e.* $\xi > h/h_0$, ξ in each formula in this clause shall be taken as $\xi = h/h_0$; however, the computed ξ is still used in calculating the stresses in the reinforcements of A_s.

5.3.8 For an eccentrically loaded circular compression member with longitudinal reinforcing steel evenly distributed along the perimeter (Figure 5.3.8), the compressive resistance shall comply with the following requirements:

$$\gamma_0 N_d \leqslant N_{ud} = \alpha f_{cd} A\left(1 - \frac{\sin 2\pi\alpha}{2\pi\alpha}\right) + (\alpha - \alpha_t) f_{sd} A_s \quad (5.3.8\text{-}1)$$

$$\gamma_0 N_d \eta e_0 \leqslant M_{ud} = \frac{2}{3} f_{cd} A r \frac{\sin^3 \pi\alpha}{\pi} + f_{sd} A_s r_s \frac{\sin\pi\alpha + \sin\pi\alpha_t}{\pi} \quad (5.3.8\text{-}2)$$

$$\alpha_t = 1.25 - \alpha \quad (5.3.8\text{-}3)$$

where:

A—cross-sectional area of the circular section;

A_s—total cross-sectional area of the longitudinal reinforcing steel;

N_{ud}, M_{ud}—design compressive resistance and flexural resistance, respectively;

r—radius of the circular section;

r_s—radius of the circle formed by the centroids of the longitudinal reinforcing steel;

e_0—eccentricity of axial force with respect to the center of gravity of the cross-section;

α—ratio of the radius angle (rad) corresponding to the cross-sectional area of concrete in the compression zone to 2π;

α_t—ratio of the cross-sectional area of longitudinal reinforcing steel in tension to the total cross-sectional area of longitudinal reinforcing steel, if α is greater than 0.625, then $\alpha_t = 0$.

Note: This clause is applicable to the case that the number of longitudinal reinforcing steel in the cross-section is not less than 8.

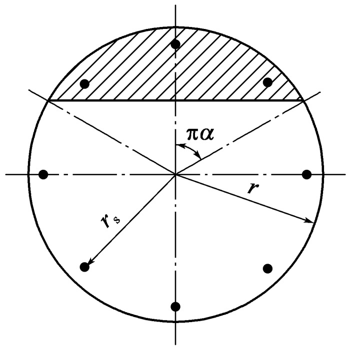


Figure 5.3.8 Circular section with longitudinal reinforcing steel evenly distributed along the perimeter

For an eccentrically loaded compression member with a circular section whose longitudinal reinforcing steel is evenly distributed along its perimeter, if its concrete strength class is between C30 and C50 and the longitudinal reinforcement ratio is between 0.5% and 4%, the compressive resistance of the member may be determined by Appendix F of the *Specifications*.

5.3.9 For an eccentrically loaded compression member with a slenderness ratio of $l_0/i > 17.5$, the amplification factor for eccentricity of axial force at the ultimate limit state, η, shall be considered. For an eccentrically loaded compression member with a rectangular section, T-section, I-section, or circular section, η may be calculated according to the following equations:

$$\eta = 1 + \frac{1}{1300 e_0/h_0}\left(\frac{l_0}{h}\right)^2 \xi_1 \xi_2 \tag{5.3.9-1}$$

$$\zeta_1 = 0.2 + 2.7\frac{e_0}{h_0} \leqslant 1.0 \tag{5.3.9-2}$$

$$\zeta_2 = 1.15 - 0.01\frac{l_0}{h} \leqslant 1.0 \tag{5.3.9-3}$$

where:

l_0—effective length of the member, which shall be taken from Appendix E of the *Specifications*;

e_0—eccentricity of the axial force with respect to the gravity axis of the section, which shall not be less than 20 mm or 1/30 of the maximum cross-sectional dimensions in the direction of eccentricity, whichever is greater;

h_0—effective depth of the cross-section, $h_0 = r + r_s$ for circular sections;

h—depth of the cross-section, $h = 2r$ for circular sections;

ζ_1—coefficient of influence of load eccentricity ratio on the curvature of the section;

ζ_2—coefficient for the influence of slenderness ratio of the member on the curvature of the section.

5.3.10 For an eccentrically loaded compression member with a rectangular, T-section, or I-section, in addition to the compressive resistance in the plane of the bending moment, the compressive resistance in the direction perpendicular to the plane of the bending moment shall also be checked as axially loaded compression members, in which the effect of the bending moment is ignored, but the influence of stability factor φ shall be considered.

5.3.11 For reinforced concrete compression members with biaxial eccentricities in which two axes of symmetry are perpendicular to each other (Figure 5.3.11), compressive resistance may be checked according to the following requirements:

$$\gamma_0 N_d \leqslant \frac{1}{\dfrac{1}{N_{ux}} + \dfrac{1}{N_{uy}} - \dfrac{1}{N_{u0}}} \tag{5.3.11}$$

where:

N_{u0}—design axial compressive resistance. It is calculated by Eq. (5.3.1) with equal sign, in which $\gamma_0 N_d$ is substituted by N_{u0}. In the calculation, all longitudinal reinforcement is counted, but the stability factor φ is not considered [Translator's note: $N_{u0} = 0.9(f_{cd}A + f'_{sd}A'_s)$];

N_{ux}—design eccentric compressive resistance of the member considering the axial force is acted along the x-axis. In the calculation, all longitudinal reinforcement is counted, and the corresponding eccentricity is considered. Herein η_x is calculated in accordance with Clause 5.3.9 of the *Specifications*. If longitudinal reinforcement is arranged on both the top and bottom sides, N_{ux} may be calculated in accordance with Clause 5.3.4 and Clause 5.3.5 of the *Specifications*. If longitudinal reinforcement is evenly distributed along the web depth, N_{ux} may be calculated in accordance with Clause 5.3.7 of the *Specifications*. In the above calculations, the equal sign is adopted for all formulas and $\gamma_0 N_d$ is replaced by N_{ux};

N_{uy}—design eccentric compressive resistance of the member considering the axial force is acted along the y-axis. In the calculation, all longitudinal reinforcement is counted, and the corresponding eccentricity is considered. Herein η_y is calculated in accordance with Clause 5.3.9 of the *Specifications*; the calculation method and equation in calculating N_{uy} are the same as those in calculating N_{ux}.

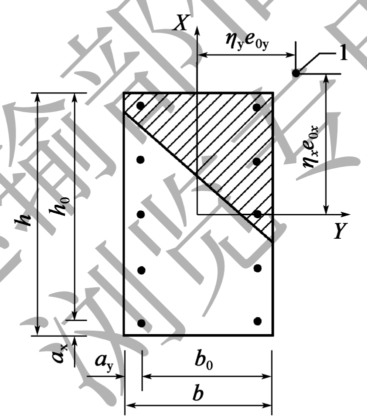


Figure 5.3.11 Reinforced concrete compression member with biaxial eccentricities

1-Point of application of axial force

5.4 Tension Members

5.4.1 The tensile resistance of axially loaded tension members shall comply with the following provisions:

$$\gamma_0 N_d \leqslant N_{ud} = f_{sd}A_s + f_{pd}A_p \tag{5.4.1}$$

where:

N_{ud}—design axial tensile resistance;

A_s, A_p—total cross-sectional area of reinforcing steel and prestressing steel, respectively.

5.4.2 The tensile resistance of eccentrically loaded tension member with rectangular section shall be checked according to the following provisions:

1 For tension members with small eccentricity in which axial force acts between the resultant force in tension reinforcement of (A_s and A_p) and that of the compression reinforcement of (A'_s and A'_p), its tensile resistance shall be checked according to the following provisions [Figure 5.4.2a)]:

$$\gamma_0 N_d e \leqslant f_{sd} A'_s (h_0 - a'_s) + f_{pd} A'_p (h_0 - a'_p) \tag{5.4.2-1}$$

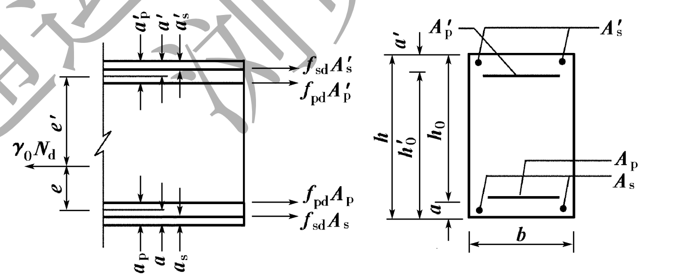
$$\gamma_0 N_d e' \leqslant f_{sd} A_s (h'_0 - a_s) + f_{pd} A_p (h'_0 - a_p) \tag{5.4.2-2}$$

2 For tension members with large eccentricity in which axial force does not act between the resultant force in the tension reinforcement of (A_s and A_p) and that in the compression reinforcement of (A'_s and A'_p), its tensile resistance shall be checked according to the following provisions [Figure 5.4.2b)]:

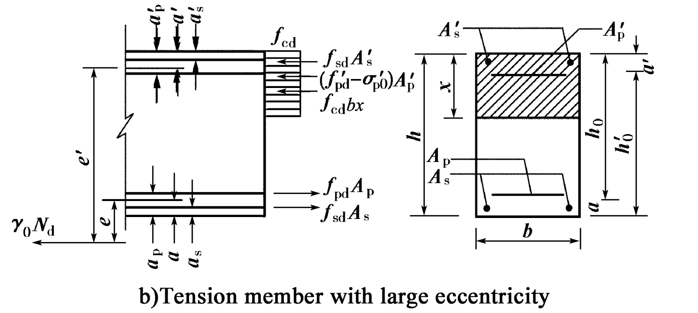
$$\gamma_0 N_d \leqslant f_{sd} A_s + f_{pd} A_p - f'_{sd} A'_s - (f'_{pd} - \sigma'_{p0}) A'_p - f_{cd} bx \tag{5.4.2-3}$$

$$\gamma_0 N_d e \leqslant f_{cd} bx \left(h_0 - \frac{x}{2}\right) + f'_{sd} A'_s (h_0 - a'_s) + (f'_{pd} - \sigma'_{p0}) A'_p (h_0 - a'_p) \tag{5.4.2-4}$$

Herein the depth of compression zone, x, shall satisfy the requirements of Eq. (5.2.2-3). If compression reinforcement is taken into account in the calculation, x shall also satisfy the requirements of Eq. (5.2.2-4) or Eq. (5.2.2-5). If x can not satisfy the requirements, x shall be calculated by Eq. (5.2.4-1) or Eq. (5.2.4-2), but M_d in those equations shall be substituted by $N_d e'$ and $N_d e'_s$, respectively.



a)Tension member with small eccentricity



b)Tension member with large eccentricity

Figure 5.4.2 Calculation on tensile resistance of eccentrically loaded tension members with rectangular section

5.4.3 For biaxial eccentrically loaded reinforced concrete tension members of rectangular sections with symmetrical reinforcement, tensile resistance shall be checked according to the following provisions:

$$\gamma_0 N_d \leqslant \frac{1}{\dfrac{1}{N_{ud}} + \sqrt{\left(\dfrac{e_{0x}}{M_{ux}}\right)^2 + \left(\dfrac{e_{0y}}{M_{uy}}\right)^2}} \tag{5.4.3}$$

where:

N_{ud}—design axial tensile resistance of the member, which is calculated by Eq. (5.4.1);

e_{0x}, e_{0y}—eccentricity of axial tension force with respect to the x-axis and y-axis passing through the center of gravity of the cross-section, respectively;

M_{ux}, M_{uy}—design flexural resistance along the direction of the x-axis and y-axis, respectively, which are calculated according to the provisions in Clause 5.2 of the *Specifications*.

5.4.4 For an eccentrically loaded tension member of a circular section with longitudinal reinforcing steel evenly distributed along the perimeter, its tensile resistance shall be checked according to the following provisions:

$$\gamma_0 N_d \leqslant \frac{1}{\dfrac{1}{N_{ud}} + \dfrac{e_0}{M_{ud}}} \tag{5.4.4}$$

where:

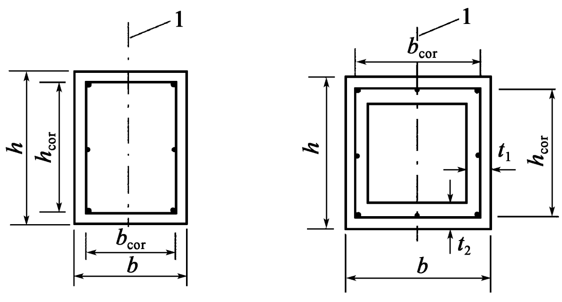
N_{ud}—design axial tensile resistance, which is calculated by Eq. (5.4.1);

e_0—eccentricity of axial tension force with respect to the gravity axis of section;

M_{ud}—design flexural resistance, which is calculated according to the provisions in Clause 5.3.8 of the *Specifications* and taking $N_{ud} = 0$.

5.5 Torsion Members

5.5.1 For pure torsion member of rectangular section, or box section with wall thickness $t_2 \geqslant 0.1b$ and $t_1 \geqslant 0.1h$ (Figure 5.5.1), torsional resistance shall be checked according to the following provisions:



a) Rectangular section ($h>b$) b) Box section ($h>b$)

Figure 5.5.1 Torsion members of rectangular and box section

1-Action plane of bending moment

$$\gamma_0 T_d \leqslant 0.35\beta_a f_{td} W_t + 1.2\sqrt{\zeta}\frac{f_{sv} A_{sv1} A_{cor}}{s_v} \tag{5.5.1-1}$$

$$\zeta = \frac{f_{sd} A_{st} S_v}{f_{sv} A_{sv1} U_{cor}} \tag{5.5.1-2}$$

For reinforced concrete members, ζ shall meet the requirement of $0.6 \leqslant \zeta \leqslant 1.7$. If $\zeta > 1.7$, $\zeta = 1.7$ is adopted.

If $e_{p0} \leqslant h/6$ and $\zeta \geqslant 1.7$, the prestress influence item of $0.05\frac{N_{p0}}{A_0}W_t$ in prestressed concrete members shall be added on the right-hand side of Eq. (5.5.1-1), and $\zeta = 1.7$ is adopted in the calculation. If $e_{p0} > h/6$ or $\zeta < 1.7$, the prestress influence item may not be taken into account, and prestressed concrete members are calculated as reinforced concrete members.

In the above formulas:

T_d—design torsional resistance;

ζ—ratio of strength of longitudinal reinforcement to that of stirrup in pure torsion member;

β_a—reduction coefficient of effective wall thickness of box section, if $0.1b \leqslant t_2 \leqslant 0.25b$ or $0.1h \leqslant t_1 \leqslant 0.25h$, the smaller one between $\beta_a = 4\frac{t_2}{b}$ and $\beta_a = 4\frac{t_1}{h}$ is adopted; if $t_2 > 0.25b$ or $t_1 > 0.25h$, let $\beta_a = 1.0$; for rectangular section, $\beta_a = 1.0$ is taken;

b—width of rectangular or box section;

h—depth of rectangular or box section;

t_1—wall thickness of the long side of the box section;

t_2—wall thickness of the short side of the box section;

f_{td}—design axial tensile strength of concrete;

W_t—plastic section modulus of rectangular or box section to resist torsion, which is calculated according to Clause 5.5.2 of the *Specifications*;

A_{sv1}—cross-sectional area of single-leg stirrup in the calculation of pure torsion;

f_{sv}—design tensile strength of stirrup, which is taken from Table 3.2.3-1;

A_{st}—total cross-sectional area of longitudinal reinforcing steel symmetrically arranged along the perimeter in the calculation of pure torsion;

f_{sd}—design axial tensile strength of longitudinal reinforcement, which is taken from Table 3.2.3-1;

A_{cor}—core area of cross-section surrounded by the internal surface of the stirrup, $A_{cor} = b_{cor}h_{cor}$, herein b_{cor} and h_{cor} are the short side length and the long side length of core section, respectively;

U_{cor}—perimeter of the cross-section of concrete core, $U_{cor} = 2(b_{cor} + h_{cor})$;

s_v—stirrup spacing in the calculation of pure torsion;

e_{p0}—eccentricity of resultant force in prestressing and reinforcing steel with respect to the gravity axis of the transformed section, both pretensioned and post-tensioned members are calculated by Eq. (6.1.7-2), but σ_{p0} and σ'_{p0} in the equation are calculated by Eq. (6.1.6-2) for pretensioned members and Eq. (6.1.6-5) for post-tensioned members, respectively.

N_{p0}—resultant force in prestressing and reinforcing steel when normal stress of concrete equals to zero, both pretensioned and post-tensioned members are calculated by Eq. (6.1.7-1), but σ_{p0} and σ'_{p0} in the equation are calculated by Eq. (6.1.6-2) for pretensioned members and Eq. (6.1.6-5) for post-tensioned members, respectively. If $N_{p0} > 0.3 f_{cd} A_0$, let $N_{p0} = 0.3 f_{cd} A_0$, herein A_0 is the area of the transformed section of the member.

5.5.2 Plastic section modulus of members with rectangular or box section to resist torsion shall be calculated according to the following equations:

1 For rectangular section [Figure 5.5.1a)],

$$W_t = \frac{b^2}{6}(3h - b) \tag{5.5.2-1}$$

2 For box section [Figure 5.5.1b)],

$$W_t = \frac{b^2}{6}(3h - b) - \frac{(b - 2t_1)^2}{6}[3(h - 2t_2) - (b - 2t_1)] \tag{5.5.2-2}$$

5.5.3 For a member with a rectangular or box section subjected to bending, shear and torsion (Figure 5.5.1), its cross-section shall comply with the requirements of the following formulas:

$$\frac{\gamma_0 V_d}{bh_0} + \frac{\gamma_0 T_d}{W_t} \leqslant f_{cv} \tag{5.5.3-1}$$

If the following condition is satisfied,

$$\frac{\gamma_0 V_d}{bh_0} + \frac{\gamma_0 T_d}{W_t} \leqslant 0.5 \alpha_2 f_{td} \tag{5.5.3-2}$$

torsional resistance of the member may not be checked, and the member shall only be reinforced in accordance with the detailing requirements specified in the Clause 9.3.13 of the *Specifications*.

In the above formulas:

V_d—design shear force (N);

T_d—design torsional moment (N·mm);

f_{cv}—design nominal shear stress (MPa), $f_{cv} = 0.51\sqrt{f_{cu,k}}$;

b—width of rectangular section or total web widths of box section, perpendicular to the plane of bending moment (mm);

h_0—effective depth of rectangular or box section, parallel to the plane of the bending moment (mm);

W_t—plastic section modulus of cross-section to resist torsion (mm^3).

In Eq. (5.5.3-2), α_2 refers to Clause 5.2.9 of the *Specifications*. If the influence of the prestress can be ignored in accordance with the provisions in Clause 5.5.1 of the *Specifications*, then $\alpha_2 = 1$.

5.5.4 For shear-torsion members with rectangular or box section, the shear-torsion resistance shall be checked in accordance with the following formulas:

1 Shear resistance

$$\gamma_0 V_d \leqslant 0.5\times 10^{-4}\alpha_1\alpha_2\alpha_3(10-2\beta_t)bh_0\sqrt{(2+0.6P)\sqrt{f_{cu,k}}\rho_{sv}f_{sv}} \tag{5.5.4-1}$$

2 Torsional resistance

$$\gamma_0 T_d \leqslant \beta_t\left(0.35\beta_a f_{td} + 0.05\frac{N_{p0}}{A_0}\right)W_t + 1.2\sqrt{\xi}\frac{f_{sv}A_{sv1}A_{cor}}{S_v} \tag{5.5.4-2}$$

$$\beta_t = \frac{1.5}{1 + 0.5\dfrac{V_d W_t}{T_d b h_0}} \tag{5.5.4-3}$$

where:

β_t—reduction coefficient for torsional resistance of concrete in shear-torsion member. If $\beta_t < 0.5$, $\beta_t = 0.5$; if $\beta_t > 1.0$, $\beta_t = 1.0$;

W_t—plastic section modulus of cross-section to resist torsion. For shear-torsion members with box section, W_t shall be substituted by $\beta_a W_t$;

b—width of rectangular section or total web width of box section.

Refer to Clause 5.2.9 and Clause 5.5.1 of the *Specifications* for the definitions and units of other symbols.

When the influence of the prestress can be ignored in accordance with the provisions in Clause 5.5.1 of the *Specifications*, $\alpha_2 = 1$ is adopted in Eq. (5.5.4-1), and the second item inside the parenthesis at the right-hand side of Eq. (5.5.4-2) is taken as zero.

5.5.5 For torsion members with T-section, I-section, or box section having flanges, their torsional resistance may be calculated by dividing the cross-sections into several rectangular sections:

1 The design torsional moment of webs or rectangular boxes without flange, compression flanges, and tension flanges, shall be calculated by the following equations:

$$T_{wd}=\frac{W_{tw}}{W_t}T_d \tag{5.5.5-1}$$

$$T'_{fd}=\frac{W'_{tf}}{W_t}T_d \tag{5.5.5-2}$$

$$T_{fd}=\frac{W_{tf}}{W_t}T_d \tag{5.5.5-3}$$

where:

T_d—design torsional moment resisted by T-section, I-section or box section with flange;

T_{wd}—design torsional moment distributed to webs or rectangular boxes without flange;

T'_{fd}, T_{fd}—design torsional moment distributed to compression flange and tension flange, respectively;

W_{tw}, W'_{tf}, W_{tf}—plastic section modulus of webs or rectangular box without flange, compression flange, and tension flange (all are subjected to torsion moments), respectively;

W_t—total plastic section modulus of T-section, I-sections, or box sections with flanges (all are subjected to torsion moments).

2 Plastic section modulus of different types of cross-section subjected to torsion:

1) Plastic section modulus of webs, rectangular boxes without flange (both are subjected to torsion moments) shall be calculated in accordance with Clause 5.5.2 of the *Specifications*;

2) Plastic section modulus of compression flange to resist torsion shall be calculated by Eq. (5.5.5-4);

$$W'_{tf}=\frac{h'^2_f}{2}(b'_f-b) \tag{5.5.5-4}$$

3) Plastic section modulus of tension flange to resist torsion shall be calculated by Eq. (5.5.5-5);

$$W_{tf}=\frac{h^2_f}{2}(b_f-b) \tag{5.5.5-5}$$

where:

b'_f, h'_f—width and depth of T-section, I-section, or compression flanges of box sections with flanges (Figure 5.5.5), respectively, which shall satisfy the requirement of $b'_f \leqslant b+6h'_f$;

b_f, h_f—width and depth of I-section tension flange, respectively, which shall satisfy the requirement of $b_f \leqslant b+6h_f$;

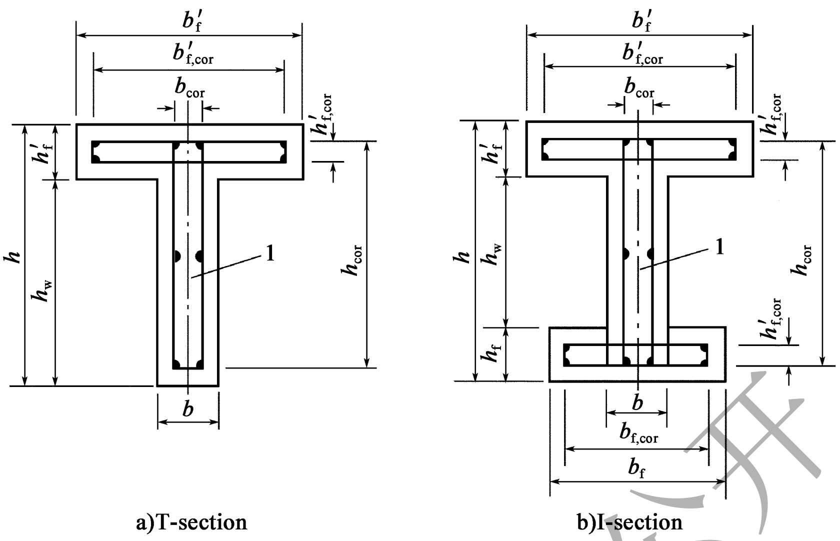


Figure 5.5.5 T-section and I-section Torsion members

1-Action plane of bending moment

3 Total plastic section modulus of different types of cross-section to resist torsion:

1) T-sections and box sections with flanges

$$W_t = W_{tw} + W'_{tf} \tag{5.5.5-6}$$

2) I-sections

$$W_t = W_{tw} + W'_{tf} + W_{tf} \tag{5.5.5-7}$$

4 When webs of a T-section, I-section, or the rectangular box in a box section with flanges are used as shear-torsion members, the shear-torsion resistance shall be calculated in accordance with the provisions in Clause 5.5.4 of the *Specifications*, where T_d and W_t in the equation is substituted by T_{wd} and W_{tw}, respectively. The torsional resistance of the compression flanges or tension flanges which are regarded as pure torsion members shall be calculated in accordance with the provisions in Clause 5.5.1 of the *Specifications*. In the calculation, T_d and W_t in Eq. (5.5.1-1) shall be substituted by T'_{fd} and W'_{tf}, or T_{fd} and W_{tf}.

5 For members with a T-section, I-section, or box section with flanges subjected to combined flexure, shear and torsion, their sections shall meet the requirement of the provisions in Clause 5.5.3 of the *Specifications*.

Note: For a T-section or I-section torsion member, its webs shall meet the requirement of $b/h_w \geqslant 0.15$. Herein, b and h_w are the width and clear depth of the webs, respectively (Figure 5.5.5).

5.5.6 For members with a rectangular section, T-section, I-section, or box section with flanges subjected to combined flexure, shear and torsion, their longitudinal reinforcements and stirrups shall be calculated and designed in accordance with the following provisions, respectively:

1 The longitudinal reinforcement shall be arranged according to the cross-sectional area required by the flexural resistance, which is calculated by taking the member as a flexural member.

2 The longitudinal reinforcement and stirrup for a rectangular section, webs of T-section and I-section, as well as a rectangular box without flange shall be calculated as those in shear-torsion members:

1) The required cross-sectional area of longitudinal reinforcement for torsional resistance shall be calculated according to Clause 5.5.4 of the *Specifications*, and it shall be arranged evenly and symmetrically along the perimeter;

2) Cross-sectional area of the stirrup for shear resistance and torsional resistance shall be calculated according to Clause 5.5.4 of the *Specifications*.

3 The required cross-sectional areas of longitudinal reinforcement and stirrup for torsional resistance in compression flanges or tension flanges of T-sections, I-sections or rectangular boxes with flanges shall be calculated according to Clause 5.5.1 of the *Specifications*, in which the longitudinal reinforcement shall be evenly and symmetrically arranged along the perimeter.

5.6 Members Subjected to Punching Shear

5.6.1 For reinforced concrete slabs without punching shear reinforcement under concentrated reaction force, punching shear resistance may be checked in accordance with the following formula (Figure 5.6.1):

$$\gamma_0 F_{ld} \leqslant (0.7\beta_h f_{td} + 0.15\sigma_{pc,m}) U_m h_0 \tag{5.6.1}$$

where:

F_{ld}—design value of the maximum concentrated reaction force. In calculating the punching shear resistance of the slab supported by the pier column, it may be taken from the design value of the maximum axial force resisted by the pier column minus the design load in the range of the truncated pyramid for punching shear failure on top of the column;

$\sigma_{pc,m}$—effective average compressive stress caused by prestressing force on the section of the slab with prestressing steel, its value should be controlled within 1.0 MPa ~ 3.5 MPa;

β_h—effect coefficient of sectional depth. If $h \leqslant 300$ mm, $\beta_h = 1.0$. If $h \geqslant 800$ mm, $\beta_h = 0.85$. If β_h is between them, it is calculated by linear interpolation. Herein, h is the slab thickness;

U_m—perimeter of the cross-section of the truncated pyramid that is $h_0/2$ distant from the border of the concentrated reaction force. For circular pier columns, it may be converted to a square column whose side length is 0.8 times the diameter to calculate U_m;

h_0—effective depth of the slab.

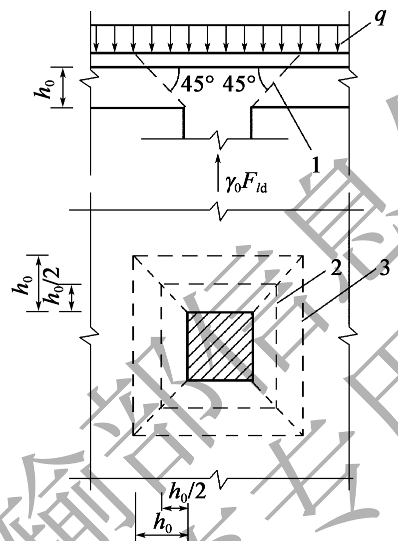


Figure 5.6.1 Calculation on punching shear resistance of the slab

1-Inclined section of the truncated pyramid for punching shear failure; 2- Perimeter of the cross-section of the truncated pyramid for punching shear failure that is $h_0/2$ distant from the border of the concentrated reaction force; 3-Line on the bottom surface of the truncated pyramid for punching shear failure

5.6.2 Under the action of concentrated reaction force, if the punching shear resistance cannot satisfy the requirement of Eq. (5.6.1) while the slab thickness is restricted, punching shear reinforcement may be arranged. In this case, the punching section shall meet the following requirements:

$$\gamma_0 F_{ld} \leqslant 1.05 \beta_h f_{td} U_m h_0 \qquad (5.6.2\text{-}1)$$

Punching shear resistance of concrete slab arranged with punching shear reinforcement may be calculated in accordance with the following provisions:

1 If stirrups are arranged,

$$\gamma_0 F_{ld} \leqslant (0.35 \beta_h f_{td} + 0.15 \sigma_{pc,m}) U_m h_0 + 0.75 f_{sv} A_{svu} \qquad (5.6.2\text{-}2)$$

2 If bent bars are arranged,

$$\gamma_0 F_{ld} \leqslant (0.35\beta_h f_{td} + 0.15\sigma_{pc,m}) U_m h_0 + 0.75 f_{sv} A_{sbu} \sin\theta \tag{5.6.2-3}$$

where:

A_{svu}—total cross-sectional area of stirrups intersecting with the inclined section of the truncated pyramid for punching shear failure;

A_{sbu}—total cross-sectional area of bent bars intersecting with the inclined section of truncated pyramid for punching shear failure;

f_{sv}—design tensile strength of stirrups;

f_{sb}—designe tensile strength of bent bars;

θ—angle between the bent bar and the bottom surface of the slab.

For sections outside the truncated pyramid for punching shear failure in the slab arranged with punching shear reinforcement, punching shear resistance shall also be calculated according to Clause 5.6.1 of the *Specifications*. In this case, the most unfavorable perimeter at $0.5h_0$ from the truncated pyramid for punching shear failure shall be adopted for U_m.

Note: Shear stirrups or bent bars arranged in concrete slab against punching shear failure shall meet the detailing requirements specified in Clause 9.2.9 of the *Specifications*.

5.6.3 For a spread footing of a rectangular pier column, the punching shear resistance of the column-footing interface and the step interface of a stepped footing may be checked in accordance with the following provisions (Figure 5.6.3):

$$\gamma_0 F_{ld} \leqslant 0.7\beta_h f_{td} b_m h_0 \tag{5.6.3-1}$$

$$F_{ld} = p_s A \tag{5.6.3-2}$$

$$b_m = \frac{b_t + b_b}{2} \tag{5.6.3-3}$$

where:

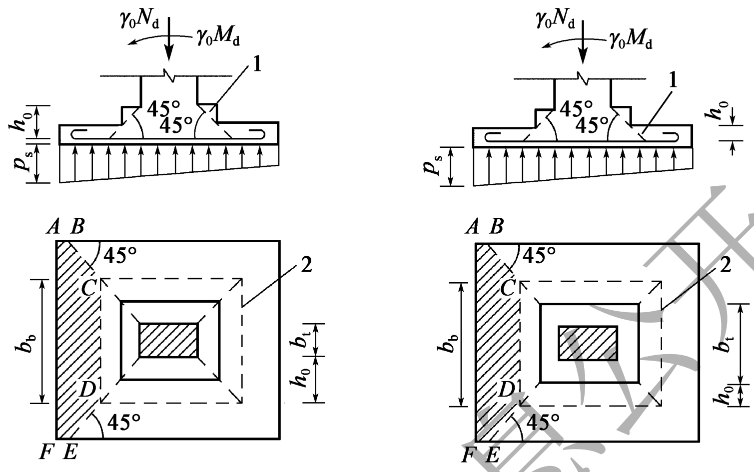
b_t—the upper side length of the inclined section in the most unfavorable side at the truncated pyramid for punching shear failure. In calculating the punching shear resistance of the column-footing interface, the column width is adopted; in calculating the punching shear resistance of the step-to-step interface of a stepped footing, the width of the upper step is adopted;

b_b—the bottom side length of the inclined section in the most unfavorable side at the truncated pyramid for punching shear failure. In calculating the punching shear resistance of the column-footing interface, the column width plus 2 times of the effective depth of the footing is adopted; in calculating the punching shear resistance of the step interface of a stepped footing, the width of the upper step plus 2 times of the effective depth of footing below the interface is adopted;

h_0—effective depth of the footing inner the truncated pyramid for punching shear failure;

p_s—reaction force within unit area of footing under the design load (the dead load of the footing and the soil weight on the footing may be deducted). The maximum unit reaction force may be taken if the footing is eccentrically loaded;

A—base polygon area in the calculation of punching shear force (the shadow area of $ABCDEF$ in Figure 5.6.3).



a)Column-footing interface　　b)Step interface

Figure 5.6.3　Punching shear resistance of the rectangular spread footing

1-Inclined section in the most unfavorable side at the truncated pyramid for punching shear failure; 2-Line of the bottom surface of the truncated pyramid for punching shear failure

5.7　Members Subjected to Local Compression

5.7.1　Cross-sectional dimension of local compression zone in a reinforced concrete member with confinement reinforcement shall meet the following requirements:

$$\gamma_0 F_{ld} \leqslant 1.3\eta_s \beta f_{cd} A_{ln} \tag{5.7.1-1}$$

$$\beta = \sqrt{\frac{A_b}{A_l}} \tag{5.7.1-2}$$

where:

F_{ld}—design local compression force on local compression area. For the local compression area ahead of the anchorage device in the post-tensioned member, 1.2 times the maximum compression force in jacking shall be adopted;

f_{cd}—design compressive strength of concrete. For a post-tensioned prestressed concrete member, it shall be obtained from Table 3.1.4 directly or by linear interpolation according to the cube compressive strength when the member is jacking;

η_s—adjustment coefficient for compressive strength of concrete under local compression. If the concrete strength class is C50 or below, $\eta_s = 1.0$; if it is C80, $\eta_s = 0.76$; if it is between C50 and C80, η_s is taken by linear interpolation;

β—increase coefficient of compressive strength of concrete under local compression;

A_b—effective base area for local compression, which may be determined by the principle that the effective base area and the local compression area are concentric and symmetrical; for common situations, it may be determined according to Figure 5.7.1;

A_{ln}, A_l—local compression area of concrete. If the local compression surface with opening, A_{ln} is the area without the opening, while A_l is the total area including the opening. When the steel base plate is arranged under the bearing face, the local compression area shall be measured in the spread area by 45° from the base plate. For the anchor with trumpet pipe and integrated together with the base plate, A_{ln} may be obtained by subtracting the inner opening area at the end of the trumpet pipe from the base plate area.

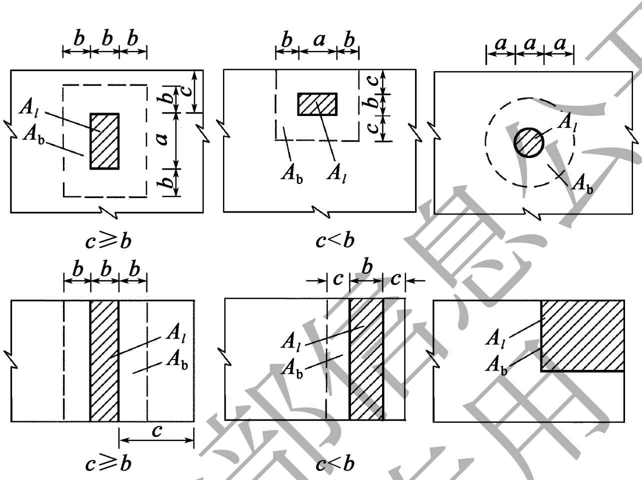


Figure 5.7.1 Base area A_b under local compression

5.7.2 For a reinforced concrete member with confinement reinforcement for local compression, its local compression resistance shall be checked in accordance with the following provisions:

$$\gamma_0 F_{ld} \leqslant 0.9(\eta_s \beta f_{cd} + k\rho_v \beta_{cor} f_{sd}) A_{ln} \tag{5.7.2-1}$$

$$\beta_{cor} = \sqrt{\frac{A_{cor}}{A_l}} \tag{5.7.2-2}$$

The volumetric ratio of confinement reinforcement (the confinement reinforcement volume per unit concrete volume within the core area A_{cor}) shall be determined by the following equations:

For square wire fabric,

$$\rho_v = \frac{n_1 A_{s1} l_1 + n_2 A_{s2} l_2}{A_{cor} s} \tag{5.7.2-3}$$

Here, the difference in the cross-sectional area of reinforcement in the two directions of the wire fabric shall not exceed 50%.

For spiral reinforcement,

$$\rho_v = \frac{4A_{ss1}}{d_{cor} s} \tag{5.7.2-4}$$

where:

β_{cor}—increase coefficient of local compression resistance for confinement reinforcement; if

$A_{cor} > A_b$, $A_{cor} = A_b$ shall be adopted;

k—influence coefficient of confinement reinforcement, which is taken from Clause 5.3.2 of the *Specifications*;

A_{cor}—area of concrete core in the range of the inner surface of square wire fabric or spiral confinement reinforcement. Its centroid shall coincide with that of A_l, and it shall be calculated according to the concentric and symmetrical principle;

n_1, A_{s1}—number of steel bars and cross-sectional area of a single steel bar along l_1 direction of square wire fabric, respectively;

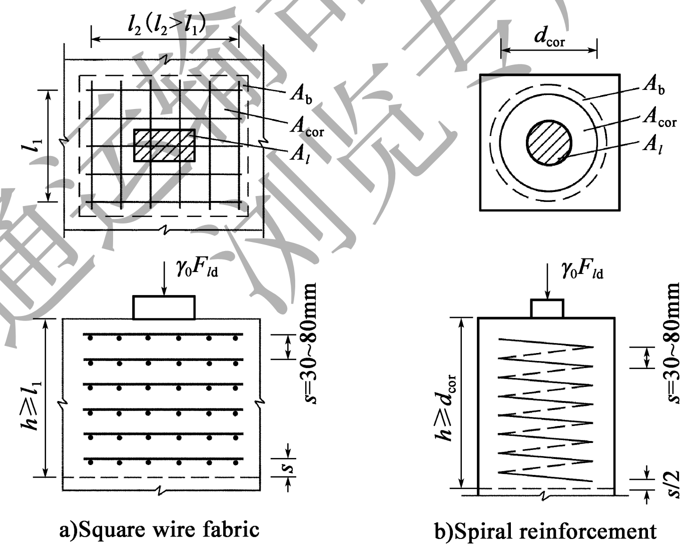
n_2, A_{s2}—number of steel bars and cross-sectional area of a single steel bar along l_2 direction of square wire fabric, respectively;

A_{ss1}—cross-sectional area of a single spiral confinement reinforcement;

d_{cor}—diameter of the area of concrete core in the range of the inner surface of the spiral confinement reinforcement;

s—layer spacing of square wire fabric or spiral confinement reinforcements.

Note: The square wire fabric shall not be less than 4 layers, and the spiral reinforcement shall not be less than 4 coils; for the anchor plate with trumpet pipe, the length of all coils of spiral reinforcement below the plate shall not be less than the length of the trumpet pipe.



a)Square wire fabric b)Spiral reinforcement

Figure 5.7.2 Reinforcement arrangement for local compression area

6 Design for Serviceability Limit State in Persistent Situations

6.1 General

6.1.1 In persistent situation design for highway bridges and culverts, cracking resistance, crack width and deflection of members under frequent combination of actions, quasi-permanent combination of actions, or frequent combination of actions considering the long-term effect shall be checked according to the requirements for serviceability limit state. All calculated values shall not exceed the corresponding limits stipulated in the *Specifications*. In the above various combinations, the impact effect of vehicular load may not be considered.

6.1.2 For prestressed concrete structures, the following members may be designed according to the requirements for bridge service and local environmental conditions:

1 Fully prestressed concrete members: the tensile stress is not permitted to appear on the extreme tension fiber of the critical cross-section under the frequent combination of actions.

2 Partially prestressed concrete members: the tensile stress is permitted to appear on the extreme tension fiber of the critical cross-section under the frequent combination of actions. They are referred to as Type A prestressed concrete members when tensile stress does not exceed the limit value, and Type B prestressed concrete members when tensile stress exceeds the limit value.

6.1.3 Cracking resistance and crack width of concrete bridges with box section should be checked according to Table 6.1.3.

Table 6.1.3 Requirements in checking for cracking resistance and crack width of structures with box section

Component	Requirements		
	Fully or Type A prestressed members		Type B prestressed members and reinforced concrete members
Top flange	Normal stress of the upper edge in the longitudinal direction	Conform to the provisions in Section 6.3 of the *Specifications*	Check for crack width in accordance with the provisions in Section 6.4 of the *Specifications*
	Normal stress of the upper edge and lower edge in the transverse direction		
	In-plane principal stress		
Bottom flange	Normal stress of the lower edge in the longitudinal direction		
	Normal stress of the upper edge and lower edge in the transverse direction		
	In-plane principal stress		
Web	In-plane principal stress		

6.1.4 Maximum jacking stress of prestressing steel in a prestressed concrete member, σ_{con}, shall comply with the following provisions:

1 Maximum jacking stress for steel wires and strands

For internal tendons: $$\sigma_{con} \leq 0.75 f_{pk} \tag{6.1.4-1}$$

For external tendons: $$\sigma_{con} \leq 0.70 f_{pk} \tag{6.1.4-2}$$

2 Maximum jacking stress for prestressing threaded bar

$$\sigma_{con} \leq 0.85 f_{pk} \tag{6.1.4-3}$$

where:

f_{pk}—characteristic tensile strength of prestressing steel, which is taken from Table 3.2.2-2.

If overstress jacking is used for members or the loss of prestress due to the anchor set is considered in the loss of prestress, the tensile stress limit in prestressing steel (the value shown on the oil pump of the jack) may be added with a value of $0.05 f_{pk}$.

6.1.5 In calculating the stress of a prestressed concrete member in the elastic stage, the cross-sectional properties of the member may be adopted in accordance with the following provisions:

1 For pretensioned members, the transformed section is used.

2 For post-tensioned members, in calculating the stress caused by the action and external

tendon, the net cross-section is used before mortar grouting of the tendon duct, and the transformed section is used after the internal tendon bonded with the concrete; in calculating the stress caused by the internal tendon, the net cross-section is used, except as specified otherwise.

3 If the cross-sectional properties have only a small effect on the calculated stresses or the control conditions, the gross cross-section may also be used.

6.1.6 Normal stress of concrete caused by the prestressing force as well as the stress of prestressing steel at the corresponding stage shall be calculated according to the following equations:

1 Prestensioned concrete members:

Normal compressive stress σ_{pc} and tensile stress σ_{pt} of concrete caused by prestressing force:

$$\begin{matrix}\sigma_{pc}\\ \sigma_{pt}\end{matrix} = \frac{N_{p0}}{A_0} \pm \frac{N_{p0} e_{p0}}{I_0} y_0 \tag{6.1.6-1}$$

Stress of prestressing steel when the normal stress of concrete at point of the resultant force in prestressing steel is equal to zero:

$$\left.\begin{aligned}\sigma_{p0} &= \sigma_{con} - \sigma_l + \sigma_{l4}\\ \sigma'_{p0} &= \sigma'_{con} - \sigma'_l + \sigma'_{l4}\end{aligned}\right\} \tag{6.1.6-2}$$

Effective prestress of prestressing steels at the corresponding stage

$$\left.\begin{aligned}\sigma_{pe} &= \sigma_{con} - \sigma_l\\ \sigma'_{pe} &= \sigma'_{con} - \sigma'_l\end{aligned}\right\} \tag{6.1.6-3}$$

2 Post-tensioned members with internal tendons:

Normal compressive stress σ_{pc} and tensile stress σ_{pt} of concrete caused by prestressing force:

$$\begin{matrix}\sigma_{pc}\\ \sigma_{pt}\end{matrix} = \frac{N_p}{A_n} \pm \frac{N_p e_{pm}}{I_n} y_n \pm \frac{M_{p2}}{I_n} y_n \tag{6.1.6-4}$$

Stress of prestressing steel when the normal stress of concrete at the point of the resultant force in prestressing steel is equal to zero:

$$\left.\begin{aligned}\sigma_{p0} &= \sigma_{con} - \sigma_l + \alpha_{EP}\sigma_{pc}\\ \sigma'_{p0} &= \sigma'_{con} - \sigma'_l + \alpha_{EP}\sigma'_{pc}\end{aligned}\right\} \tag{6.1.6-5}$$

Effective prestress of prestressing steels at the corresponding stage

$$\left.\begin{aligned}\sigma_{pe} &= \sigma_{con} - \sigma_l\\ \sigma'_{pe} &= \sigma'_{con} - \sigma'_l\end{aligned}\right\} \tag{6.1.6-6}$$

3 Post-tensioned members with both internal and external tendons:

Normal compressive stress σ_{pc} and tensile stress σ_{pt} of concrete caused by prestressing force:

$$\left.\begin{array}{l}\sigma_{pc}\\ \sigma_{pt}\end{array}\right\} = \frac{N_{p,ex}}{A_{ex}} \pm \frac{N_{p,ex}e_{p,ex}}{I_{ex}}y_{ex} \pm \frac{M_{p2,ex}}{I_{ex}}y_{ex} \qquad (6.1.6\text{-}7)$$

Stress of the internal tendon at the corresponding stage is calculated according to Eq. (6.1.6-5) and Eq. (6.1.6-6).

Effective prestress of the external tendon at the corresponding stage

$$\left.\begin{array}{l}\sigma_{pe,ex} = \sigma_{con} - \sigma_l\\ \sigma'_{pe,ex} = \sigma'_{con} - \sigma'_l\end{array}\right\} \qquad (6.1.6\text{-}8)$$

where:

A_n—area of the net cross-section, *i. e.*, the sum of the total cross-sectional area of concrete after deducting the weakened parts, e. g. ducts, and the transformed cross-sectional area of longitudinal reinforcing steel. For the cross-section composed of concrete of different strength classes, the cross-sectional area shall be calculated by converting all the concrete to the same strength class according to the ratio of elastic modulus of concrete;

A_0—area of the transformed section, including the net area of section, A_n, and the transformed sectional area of concrete from the total cross-sectional area of the longitudinal internal tendons;

A_{ex}—cross-sectional area of a prestressed concrete member by post-tensioning with both internal and external tendons, which is taken in accordance with the provisions in Clause 6.1.5 of the *Specifications*, and the grouting influence in the duct is taken into account;

N_{p0}, N_p—resultant force in prestressing and reinforcing steel for pretensioned and post-tensioned members, respectively, which is calculated by Eq. (6.1.7-1) and Eq. (6.1.7-3) of the *Specifications*;

$N_{p,ex}$—resultant force in prestressing and reinforcing steel for post-tensioned member with both internal and external tendons, which is calculated by Eq. (6.1.7-5) of the *Specifications*;

I_0, I_n—moment of inertia of transformed section and net cross-section, respectively;

I_{ex}—the section moment of inertia of post-tensioned prestressed concrete members with both internal and external tendons, which is taken in accordance with the provisions in Clause 6.1.5 of the *Specifications*. In its calculation, the influence of grouting in the duct is taken into account;

e_{p0}, e_{pn}—distance from the center of gravity of the transformed section or net cross-section to the resultant force in prestressing steel and reinforcing steel, respectively, which is calculated by Eq. (6.1.7-2) and Eq. (6.1.7-4) of the *Specifications*;

$e_{p,ex}$—distance from the center of gravity of cross-section of post-tensioned member with both internal and external tendons to the resultant force in internal tendons, external tendons and reinforcing steel, which is calculated by Eq. (6.1.7-6) of the *Specifications*;

y_0, y_n—distance from the center of gravity of the transformed section and net section to the location where the fiber stress is investigated, respectively;

y_{ex}—distance from the gravity center of the section to the location where the fiber stress is investigated in a post-tensioned member with both internal and external tendons;

σ_{con}, σ'_{con}—maximum jacking stress for prestressing steel in the tension and compression zones, respectively, which is determined according to Clause 6.1.4 of the *Specifications*;

σ_l, σ'_l—loss of prestress at the corresponding construction stages in the tension and compression zones, respectively, which is calculated in accordance with the provisions from Clause 6.2.2 to Clause 6.2.7 of the *Specifications*; at the service stage, it is the total loss of prestress;

σ_{l4}, σ'_{l4}—loss of prestress due to elastic shortening of concrete in the tension and compression zones, respectively, which is calculated by Eq. (6.2.5-2) of the *Specifications*;

α_{EP}—ratio of elastic modulus of prestressing steel, E_p, to that of concrete, E_c, in which E_p and E_c are taken from Table 3.2.4 and Table 3.1.5 of the *Specifications*, respectively;

M_{p2}, $M_{p2,ex}$—secondary bending moment caused by prestressing force N_p or $N_{p,ex}$ in prestressed concrete continuous beams and other statically indeterminate structures.

Note: 1 In Eq. (6.1.6-1), Eq. (6.1.6-4), and Eq. (6.1.6-7), the second and the third items on the right-hand side of the equations are positive if their stress direction is the same with that of the first item, and are negative if the direction is contrary. Positive sign means compression and negative sign means tension.

2 In Eq. (6.1.6-5), σ_{pc} and σ'_{pc} are concrete normal stress caused by N_p at the centroids of prestressing steel in the compression and tension zones, respectively. Positive and negative values are adopted when they are tensile stress and compressive stress, respectively.

3 In calculating σ_{p0} and σ'_{p0} for post-tensioned members with both internal and external tendons, σ_{pc} and σ'_{pc} are normal stress of concrete caused by $N_{p,ex}$ at the centroids of prestressing steels in the compression and tension zones, respectively, which is calculated by Eq. (6.1.6-7), and positive and negative values are adopted when they are tensile stress and compressive stress.

6.1.7 Resultant forces in prestressing steel and reinforcing steel as well as their eccentricities shall be calculated by the following equations:

1 Pretensioned members [Figure 6.1.7a)]:

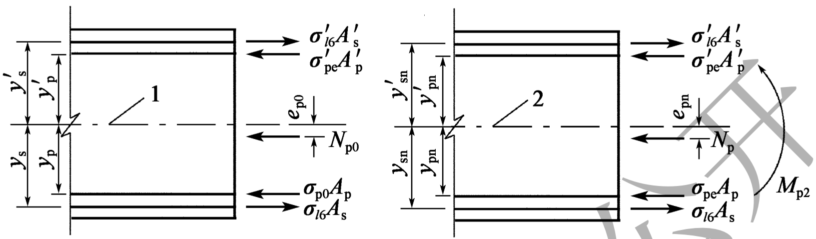
$$N_{p0} = \sigma_{p0} A_p + \sigma'_{p0} A'_p - \sigma_{l6} A_s - \sigma'_{l6} A'_s \qquad (6.1.7\text{-}1)$$

$$e_{p0}=\frac{\sigma_{p0}A_p y_p-\sigma'_{p0}A'_p y'_p-\sigma_{l6}A_s y_s+\sigma'_{l6}A'_s y'_s}{N_{p0}} \quad (6.1.7\text{-}2)$$

2 Post-tensioned members [Figure 6.1.7b)]:

$$N_p=\sigma_{pe}A_p+\sigma'_{pe}A'_p-\sigma_{l6}A_s-\sigma'_{l6}A'_s \quad (6.1.7\text{-}3)$$

$$e_{pn}=\frac{\sigma_{pe}A_p y_{pn}-\sigma'_{pe}A'_p y'_{pn}-\sigma_{l6}A_s y_{sn}+\sigma'_{l6}A'_s y'_{sn}}{N_p} \quad (6.1.7\text{-}4)$$



a)Pretensioned member   b)Post-tensioned member

Figure 6.1.7 Resultant force and its eccentricity of prestressing and reinforcing steel

1-Gravity axis of transformed section; 2-Gravity axis of net cross-section

3 Post-tensioned members with both internal and external tendons:

$$N_{p,ex}=\sigma_{pe}A_p+\sigma'_{pe}A'_p+\sigma_{pe,ex}A_{p,ex}+\sigma'_{pe,ex}A'_{p,ex}-\sigma_{l6}A_s-\sigma'_{l6}A'_s \quad (6.1.7\text{-}5)$$

$$e_{p,ex}=\frac{\sigma_{pe}A_p y_p-\sigma'_{pe}A'_p y'_p+\sigma_{pe,ex}A_{p,ex}y_{p,ex}-\sigma'_{pe,ex}A'_{p,ex}y'_{p,ex}-\sigma_{l6}A_s y_s+\sigma'_{l6}A'_s y'_s}{N_{p,ex}} \quad (6.1.7\text{-}6)$$

where:

σ_{p0}, σ'_{p0}—stress of prestressing steel when the normal stress of concrete at point of the resultant force of internal tendons in compression or tension zone is equal to zero, respectively, which are calculated by the equations in Clause 6.1.6 of the *Specifications*;

σ_{pe}, σ'_{pe}—effective prestress of the internal tendon in the compression and tension zones, respectively, which are calculated by the equations in Clause 6.1.6 of the *Specifications*;

$\sigma_{pe,ex}$, $\sigma'_{pe,ex}$— effective stress of external tendon in the compression and tension zones, respectively, which are calculated by the equations in Clause 6.1.6 of the *Specifications*;

A_p, A'_p—cross-sectional area of the internal tendon in the compression and tension zones, respectively;

$A_{p,ex}$, $A'_{p,ex}$—cross-sectional area of the external tendon in the compression and tension zones, respectively;

A_s, A'_s—cross-sectional area of reinforcing steel in the compression and tension zones, respectively;

y_p, y'_p—distance from the resultant force of the internal tendon in the compression and tension zones to the gravity axis of the transformed section, respectively;

y_s, y'_s—distance from the centroid of reinforcing steel in the compression and tension zones to the gravity axis of the transformed section, respectively;

y_{pn}, y'_{pn}—distance from the resultant force of internal tendon in the compression and tension zones to the gravity axis of net cross-section, respectively;

y_{sn}, y'_{sn}—distance from the centroid of reinforcing steel in the compression and tension zones to the gravity axis of net cross-section, respectively;

$y_{p,ex}$, $y'_{p,ex}$—distance from the centroid of external tendon in compression and tension zones to the gravity axis of cross-section of post-tensioned members with both internal and external tendons, respectively;

σ_{l6}, σ'_{l6}—loss of prestress due to shrinkage and creep of concrete at the point of the resultant force of prestressing steel in compression and tension zones, respectively, which are calculated in accordance with the provisions in Clause 6.2.7 of the *Specifications*.

Note: 1. When A'_p in Eq. (6.1.7-1) to Eq. (6.1.7-4) is equal to zero, σ'_{l6} in the equation shall be adopted as 0.

2. When $A'_p = A'_{p,ex}$ in Eq. (6.1.7-5) and Eq. (6.1.7-6) are equal to zero, σ'_{l6} in the equations shall be adopted as 0.

3. Eq. (6.1.7-6) is obtained according to the transformed section with ducts after grouting. If the ducts for internal tendons are not grouted, the net cross-section is used in the calculation, and y_p, y'_p, y_s, y'_s in the equation shall be substituted by y_{pn}, y'_{pn}, y_{sn}, y'_{sn}, respectively.

6.1.8 In checking the cracking resistance of the cross-section and inclined section at the end of a pretensioned member, the stress of prestressing steel is taken as zero at the end of the member and taken as the effective prestress σ_{pe} at the end of the transfer length l_{tr}, while it is taken by linear interpolation at other sections within the transfer length (Figure 6.1.8). The transfer length in prestressing steel shall be taken from Table 6.1.8.

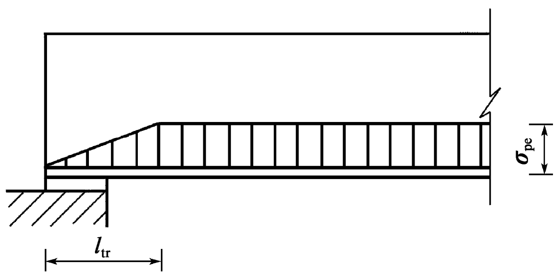


Figure 6.1.8 Effective stress within the transfer length in prestressing steel

Table 6.1.8 Transfer length in prestressing steel

Type of prestressing steel	Concrete strength class			
	C40	C45	C50	≥C55
Strand of 1 × 7, σ_{pe} = 1000 MPa	67 d	64 d	60 d	58 d
Deformed wire, σ_{pe} = 1000 MPa	58 d	56 d	53 d	51 d

Note: 1. Transfer length shall be determined based on the cube compressive strength of concrete when prestressing steel is released, f'_{cu}. If f'_{cu} is between two strength classes of concrete shown in the table, the transfer length shall be computed by linear interpolation.
2. If the effective prestress of prestressing steel, σ_{pe}, is different from the value shown in the table, the transfer length shall be increased or decreased according to the proportion of actual stress to the stress stipulated in the table.
3. For members constructed by the method of abrupt release of the prestressing steel, the starting point of the transfer length in the calculation is $0.25l_{tr}$ from the members' end.
4. d is the nominal diameter of the prestressing steel.

6.2 Loss of Prestress

6.2.1 In the calculation for the serviceability limit state of prestressed members, loss of prestress due to the following factors shall be taken into account:

Friction between prestressing steel and duct wall	σ_{l1}
Anchorageset, reinforcement retraction and joint compression	σ_{l2}
Temperature difference between prestressing steel and jacking frame	σ_{l3}
Elastic shortening of concrete	σ_{l4}
Stress relaxation of prestressing steel	σ_{l5}
Shrinkage and creep of concrete	σ_{l6}

Besides, the losses of prestress due to other factors, such as friction between prestressing steel and the anchorages, and elastic deformation of the jacking frame, shall also be considered.

Loss of prestress should be determined based on test data. If no reliable test data is available, it may be calculated according to the provisions in this section.

6.2.2 Loss of prestress due to friction between prestressing steel and duct wall may be calculated by Eq. (6.2.2):

$$\sigma_{l1} = \sigma_{con}\left[1 - e^{-(\mu\theta + kx)}\right] \tag{6.2.2}$$

where:

σ_{con}—maximum jacking stress ahead of anchorage of prestressing steel;

μ—friction coefficient between prestressing steel and duct wall, which is taken from Table 6.2.2;

θ—sum of tangent angular change of the curved duct from the jacking end to the design section (rad);

k—coefficient for the influence of partial deviation (wobble) per unit meter of duct on friction, which is taken from Table 6.2.2;

x—length of duct from the jacking end to the design section (m), it may be taken approximately as the projected length of the duct on the longitudinal axis of the member.

Table 6.2.2 Coefficients k and μ

Type of prestressing steel	Duct type	k	μ	
			Steel strand, steel wire bundle	Prestressing threaded bar
Internal tendon	Embedded corrugated metal duct	0.0015	0.20 ~ 0.25	0.50
	Embedded corrugated plastic duct	0.0015	0.15 ~ 0.20	—
	Embedded iron sheet duct	0.0030	0.35	0.40
	Embedded steel duct	0.0010	0.25	—
	Formed with removable cores	0.0015	0.55	0.60
External tendon	Steel tube	0	0.20 ~ 0.25 (0.08 ~ 0.10)	—
	High-density polyethylene duct	0	0.12 ~ 0.15 (0.08 ~ 0.10)	—

Note: For loss of prestress caused by friction between the external strand and duct wall, only the duct section with deviators and anchorage devices are counted, coefficients k and μ should be determined by test data. If there are no reliable test data, coefficients k and μ are taken from Table 6.2.2. For coefficient μ, the value in the parentheses is used for unbonded steel strands and outside the parentheses for smooth steel strands, respectively.

6.2.3 Losses of prestress due to anchorage set, reinforcement retraction and joint compression may be calculated in accordance with the following provisions:

1 For straight prestressing steel,

$$\sigma_{l2} = \frac{\sum \Delta l}{l} E_{\mathrm{p}} \tag{6.2.3}$$

where:

Δl—magnitude of anchorage set, reinforcement retraction and joint compression, which is taken from Table 6.2.3;

l—distance from the jacking end to the anchoring end.

Table 6.2.3 Magnitude of anchorage set, reinforcement retraction and joint compression

Type of anchorage and joint		Δl(mm)	Type of anchorage and joint	Δl(mm)
Steel conical anchorage for steel wire bundle		6	Button-head anchorage	1
Wedge anchorage	With pressure on wedge	4	Gap of each added pad	2
	Without pressure on wedge	6	Cement mortar joint	1
Gap of nut in anchorage		1 ~ 3	Epoxy resin mortar joint	1

Note: If the anchorage with nut is anchored after one time jacking, 2 ~ 3 mm should be taken for Δl; if it is anchored after jacking twice, 1 mm may be taken for Δl.

2 For curved prestressing steel, the calculation may be carried out by referring to Appendix G of the *Specifications*.

6.2.4 Loss of prestress due to the temperature difference between prestressing steel and jacking frame, σ_{l3}(MPa), may be calculated by Eq. (6.2.4):

$$\sigma_{l3} = 2(t_2 - t_1) \qquad (6.2.4)$$

where:

t_2—highest temperature of tension reinforcement during heat curing for concrete (℃);

t_1—field temperature during jacking (℃).

Note: In order to reduce loss of prestress due to temperature difference, multi-step curing measures may be adopted.

6.2.5 Loss of prestress due to elastic shortening of concrete may be calculated in accordance with the following provisions:

1 For post-tensioned members, when multi-step jacking is adopted, loss of prestress in the early batch of tensioned prestressing steel due to the elastic shortening caused by the latter batch of tensioned prestressing steel may be calculated by Eq. (6.2.5-1):

$$\sigma_{l4} = \alpha_{EP} \sum \Delta\sigma_{pc} \qquad (6.2.5\text{-}1)$$

where:

$\Delta\sigma_{pc}$—normal stress of concrete caused by the latter batch of jacked prestressing steel at the centroid of the early batch of jacked prestressing steel in the design section;

α_{EP}—ratio of elastic modulus of prestressing steel to that of concrete.

2 For pretensioned members, when prestressing steel is released, the loss of prestress due to elastic shortening of concrete may be calculated by Eq. (6.2.5-2):

$$\sigma_{l4} = \alpha_{EP}\sigma_{pc} \qquad (6.2.5\text{-}2)$$

where:

$\Delta\sigma_{pc}$—normal stress of concrete at the centroid of reinforcement in the investigated section, caused by all prestressing forces of reinforcement.

Note: For post-tensioned members, the simplified calculation method for the loss of prestress due to elastic shortening of concrete is depicted in Appendix H of the *Specifications*.

6.2.6 Loss of prestress due to relaxation of prestressing steel may be calculated in accordance with the following provisions:

1 Prestressing steel wires and strands:

$$\sigma_{l5} = \Psi \cdot \zeta\left(0.52\frac{\sigma_{pe}}{f_{pk}} - 0.26\right)\sigma_{pe} \tag{6.2.6-1}$$

where:

ψ—coefficient for jacking. For one-time jacking, $\psi = 1.0$; for overstress jacking, $\psi = 0.9$; (Translator's note: one-time jacking refers to jacking the prestressing steel to its design prestressing force at one time, while overstress jacking refers to jacking the prestressing steel to a prestressing force over the design value and then retracting to its design value);

ζ—coefficient of reinforcement relaxation. For Class-I relaxation (normal relaxation), $\zeta = 1.0$; for Class-II relaxation (low relaxation), $\zeta = 0.3$;

σ_{pe}—stress of prestressing steel when it is anchored by force transition. For post-tensioned members, $\sigma_{pe} = \sigma_{con} - \sigma_{l1} - \sigma_{l2} - \sigma_{l4}$; for pretensioned members, $\sigma_{pe} = \sigma_{con} - \sigma_{l2}$.

2 Prestressing threaded bar:

For one-time jacking,

$$\sigma_{l5} = 0.05\sigma_{con} \tag{6.2.6-2}$$

For overstress jacking,

$$\sigma_{l5} = 0.035\sigma_{con} \tag{6.2.6-3}$$

Note: 1. When the loss of prestress due to relaxation of prestressing steel by overstressing is adopted, the jacking procedure shall comply with the related specifications of China.

2. If the loss of prestress of prestressing steel wire and strand due to relaxation is calculated stage by stage, the ratio of median to ultimate values of prestress loss may be taken from Appendix C of the *Specification*.

6.2.7 Loss of prestress due to shrinkage and creep of concrete may be calculated in accordance with the following equations:

$$\sigma_{l6}(t) = \frac{0.9[E_p\varepsilon_{cs}(t,t_0) + \alpha_{EP}\sigma_{pc}\varphi(t,t_0)]}{1 + 15\rho\rho_{ps}} \tag{6.2.7-1}$$

$$\sigma'_{l6} = \frac{0.9\left[E_p \varepsilon_{cs}(t,t_0) + \alpha_{EP}\sigma'_{pc}\varphi(t,t_0)\right]}{1 + 15\rho'\rho'_{ps}} \tag{6.2.7-2}$$

$$\rho = \frac{A_p + A_s}{A},\ \rho' = \frac{A'_p + A'_s}{A} \tag{6.2.7-3}$$

$$\rho_{ps} = 1 + \frac{e_{ps}^2}{i^2},\ \rho'_{ps} = 1 + \frac{e'^2_{ps}}{i^2} \tag{6.2.7-4}$$

$$e_{ps} = \frac{A_p e_p + A_s e_s}{A_p + A_s},\ e'_{ps} = \frac{A'_p e'_p + A'_s e'_s}{A'_p + A'_s} \tag{6.2.7-5}$$

where:

$\sigma_{l6}(t)$, $\sigma'_{l6}(t)$—loss of prestress due to shrinkage and creep of concrete at the cross-sectional centroid of all longitudinal reinforcement in the compression and tension zones of the member, respectively;

σ_{pe}, σ'_{pe}—normal compressive stress of concrete due to prestressing force at the cross-sectional centroid of all longitudinal reinforcement in the compression and tension zones of the member, respectively, which shall be calculated in accordance with the provisions of Clause 6.1.6 and Clause 6.1.7 of the *Specifications*. Herein, if only the prestress loss of the prestressing steel in jacking (first batch) is taken into account, the prestress loss of σ_{l6} and σ'_{l6} shall be taken as zero for reinforcing steel. The value of σ_{pc} and σ'_{pc} shall not exceed 50% of the cube compressive strength of concrete, f'_{cu}, when prestressing steel is anchored by force transmission. When σ'_{pc} is tensile stress, it shall be taken as zero. In calculating σ_{pc} and σ'_{pc}, the influence of member self-weight may be considered based on the fabrication conditions;

E_p—elastic modulus of prestressing steel;

α_{EP}—ratio of elastic modulus of prestressing steel to that of concrete;

ρ, ρ'—all longitudinal reinforcement ratio in the compression and tension zones, respectively;

A—cross-sectional area of a member. For pretensioned members, $A = A_0$; for post-tensioned members, $A = A_n$;

i—cross-sectional gyratory radius, $i^2 = I/A$. For pretensioned members, $I = I_0$, $A = A_0$; for post-tensioned members, $I = I_n$, $A = A_n$;

e_p, e'_p—distance from the centroids of prestressing steel in the compression and tension zones of the member to the center of gravity of the cross-section of the member, respectively;

e_s, e'_s—distance from the centroids longitudinal reinforcing steel in the compression and tension zones of the member to the center of gravity of the cross-section of the member, respectively;

e_{ps}, e'_{ps}—distance from the centroids of prestressing steel and longitudinal reinforcing steel in the compression and tension zones of the member to the gravity axis of the cross-section of the member, respectively;

$\varepsilon_{cs}(t,t_0)$—shrinkage strain from the concrete age when prestressing force is transferred and prestressing steel is anchored, t_0, to the concrete age being considered for calculating, t. It may be calculated according to Appendix C of the *Specifications*;

$\varphi(t,t_0)$—creep coefficient of concrete when the loading age is t_0 and the calculating age is t, which may be calculated according to Appendix C of the *Specifications*.

6.2.8 Loss of prestress of prestressed concrete members at each stage may be combined based on the provisions in Table 6.2.8:

Table 6.2.8 Combination of prestress loss in each stage

Combination of prestress losses	Pretensioned member	Post-tensioned member with internal tendon	Post-tensioned member with both internal and external tendons	
			Internal tendon	External tendon
Loss at transfer and anchorage (for the first batch) $\sigma_{l\mathrm{I}}$	$\sigma_{l2}+\sigma_{l3}+\sigma_{l4}+0.5\sigma_{l5}$	$\sigma_{l1}+\sigma_{l2}+\sigma_{l4}$		
Loss after transfer and anchorage (for the second batch) $\sigma_{l\mathrm{II}}$	$0.5\sigma_{l5}+\sigma_{l6}$	$\sigma_{l5}+\sigma_{l6}$		

6.3 Check for Cracking Resistance

6.3.1 Cracking resistances of cross-section and inclined section of prestressed concrete flexural members shall be checked in accordance with the following provisions:

1 Cross-sectional tensile stress of concrete shall satisfy the following requirements:

1) Fully prestressed concrete members

For precast members,

$$\sigma_{st}-0.85\sigma_{pc}\leqslant 0 \quad (6.3.1\text{-}1)$$

For cast-in-place or mortar jointed precast longitudinal segments,

$$\sigma_{st}-0.80\sigma_{pc}\leqslant 0 \quad (6.3.1\text{-}2)$$

2) Type A prestressed concrete members

$$\sigma_{st}-\sigma_{pc}\leqslant 0.7f_{tk} \quad (6.3.1\text{-}3)$$

$$\sigma_{lt}-\sigma_{pc}\leqslant 0 \quad (6.3.1\text{-}4)$$

3) For Type B prestressed concrete flexural members, compressive stress at extreme tension fiber of critical section under self-weight shall not be decompressed.

2 Principal tensile stress σ_{tp} in the concrete of the inclined section shall satisfy the following requirements:

1) Fully prestressed concrete members

For precast members,

$$\sigma_{tp} \leqslant 0.6 f_{tk} \tag{6.3.1-5}$$

For cast-in-place members (including precast segmental members),

$$\sigma_{tp} \leqslant 0.4 f_{tk} \tag{6.3.1-6}$$

2) Type A and Type B Prestressed concrete members

For precast members,

$$\sigma_{tp} \leqslant 0.7 f_{tk} \tag{6.3.1-7}$$

For cast-in-place members (including precast segmental members),

$$\sigma_{tp} \leqslant 0.5 f_{tk} \tag{6.3.1-8}$$

where:

σ_{st}—normal tensile stress of concrete at the extreme fiber of the section under investigation for cracking resistance in a member subjected to the frequent combination of actions. The normal tensile stress σ_{st} is calculated by Eq. (6.3.2-1);

σ_{lt}—normal tensile stress of concrete at the extreme fiber of the section under investigation for cracking resistance in a member subjected to the quasi-permanent combination of actions. The normal tensile stress σ_{st} is calculated by Eq. (6.3.2-2);

σ_{pc}—prestress on concrete at the extreme fiber of the section under investigation for cracking resistance. The prestress is caused by the prestressing force after subtracting all prestress losses and it is calculated in accordance with the provisions in Clause 6.1.6 of the *Specifications*;

σ_{tp}—principal tensile stress of concrete caused by the frequent combination of actions and prestressing force, which is calculated in accordance with the provisions in Clause 6.1.6 of the *Specifications*;

f_{tk}—characteristic tensile strength of concrete, which is obtained from Table 3.1.3 of the *Specifications*.

6.3.2 The normal tensile stress of concrete at the extreme fiber of the section under investigation for cracking resistance shall be calculated by the following equations:

$$\sigma_{st} = \frac{M_s}{W_0} \tag{6.3.2-1}$$

$$\sigma_{lt} = \frac{M_l}{W_0} \tag{6.3.2-2}$$

where:

M_s—bending moment due to the frequent combination of actions;

M_l—bending moment due to the quasi-permanent combination of self-weight of the structure and those loads directly acting on it, including vehicle, pedestrian and wind loads.

Note: For post-tensioned members, in calculating the tensile stress due to the self-weight of the members in the jacking stage, W_0 in Eq. (6.3.2-1) and Eq. (6.3.2-2) may be substituted by W_n. Herein, W_n is the elastic moment of the net cross-section with respect to extreme tension fiber in the check of cracking resistance.

6.3.3 Principal compressive and tensile stress of concrete in a prestressed concrete flexural member under frequent combination of actions and prestressing forces shall be calculated by the following equations:

$$\begin{matrix}\sigma_{tp} \\ \sigma_{cp}\end{matrix} = \frac{\sigma_{cx} + \sigma_{cy}}{2} \mp \sqrt{\left(\frac{\sigma_{cx} - \sigma_{cy}}{2}\right)^2 + \tau^2} \tag{6.3.3-1}$$

$$\sigma_{cx} = \sigma_{pc} + \frac{M_s y_0}{I_0} \tag{6.3.3-2}$$

$$\sigma_{cy} = \sigma_{cy,pv} + \sigma_{cy,ph} + \sigma_{cy,t} + \sigma_{cy,l} \tag{6.3.3-3}$$

$$\sigma_{cy,pv} = 0.6\frac{n\sigma'_{pe}A_{pv}}{bs_p} \tag{6.3.3-4}$$

$$\tau = \frac{V_s S_0}{bI_0} - \frac{\sum \sigma''_{pe} A_{pb} \sin\theta_p \cdot S_n}{bI_n} \tag{6.3.3-5}$$

where:

σ_{cx}—normal stress of concrete at the point of the calculated principal stress, which is caused by prestressing force and bending moment M_s under frequent combination of actions;

σ_{cy}—vertical compressive stress of concrete;

$\sigma_{cy,pv}$, $\sigma_{cy,ph}$, $\sigma_{cy,t}$, $\sigma_{cy,l}$—frequent vertical compressive stress of concrete caused by prestressing forces of vertical prestressing steel, by transverse prestressing steel, by transverse temperature gradient, and by vehicular loads, respectively;

τ—shear stress of concrete at the principal stress point, which is induced by the prestressing force of curved tendon and the shear force V_s caused by the frequent combination of actions. Shear stress caused by torsional moment shall also be taken into account if there is such a moment in the investigated section;

σ_{pc}—normal stress of concrete at the principal stress point, which is caused by the longitudinal prestressing force after subtracting all prestress losses. It is calculated by Eq. (6.1.6-1) or Eq. (6.1.6-4);

y_0—distance from the gravity axis of the transformed section to the point of calculated principal stress;

n—number of vertical prestressing steel in the same cross-section;

σ'_{pe}, σ''_{pe}— effective prestress of vertical prestressing steel and longitudinal curved tendon after subtracting all prestress losses;

A_{pv}—cross-sectional area of single vertical prestressing steel;

s_p—spacing of vertical prestressing steel;

b—width of web plate at the point of calculated principal stress;

A_{pb}—cross-sectional area of the curved tendon in the same bent-up plane on the design section;

S_0, S_n—statical moment of the transformed section above (or under) the point of calculated principal stress with respect to its gravity axis, and area moment of the net cross-section with respect to its gravity axis, respectively;

θ_p—angle between the tangent of the curved tendon and the longitudinal axis of the member in the section under investigation.

Note: For σ_{cx}, σ_{cy}, σ_{pc} and $\frac{M_s y_0}{I_0}$ In Eq. (6.3.3-1) and Eq. (6.3.3-2), the positive sign means compression and the negative sign means tension.

6.4 Check for Crack Width

6.4.1 Crack width of reinforced concrete members and Type B prestressed members shall be checked under the frequent combination of actions with consideration of the influence of long-term effect.

6.4.2 The calculated maximum crack width of reinforced concrete members and Type B prestressed members in all kinds of environments shall not exceed the limit specified in Table 6.4.2.

Table 6.4.2 Maximum crack width

Category of environment	Maximum crack width (mm)	
	Reinforced concrete members, Type B prestressed concrete members with prestressing threaded bar	Type B prestressed concrete members with prestressing steel strands or steel wires
Class I-General	0.20	0.10
Class II-Freeze-thaw	0.20	0.10
Class III-Coastal or marine chloride	0.15	0.10

continued

Category of environment	Maximum crack width (mm)	
	Reinforced concrete members, Type B prestressed concrete members with prestressing threaded bar	Type B prestressed concrete members with prestressing steel strands or steel wires
Class IV-Deicing salt and other chlorides	0.15	0.10
Class V-Salt crystallization	0.10	Prohibition of use
Class VI-Chemical corrosive	0.15	0.10
Class VII-Abrasion	0.20	0.10

6.4.3 For flexural members of reinforced concrete and Type B prestressed concrete, the maximum crack width W_{cr} (mm) may be calculated by Eq. (6.4.3):

$$W_{\mathrm{cr}} = C_1 C_2 C_3 \frac{\sigma_{\mathrm{ss}}}{E_{\mathrm{s}}} \left(\frac{c + d}{0.36 + 1.7\rho_{\mathrm{te}}} \right) \tag{6.4.3}$$

where:

C_1—surface shape factor for reinforcement. For plain bar, $C_1 = 1.40$; for deformed bar, $C_1 = 1.00$; for deformed bar with epoxy resin coating, $C_1 = 1.15$;

C_2—influence coefficient for long-term effect, $C_2 = 1 + 0.5\frac{M_l}{M_{\mathrm{s}}}$, where, M_l and M_{s} are the designed bending moments (or axial forces) calculated according to frequent combination of actions and quasi-permanent combination of actions in Clause 6.3.2 of the *Specifications*;

C_3—coefficient related to the mechanical properties of the members. For flexural reinforced concrete slabs, $C_3 = 1.15$. For other flexural members, $C_3 = 1.0$. For axially loaded tension members, $C_3 = 1.2$. For eccentrically loaded tension members, $C_3 = 1.1$. For eccentrically loaded compression members with circular sections, $C_3 = 0.75$. For eccentrically loaded compression members with other sections, $C_3 = 0.9$;

σ_{ss}—stress in reinforcement, which is calculated according to Clause 6.4.4 of the *Specifications*;

c—thickness of concrete cover for the outmost row of longitudinal tension reinforcement (mm). If $c > 50$ mm, 50 mm is taken;

d—diameter of longitudinal tension reinforcement (mm). When reinforcement with different diameters is used, d is changed to the equivalent diameter d_{e}, $d_{\mathrm{e}} = \frac{\sum n_i d_i^2}{\sum n_i d_i}$, in which n_i is the number of reinforcements of the ith type in the tension zone, d_i is the diameter of reinforcement of the ith type in the tension zone, which is taken from Table 6.4.3. For welded reinforcement frame in Clause 9.3.11 of the *Specifications*, d and d_{e} in Eq. (6.4.3-1) shall be multiplied by a factor of 1.3;

ρ_{te}—effective reinforcement ratio for longitudinal tension reinforcement, which is calculated according to Clause 6.4.5 of the *Specifications*. If $\rho_{te} > 1.0$, $\rho_{te} = 1.0$ is adopted. If $\rho_{te} < 0.01$, $\rho_{te} = 0.01$ is adopted.

Table 6.4.3 Diameter of reinforcement in tension zone

Type of reinforcement in tension zone	Single reinforcing steel	Bundle of reinforcing steel	Bundle of steel strands	Bundle of steel wires
Value of d_i	Nominal diameter d	Equivalent diameter d_{se}	Equivalent diameter d_{pe}	

Note: 1. $d_{se} = \sqrt{n}d$, n is the number of reinforcing steel in the bundle, and d is the nominal diameter of a single reinforcing steel.

2. $d_{pe} = \sqrt{n}d_p$, n is the number of steel strands or steel wires in a bundle, and d_p is the nominal diameter of a single steel strand or steel wire.

Crack width may not be checked for those members including eccentrically loaded compression members with rectangular sections, T-sections and I- sections that satisfy the requirement of $e_0/h \leqslant 0.55$, or eccentrically loaded compression members with circular sections that satisfy the requirement of $e_0/r \leqslant 0.55$.

6.4.4 Stress of longitudinal tension reinforcement in the cracking section under frequent combination of actions, σ_{ss}, may be calculated by the following equations:

1 Reinforced concrete members with rectangular sections, T-sections or I-sections

Axially loaded tension members

$$\sigma_{ss} = \frac{N_s}{A_s} \tag{6.4.4-1}$$

Flexural members

$$\sigma_{ss} = \frac{M_s}{0.87A_s h_0} \tag{6.4.4-2}$$

Eccentrically loaded tension members

$$\sigma_{ss} = \frac{N_s e'_s}{A_s(h_0 - a'_s)} \tag{6.4.4-3}$$

Eccentrically loaded compression members

$$\sigma_{ss} = \frac{N_s(e_s - z)}{A_s z} \tag{6.4.4-4}$$

$$z = \left[0.87 - 0.12(1 - \gamma'_f)\left(\frac{h_0}{e_s}\right)^2\right]h_0 \tag{6.4.4-5}$$

$$e_s = \eta_s e_0 + y_s \tag{6.4.4-6}$$

$$\gamma'_f = \frac{(b'_f - b)h'_f}{bh_0} \tag{6.4.4-7}$$

$$\eta_s = 1 + \frac{1}{4000e_0/h}\left(\frac{l_0}{h}\right)^2 \tag{6.4.4-8}$$

where:

A_s—cross-sectional area of longitudinal reinforcement in the tension zone. For axially loaded tension members, it is the total cross-sectional area of longitudinal reinforcement. For flexural members, tension and compression members with large eccentricity, it is the cross-sectional area of longitudinal reinforcement in the tension zone or at the major tension side;

e'_s—distance from the axial tension force to the resultant force in the longitudinal reinforcement in the compression zone or in the minor tension side;

e_s—distance from the axial compression force to the resultant force in longitudinal tension reinforcement;

z—distance from the resultant force in longitudinal tension reinforcement to that the resultant force of compression zone, and no larger than $0.87h_0$;

η_s—amplification factor for eccentricity of axial compression force at the serviceability limit state. If $l_0/h \leqslant 14$, $\eta_s = 1.0$ is taken;

y_s—distance from the cross-sectional gravity to the resultant force in longitudinal tension reinforcement;

γ'_f—ratio of the cross-sectional area of the compression flange to the effective cross-sectional area of the web;

b'_f, h'_f—width and depth of flange in the compression zone, respectively. In Eq. (6.4.4-7), if $h'_f > 0.2h_0$, $h'_f = 0.2h_0$ is taken;

N_s, M_s—calculated axial force and bending moment under the frequent combination of actions, respectively.

2 Eccentrically loaded compression members with circular section

$$\sigma_{ss} = \frac{0.6\left(\frac{\eta_s e_0}{r} - 0.1\right)^3}{\left(0.45 + 0.26\frac{r_s}{r}\right)\left(\frac{\eta_s e_0}{r} + 0.2\right)^2}\frac{N_s}{A_s} \tag{6.4.4-9}$$

$$\eta_s = 1 + \frac{1}{4000\frac{e_0}{2r - a_s}}\left(\frac{l_0}{2r}\right)^2 \tag{6.4.4-10}$$

where:

A_s—total cross-sectional area of longitudinal reinforcement;

N_s—calculated axial force under frequent combinations of actions;

r_s—radius of the circle where the centroid of longitudinal reinforcement is located;

r—radius of the circular section;

e_0—initial eccentricity of members;

a_s—distance from the center of single reinforcing steel to the edge of the member;

η_s—amplification factor of the eccentricity of axial compression force in the serviceability limit state. If $l_0/2r \leqslant 14.0$, $\eta_s = 1.0$ is taken.

3 Type B prestressed concrete flexural members

$$\sigma_{ss} = \frac{M_s \pm M_{p2} - N_{p0}(z - e_p)}{(A_p + A_s)z} \tag{6.4.4-11}$$

$$e = e_p + \frac{M_s \pm M_{p2}}{N_{p0}} \tag{6.4.4-12}$$

where:

z—distance from the resultant force in longitudinal reinforcing steel and prestressing steel in the tension zone to the resultant force of compression zone, which is calculated by Eq. (6.4.4-5), however, e_s in the equation is substituted by e in Eq. (6.4.4-12);

e_p—when the normal stress of concrete is zero, distance from the resultant force in longitudinal reinforcing steel and prestressing steel, N_{p0}, to the resultant force in longitudinal reinforcing and prestressing steel in tension zone;

N_{p0}—resultant force in prestressing and reinforcing steel when the normal stress of concrete is zero, which is calculated by Eq. (6.1.7-1) for both pretensioned members and post-tensioned members, where σ_{p0} and σ'_{p0} in the equation for pretensioned members are calculated by Eq. (6.1.6-2), and for post-tensioned members are calculated by Eq. (6.1.6-5);

M_{p2}— secondary bending moment due to prestressing force N_p in post-tensioned continuous beams and other statically indeterminate structures.

Note: In Eq. (6.4.4-11) and Eq. (6.4.4-12), when the acting direction of M_{p2} is the same as that of M_s, the positive sign is adopted, otherwise, the negative sign is adopted.

6.4.5 Effective reinforcement ratio for longitudinal tension reinforcement, ρ_{te}, may be calculated using the following equations:

1 Reinforced concrete members with a rectangular section, T-section or I-section

$$\rho_{te} = \frac{A_s}{A_{te}} \tag{6.4.5-1}$$

where:

A_s—cross-sectional area of longitudinal reinforcement in the tension zone. For axially loaded tension members, the total cross-sectional area of longitudinal reinforcement is taken. For flexural members, tension and compression members with large eccentricity, the cross-sectional area of longitudinal reinforcement in the tension zone or at the major tension side is taken;

A_{te}—effective cross-sectional area of tension concrete. For axially loaded tension members, the cross-sectional area of the member is taken. For flexural members, eccentrically loaded tension or compression members, $2a_s b$ is taken, where a_s is the distance from the centroid of tension reinforcement to the edge of the tension zone. For rectangular sections, b is the cross-sectional width; for T-sections or I-sections whose flange is located in the tension zone, b is the effective width of a flange in the tension zone.

2 Reinforced concrete members with circular section

$$\rho_{te} = \frac{\beta A_s}{\pi(r^2 - r_1^2)} \tag{6.4.5-2}$$

$$r_1 = r - 2a_s \tag{6.4.5-3}$$

$$\beta = (0.4 + 0.25\rho)\left[1 + 0.353\left(\frac{\eta_s e_0}{r}\right)^{-2}\right] \tag{6.4.5-4}$$

$$\rho = \frac{A_s}{\pi r^2} \tag{6.4.5-5}$$

where:

β —coefficient for the contribution of longitudinal tension reinforcement against cracking;

A_s—total cross-sectional area of longitudinal reinforcement;

r_1—difference of circular section radius and two times of the distance from the center of single reinforcement to the edge of the member;

ρ—longitudinal reinforcement ratio.

6.5 Check for Deflection

6.5.1 Deflection of reinforced and prestressed concrete flexural members may be calculated by the method of structural mechanics according to the given stiffness of the members.

6.5.2 Stiffness of flexural members may be calculated according to the following equations:

1 Reinforced concrete members

If $M_s \geqslant M_{cr}$,

$$B = \frac{B_0}{\left(\frac{M_{cr}}{M_s}\right)^2 + \left[1 + \left(\frac{M_{cr}}{M_s}\right)^2\right]\frac{B_0}{B_{cr}}} \tag{6.5.2-1}$$

If $M_s < M_{cr}$,

$$B = B_0 \tag{6.5.2-2}$$

$$M_{cr} = \gamma f_{tk} W_0 \tag{6.5.2-3}$$

where:

B —flexural stiffness of equivalent section of the cracked member;

B_0—flexural stiffness of the whole section, $B_0 = 0.95E_cI_0$;

B_{cr}—flexural stiffness of the cracked section, $B_{cr} = E_{cr}I_{cr}$;

M_s—calculated bending moment under frequent combination of actions;

M_{cr}—cracking moment;

γ—influence coefficient of concrete plasticity in the tension zone of the member, which is calculated by Eq. (6.5.2-8);

I_0—moment of inertia of transformed section of the whole section;

I_{cr}—moment of inertia of transformed section for the cracked section;

f_{tk}—characteristic tensile strength of concrete.

2 Prestressed concrete member

1) Fully prestressed concrete member and Type A prestressed concrete member

$$B_0 = 0.95E_cI_0 \tag{6.5.2-4}$$

2) Type B prestressed concrete members permitted to crack

Under the action of the cracking moment

$$B_0 = 0.95E_cI_0 \tag{6.5.2-5}$$

Under the action of (M_s-M_{cr})

$$B_{cr} = E_{cr}I_{cr} \tag{6.5.2-6}$$

The cracking moment is calculated by the following equation

$$M_{cr} = (\sigma_{pc} + \gamma f_{tk})W_0 \tag{6.5.2-7}$$

$$\gamma = \frac{2S_0}{W_0} \tag{6.5.2-8}$$

where:

S_0—statical moment of the partial area above (or below) the gravity axis of the transformed section with respect to the gravity axis, where the whole cross-section is transformed;

σ_{pc}—precompressive stress on concrete at the edge of the investigated section, which is caused by the resultant force N_{p0} in the reinforcing and prestressing steel after all prestress losses have been subtracted. Eq. (6.1.6-1) is used in the calculation for both pretensioned and post-tensioned members, but a net cross-section is taken for post-tensioned members in the calculation. N_{p0} in the equation is the same as that in Clause 6.4.4 of the *Specifications*;

W_0—elastic moment of the transformed section with respect to extreme tension fiber in cracking resistance checking.

6.5.3 In calculating the deflection of the flexural member in service, the long-term effect shall be considered, *i.e.*, the calculated deflection under frequent combination of actions according to the stiffness obtained by provisions in Clause 6.5.2 of the *Specifications* shall be multiplied by the factor for long-term deflection, η_θ. The factor η_θ may be adopted by the following provisions:

1 If the concrete strength class of the member is below C40, $\eta_\theta = 1.60$;

2 If the concrete strength class of the member is C40 and C80, η_θ is 1.45 and 1.35, respectively; if the concrete strength class of the member is between C40 and C80, η_θ may be calculated by linear interpolation.

For long-term deflection of a flexural reinforced or prestressed concrete member calculated by the above method, the maximum deflection of a beam-type bridge under the frequent combination of vehicular load (excluding impact force) and pedestrian load shall not exceed 1/600 of the effective span; the deflection of cantilever end of a beam-type bridge shall not exceed 1/300 of the cantilever length.

6.5.4 The camber of prestressed flexural members due to prestressing force may be obtained by the product of the deflection and the factor for long-term deflection, in which the deflection is calculated using structural mechanics with the stiffness of $E_c I_0$. When the camber caused by prestressing force in the service stage is calculated, the prestressing force of the prestressing steel shall deduct the total prestress loss, and 2.0 is taken as the factor for long-term deflection.

6.5.5 Pre-camber for flexural members may be set in accordance with the following provisions:

1 Reinforced flexural members

1) If the long-term deflection caused by the frequent combination of actions does not exceed 1/1600 of the effective span, pre-camber may not be set;

2) Pre-camber shall be set if the above provisions are not satisfied. The sum of long-term deflections caused by structural self-weight and caused by 1/2 variable loads with frequent values may be adopted for the pre-camber value.

2 Prestressed concrete flexural members:

1) If the long-term camber due to prestressing force is greater than the calculated long-term deflection under frequent combination of actions, pre-camber may not be set;
2) If the long-term camber due to prestressing force is less than the calculated long-term deflection under the frequent combination of actions, pre-camber shall be set. The difference between the calculated deflection and the long-term camber shall be adopted for the pre-camber value.

For flexural members with small self-weight compared with the live load, the possible negative influence due to excessive camber caused by prestressing force shall be taken into account in the design and construction. Opposite pre-camber or other measures shall be used if necessary, to avoid upheaval, cracking or even failure of the bridge deck.

6.5.6 Finite element method should be employed in the calculation for the deformation of prestressed flexural members in the construction stages. The deformation of each element of the structure shall be accumulated by its increment at each stage, in which the influence of shrinkage and creep are calculated according to Appendix C of the *Specifications* based on the loading age of concrete, $t_{0\mathrm{i}}$, and the concrete age being considered for the calculation, t_i.

7 Stress Analysis of Members in Persistent or Transient Situations

7.1 Stress Analysis for Prestressed Concrete Members in Persistent Situations

7.1.1 In the design for prestressed concrete flexural members in persistent situations, the normal compressive stress in concrete, the tensile stress in reinforcement in the tension zone and the principal compressive stress in concrete on an inclined section at their service stage shall be calculated, and shall not exceed the limit values specified in this section. In the calculation, the characteristic actions shall be taken, and the impact effect shall be considered for the vehicular load.

7.1.2 In the calculation of normal stress in prestressed concrete members at the service stage, the compressive stress σ_{pc} and the tensile stress σ_{pt} in concrete induced by the prestressing force shall be calculated in accordance with the provisions in Clauses 6.1.6 and 6.1.7 of the *Specifications*.

7.1.3 For fully and Type A prestressed concrete flexural members, the normal stress in concrete and the stress in prestressing steel induced by the characteristic actions shall be calculated by the following formulas:

1 Normal compressive stress σ_{kc} and tensile stress σ_{kt} in concrete

$$\sigma_{kc} \text{ or } \sigma_{kt} = \frac{M_k}{I_0} y_0 \qquad (7.1.3\text{-}1)$$

2 Stress in prestressing steel

$$\sigma_p = \alpha_{EP} \sigma_{kt} \qquad (7.1.3\text{-}2)$$

where:

M_k—bending moment calculated based on the combination of characteristic actions;

y_0—distance from the gravity axis of the transformed section to the investigated point in compression or tension zone.

Note: In calculating the stress of the prestressing steel, σ_{kt} in Eq. (7.1.3-2) shall be the tensile stress of concrete at the centroid of the outmost layer of the steel bars.

7.1.4 For Type B prestressed concrete flexural members permitted to crack, the normal compressive stress of concrete and the stress increment in prestressing steel induced by the characteristic actions may be calculated by the following formulas:

1 Compressive stress of concrete on cracked section

$$\sigma_{cc} = \frac{N_{p0}}{A_{cr}} + \frac{N_{p0} e_{0N} c}{I_{cr}} \tag{7.1.4-1}$$

$$e_{0N} = e_N + c \tag{7.1.4-2}$$

$$e_N = \left(\frac{M_k \pm M_{p2}}{N_{p0}}\right) - h_{ps} \tag{7.1.4-3}$$

$$h_{ps} = \frac{\sigma_{p0} A_p h_p - \sigma_{l6} A_s h_s + \sigma'_{p0} A'_p a'_p - \sigma'_{l6} A'_s a'_s}{N_{p0}} \tag{7.1.4-4}$$

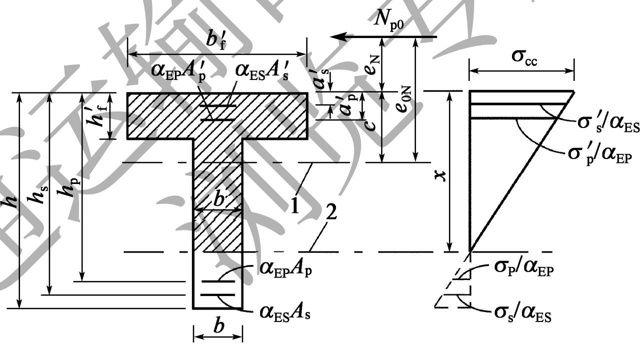


Figure 7.1.4 Stresses on cracked section

1-Gravity axis of cracked section; 2-Neutral axis of cracked section

2 Stress increment in prestressing steel of the cracked section

$$\sigma_p = \alpha_{EP}\left[\frac{N_{p0}}{A_{cr}} - \frac{N_{p0} e_{0N}(h_p - c)}{I_{cr}}\right] \tag{7.1.4-5}$$

where:

N_{p0}—resultant force in prestressing and reinforcing steel when the normal stress of concrete is zero, which is calculated by Eq. (6.1.7-1) and Clause 6.4.4 of the *Specifications* for both pretensioned members and post-tensioned members;

σ_{p0}, σ'_{p0}—stress in prestressing steel when the normal stress of concrete at the point of resultant force in prestressing steel in the compression zone and the tension zone is zero, respectively, which is calculated by Eq. (6.1.6-2) for pretensioned members and by Eq. (6.1.6-5) for post-tensioned members;

e_{0N}—distance from N_{p0} to the gravity axis of cracked section;

e_N—distance from the point of application of N_{p0} to the extreme compression fiber of the section, which is positive if N_{p0} is applied beyond the section and negative if N_{p0} is applied within the section;

c—distance from the extreme compression fiber of the section to the gravity axis of the transformed cracked section;

h_{ps}—distance from the resultant force in prestressing and reinforcing steel to the extreme compression fiber of the section;

h_p, a'_p—distance from the resultant force in prestressing steel in the tension and compression zones to the extreme compression fiber of the section, respectively;

h_s, a'_s—distance from the resultant force in reinforcing steel in the tension and compression zones to the extreme compression fiber of the section, respectively;

A_{cr}—area of transformed section for cracked section;

I_{cr}—moment of inertia of transformed section for cracked section;

α_{EP}—ratio of the modulus of elasticity of prestressing steel to that of concrete.

Notes: 1. In Eq. (7.1.4-4), σ'_{l6} shall be zero when $A'_p = 0$.

2. In Eq. (7.1.4-3), the positive sign is used when M_{p2} and M_k are in the same direction, while the negative sign is used when they are in opposite directions.

3. The value calculated from Eq. (7.1.4-5) shall be negative, representing that the steel bars are subjected to tensile stress.

4. When multiple layers of prestressing steel are arranged in the tension zone of a section, the increment of tensile stress in the outmost layer of reinforcement may be calculated only, and h_p in Eq. (7.1.4-5) shall be the distance from the centroid of the outmost layer of reinforcement to the extreme compression fiber of the section.

5. The position of the neutral axis (the depth of compression zone) in the cracked section of a prestressed concrete flexural member may be determined in accordance with Appendix J in the *Specifications*.

7.1.5 For a prestressed concrete flexural member in service, the compressive stress of concrete and the tensile stress of prestressing steel on the cross-section shall comply with the following provisions:

1 The maximum compressive stress of concrete in the compression zone

$$\left.\begin{array}{ll}\text{For members expected not to crack} & \sigma_{kc} + \sigma_{pt} \\ \text{For members permitted to crack} & \sigma_{kc}\end{array}\right\} \leqslant 0.50 f_{ck} \qquad (7.1.5\text{-}1)$$

2 The maximum tensile stress of prestressing steel in the tension zone

1) Internal prestressing steel strands and wires

$$\left.\begin{array}{ll}\text{For members expected not to crack} & \sigma_{pe}+\sigma_{p}\\ \text{For members permitted to crack} & \sigma_{p0}+\sigma_{p}\end{array}\right\}\leqslant 0.65f_{pk} \tag{7.1.5-2}$$

2) External prestressing strands

$$\sigma_{pe,ex}\leqslant 0.60f_{pk} \tag{7.1.5-3}$$

3) Prestressing threaded bars

$$\left.\begin{array}{ll}\text{For members expected not to crack} & \sigma_{pe}+\sigma_{p}\\ \text{For members permitted to crack} & \sigma_{p0}+\sigma_{p}\end{array}\right\}\leqslant 0.75f_{pk} \tag{7.1.5-4}$$

where:

σ_{pe}—effective prestress of the prestressing steel in the tension zone after subtracting all the losses of prestress for fully and Type A prestressed concrete flexural members;

σ_{pt}—normal tensile stress in concrete due to the prestressing force, which is calculated by Eq. (6.1.6-1) for pretensioned members and by Eq. (6.1.6-4) for post-tensioned members.

Note: Stress of reinforcing steel in the tension zone of prestressed concrete flexural members may not be checked.

7.1.6 The principal compressive stress σ_{cp} and the principal tensile stress σ_{tp} of concrete in prestressed concrete flexural members under the characteristic actions and the prestressing force shall be calculated in accordance with the formulas in Clause 6.3.3, whereas M_s and V_s in Eq. (6.3.3-2) and Eq. (6.3.3-5) shall be substituted by M_k and V_k, respectively. Here, M_k and V_k are the bending moment and shear force calculated by the combination of characteristic actions.

The principal compressive stress in concrete shall comply with the provision in Eq. (7.1.6-1):

$$\sigma_{cp}\leqslant 0.6f_{ck} \tag{7.1.6-1}$$

Based on the calculated principal tensile stress of concrete, stirrups are arranged in accordance with the following provisions:

In the portion where $\sigma_{tp}\leqslant 0.5f_{tk}$, stirrups may be arranged only by the detailing requirements.

In the portion where $\sigma_{tp}>0.5f_{tk}$, the stirrup spacing, s_v, may be calculated by Eq. (7.1.6-2):

$$s_v=\frac{f_{sk}A_{sv}}{\sigma_{tp}b} \tag{7.1.6-2}$$

where:

f_{sk}—characteristic tensile strength of stirrups;

A_{sv}—total cross-sectional area of stirrups in one section;

b—width of rectangular section, web width of T-section or I-section.

If the quantity of stirrups calculated by this clause is less than that calculated by the shear resistance of the inclined section, the latter shall be taken in the design.

7.2 Stress Analysis of Members in Transient Situations

7.2.1 In the design of bridge members in transient situations, stresses on cross-sections and inclined sections induced by self-weight and construction loads in the construction stage, such as fabrication, transportation and installation, shall be calculated. The calculated stresses shall not exceed the limit values specified in this section. For construction loads, their characteristic values are used in the calculation unless otherwise specified, and the load factor is not considered if they are involved in a load combination.

When the installation is carried out by cranes moving on the bridge, the installed members shall be checked, and the crane load on them shall be multiplied by a partial factor of 1.15. If the design value of the effect induced by the crane load is smaller than that calculated by the ultimate limit state in persistent situations, the checking may not be necessary.

7.2.2 In the analysis for members during transportation and installation, the self-weight of the members shall be multiplied by a dynamic factor. The dynamic factor shall comply with the provisions in *General Specifications for Design of Highway Bridges and Culverts* (JTG D60—2015).

7.2.3 When a prestressing force is applied to a member, the cube strength of concrete shall not be less than 80% of the design strength, and the modulus of elasticity shall not be less than 80% of the 28-day modulus of elasticity.

7.2.4 Stresses on cross-sections of reinforced concrete flexural members are calculated using the following formulas, and shall comply with the following provisions:

1 Compressive stress at the extreme fiber of concrete in compression

$$\sigma_{cc}^{t} = \frac{M_k^t x_0}{I_{cr}} \leqslant 0.80 f'_{ck} \tag{7.2.4-1}$$

2 Stress in tension reinforcement

$$\sigma_{si}^{t} = \alpha_{ES} \frac{M_k^t (h_{0i} - x_0)}{I_{cr}} \leqslant 0.75 f_{sk} \tag{7.2.4-2}$$

where:

M_k^t—bending moment induced by characteristic temporary construction loads;

x_0—depth of compression zone in the transformed section, which is calculated by the principle that the area moments of compression zone and tension zone in the transformed section about the neutral axis are equal;

I_{cr}—moment of inertia of the transformed section for the cracked section, which is calculated by the sum of the moments of inertia of reinforcing steel and concrete with respect to the neutral axis determined based on the calculated depth of compression zone x_0;

σ_{si}^t—stress in the reinforcement of the ith layer in the tension zone during analysis of the transient situation;

h_{0i}—distance from the extreme compression fiber to the centroid of the ith layer of reinforcement in the tension zone;

f'_{ck}—characteristic axial compressive strength of concrete at the construction stage, which corresponds to the cube compressive strength of concrete f'_{cu} and shall be determined by linear interpolation from Table 3.1.3;

f_{sk}—characteristic tensile strength of reinforcing steel, which shall be taken from Table 3.2.2-1.

7.2.5 The principal tensile stress (shear stress) σ_{tp}^t at the neutral axis of a reinforced concrete flexural member shall comply with the provision in Eq. (7.2.5):

$$\sigma_{tp}^t = \frac{V_k^t}{bz_0} \leqslant f'_{tk} \tag{7.2.5}$$

where:

V_k^t—shear force induced by characteristic construction loads;

b—width of rectangular section, web width of T-section or I-section;

z_0—distance from the resultant force in the compression zone to that in tension reinforcement, which is determined assuming the compressive stress block is triangular;

f'_{tk}—characteristic axial tensile strength of concrete during construction.

7.2.6 If the principal tensile stress at the neutral axis of a reinforced concrete flexural member complies with the following requirement:

$$\sigma_{tp}^t \leqslant 0.25 f'_{tk} \tag{7.2.6-1}$$

then, the principal tensile stress in this section is assumed to be fully carried by concrete, and the shear reinforcement may be designed according to the detailing requirements in this case.

For segments where the principal tensile stress at the neutral axis does not comply with Eq. (7.2.6-1), the principal tensile stress (shear stress) should be fully carried by stirrups and bent bars. Stirrups and bent bars may be arranged according to the shear stress block (Figure 7.2.6) and calculated by the following formulas:

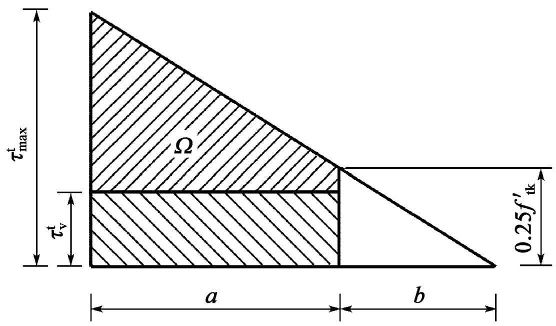


Figure 7.2.6 Distribution of shear stress in reinforced concrete flexural members

a-section where shear stress is carried by stirrups and bent bars; b-section where shear stress is carried by concrete

1 Stirrups

$$\tau^t_v = \frac{nA_{sv1}[\sigma^t_s]}{bs_v} \tag{7.2.6-2}$$

2 Bent bars

$$A_{sb} = \frac{b\Omega}{\sqrt{2}[\sigma^t_s]} \tag{7.2.6-3}$$

where:

τ^t_v—principal tensile stress (shear stress) carried by stirrups;

n—number of stirrup legs in one cross-section;

$[\sigma^t_s]$—stress limit for reinforcement in the transient situations, which is taken as $0.75f_{sk}$ according to the provisions in Clause 7.2.4 of the *Specifications*;

A_{sv1}—cross-sectional area of one leg stirrup;

s_v—stirrup spacing;

A_{sb}—total cross-sectional area of the bent bars;

Ω—area of the shear stress block carried by bent bars.

7.2.7 In the analysis of prestressed concrete flexural members in transient situations, the normal stresses induced by prestressing forces and loads may be calculated by the formulas in Clause 6.1.6 and Clause 7.1.3 of the *Specifications*. In the calculation, construction loads and the prestressing forces are used as the actions, the losses of prestress at the corresponding stage shall be subtracted from the prestress, and the properties of the section are taken in accordance with the provisions in Clause 6.1.5 of the *Specifications*.

7.2.8 For prestressed concrete flexural members, normal stresses in the extreme fiber of concrete under the construction loads, e.g., prestress and self-weight of the members, shall comply with the following provisions:

1 Compressive stress

$$\sigma_{cc}^{t} \leqslant 0.70 f'_{ck} \tag{7.2.8}$$

2 Tensile stress

1) If $\sigma_{ct}^{t} \leqslant 0.70 f'_{tk}$, the longitudinal reinforcement ratio in the pretension zone shall not be less than 0.2%;

2) If $\sigma_{ct}^{t} = 1.15 f'_{tk}$, the longitudinal reinforcement ratio in the pretension zone shall not be less than 0.4%;

3) If $0.70 f'_{tk} < \sigma_{ct}^{t} < 1.15 f'_{tk}$, the longitudinal reinforcement ratio in the pretension shall be calculated by linear interpolation between the above two values;

4) The tensile stress σ_{ct}^{t} shall not exceed $1.15 f'_{tk}$.

The above reinforcement ratio is $\frac{A'_s + A'_p}{A}$, where A'_p is included for pretensioned members, while A'_p is not included for post-tensioned members. A'_p is the cross-sectional area of prestressing steel in the pretension zone; A'_s is the cross-sectional area of reinforcing steel in the pretension zone; A is the gross area of a section of the member.

In the above formulas:

σ_{cc}^{t}, σ_{ct}^{t}—compressive and tensile stresses in the extreme fiber of concrete in precompression and pretension zones of the section in the analysis for transient situations, respectively;

f'_{ck}, f'_{tk}—characteristic axial compressive strength and characteristic axial tensile strength, corresponding to the cube compressive strength of concrete, f'_{cu}, at different construction stages, e.g., fabrication, transportation and installation, respectively, which may be calculated from Table 3.1.3 by linear interpolation.

Longitudinal reinforcement arranged in tension zones during prestressing should be deformed bars, their diameter should not be larger than 14 mm, and they shall be uniformly arranged along the outer edges of the tension zones.

8 Requirements for Member Analysis

8.1 Composite Flexural Members

8.1.1 Precast elements of composite flexural members at the construction stage shall be checked in accordance with the provisions in Section 7.2 of the *Specifications*.

8.1.2 Effects of action for composite flexural members shall be calculated for the following two phases:

1 First phase: before the cast-in-place concrete layer achieves its characteristic strength, actions including self-weights of precast members, self-weight of cast-in-place concrete layer and other additional actions during construction shall be considered.

2 Second phase: after the cast-in-place concrete layer achieves its characteristic strength, the composite beam is analyzed as a unit, and actions shall include self-weights of composite members, self-weight of bridge deck system and variable actions in service .

8.1.3 Effects of different shrinkage of concrete due to different concrete ages of precast members and cast-in-place concrete layers should be analyzed for composite flexural members.

8.1.4 Flexural resistance shall be calculated in accordance with the provisions in Section 5.2 of the *Specifications* for composite flexural members and their precast elements. Their design bending moments shall be taken in accordance with the following provisions:

For precast elements,

$$M_{1d} = M_{1Gd} + M_{1Qd} \tag{8.1.4-1}$$

For composite members (importance factor of structure, γ_0, shall be considered),

$$M_d = M_{1Gd} + M_{2Gd} + M_{2Qd} \tag{8.1.4-2}$$

where:

M_{1d}—design bending moment at the first phase due to the self-weights of precast elements and cast-in-place concrete layer which is taken as characteristic actions multiplied by a partial factor of 1.2;

M_{1Qd}—design bending moment at the first phase due to other additional actions during construction which is taken as characteristic actions multiplied by a partial factor of 1.4;

M_{2Gd}—design bending moment at the second phase due to the self-weight of the bridge deck system which is taken as the characteristic action multiplied by a partial factor of 1.2;

M_{2Qd}—design bending moment at the second phase due to the combination of variable actions in which the partial factor for actions is taken from *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015).

When the strength of the cast-in-place concrete layer is different from that of the precast element in a composite member, the strength of the cast-in-place concrete shall be used.

8.1.5 Shear resistance and flexural resistance of inclined sections shall be calculated in accordance with the provisions in Section 5.2 of the *Specifications* for composite flexural members and their precast elements, in which the partial factors for actions are taken in accordance with Clause 8.1.4 of the *Specifications*. The design shear force shall be taken in accordance with the following provisions:

For precast elements,

$$V_{1d} = V_{1Gd} + V_{1Qd} \tag{8.1.5-1}$$

For composite members (importance factor of structure, γ_0, shall be considered),

$$V_d = V_{1Gd} + V_{2Gd} + V_{2Qd} \tag{8.1.5-2}$$

where:

V_{1d}—shear force at the first phase due to the self-weights of precast elements and cast-in-place concrete layer;

V_{1Qd}—shear force at the first phase due to other additional actions during construction;

V_{2Gd}—shear force at the second phase due to the self-weight of the bridge deck system;

V_{2Qd}—shear force at the second phase due to the combination of variable actions.

For a composite member, if the concrete strengths of the cast-in-place layer and the precast element are different, the lower one shall be used in calculating the design shear resistance contributed by both concrete and stirrups on an inclined section, V_{cs} [Eq. (5.2.9-2)], and the calculated shear resistance of the composite member shall not be lower than that of the precast element. For prestressed concrete composite members, the increase coefficient due to prestress is taken as $\alpha_2 = 1.0$.

8.1.6 When the composite flexural beams conform to the detailing requirements in Clauses 9.3.16 and 9.3.17 of the *Specifications*, the shear resistance on the interface between the precast element and cast-in-place concrete layer shall satisfy the requirement in Eq. (8.1.6):

$$\gamma_0 V_d \leqslant 0.12 f_{cd} b h_0 + 0.85 f_{sv} \frac{A_{sv}}{s_v} h_0 \tag{8.1.6}$$

where:

V_d—maximum design shear force in composite beam;

f_{cd}—design axial compressive strength of concrete. If the concrete strengths in the precast element and cast-in-place concrete layer are different, the lower one shall be taken;

b—width of interface in composite beam;

h_0—effective depth of composite beam;

f_{sv}—design tensile strength of stirrup in composite beam;

A_{sv}—total cross-sectional area of each leg of the stirrup in one cross-section of the composite beam;

s_v—stirrup spacing.

8.1.7 For composite flexural members, the shear resistance on their interfaces shall satisfy the following requirement:

$$\frac{\gamma_0 V_d}{b h_0} \leqslant f_{v1} \tag{8.1.7-1}$$

where:

V_d—maximum design shear force in the composite slab (N);

b—width of interface in the composite slab (mm);

h_0—effective thickness of the composite slab (mm);

f_{v1}—design nominal shear force (MPa), which is taken as 2.00 MPa.

If Formula (8.1.7-2) is satisfied, shear reinforcement on the interface shall be arranged only in accordance with the detailing requirements in Clause 9.2.7 of the *Specifications*.

$$\frac{\gamma_0 V_d}{b h_0} \leqslant f_{v2} \tag{8.1.7-2}$$

where:

f_{v2}—design nominal shear force (MPa), which is taken as 0.45 MPa.

If Formula (8.1.7-2) is not satisfied, shear reinforcement in the interface shall be arranged in accordance with the requirement in Formula (8.1.7-3):

$$A_{sv} \geqslant 0.3 \frac{bs}{f_{sd}} \tag{8.1.7-3}$$

where:

A_{sv}—area of shear reinforcement on the interface of one cross-section (mm^2);

s—longitudinal spacing between vertical reinforcement in the interface (mm);

f_{sd}—design tensile strength of vertical reinforcement in the interface (MPa).

8.1.8 For prestressed concrete composite flexural members required not to crack at the service stage, their precast elements and composite elements shall be checked for the cracking resistance of the cross-section in accordance with Clauses 6.1.1 and 6.3.1 of the *Specifications*, respectively. In composite elements, σ_{pc} in Clause 6.3.1 is the precompressive stress at the extreme concrete fiber for cracking resistance in precast elements, f_{tk} is the characteristic tensile strength of concrete in precast elements. The normal tensile stress at the extreme concrete fiber for cracking resistance of the member under the frequent combination and quasi-permanent combination of actions shall be calculated by the following formulas:

1 For precast elements,

$$\sigma_{st} = \frac{M_{1k}}{W_{01}} \tag{8.1.8-1}$$

2 For composite elements,

$$\sigma_{st} = \frac{M_{1Gk}}{W_{01}} + \frac{M_{2s}}{W_0} \tag{8.1.8-2}$$

$$\sigma_{lt} = \frac{M_{1Gk}}{W_{01}} + \frac{M_{2l}}{W_0} \tag{8.1.8-3}$$

where:

M_{1k}—characteristic bending moment due to the actions at the first construction phase, $M_{1k} = M_{1Gk} + M_{1Qk}$, in which M_{1Gk} is the characteristic bending moment due to self-weights of precast elements and cast-in-place concrete layer at the first construction phase; M_{1Qk} is the characteristic bending moment due to other additional actions at the first construction phase;

M_{2s}—bending moment calculated based on the frequent combination of actions at the second construction phase, $M_{2s} = M_{2Gk} + \sum \psi_{1i} M_{2Qik}$, in which M_{2Gk} is the characteristic bending moment due to the self-weight of bridge deck system, M_{2Qik} is the characteristic bending moment due to the ith variable action at the service stage, and ψ_{1i} is the factor of frequent value for the ith variable action, taken in accordance with *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015);

M_{2l}—bending moment calculated based on the quasi-permanent combination of actions at the second phase, $M_{2l} = M_{2Gk} + \sum \psi_{2i} M_{2Qik}$, in which ψ_{2i} is the factor of quasi-permanent value for vehicular and pedestrian loads taken in accordance with *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015), M_{2Qik} is the characteristic bending moment due to vehicular and pedestrian loads;

W_{01}—elastic moment of the transformed section of the precast element with respect to extreme tension fiber;

W_0—elastic moment of the transformed section of the composite element with respect to extreme tension fiber. If the concrete strength of the cast-in-place concrete layer is different from that of the precast element, in the analysis, the former section shall be transformed into the latter one according to the ratio of their modulus of elasticity.

8.1.9 For prestressed concrete composite flexural members, cracking resistance shall be checked in accordance with the requirements for fully and Type A prestressed concrete members in Clause 6.3.1 of the *Specifications*, and the principal tensile stress in concrete shall be calculated in accordance with Clause 6.3.3 by considering the mechanical behaviors of the composite members.

8.1.10 The crack width shall be checked for reinforced concrete composite members. The maximum crack width under the frequent combination of actions and with the consideration of the long-term effect shall not exceed the limits specified in Clause 6.4.2 of the *Specifications*.

8.1.11 If a reinforced concrete composite flexural member is regarded as a unit, its maximum crack width may be calculated by Eq. (6.4.3), in which the influence coefficient for long-term effect C_2 and the stress in reinforcement σ_{ss} are calculated or checked by the following formulas:

1 Influence coefficient for long-term effect C_2

$$C_2 = 1 + 0.5\frac{M_{1Gk} + M_{2l}}{M_{1Gk} + M_{2s}} \tag{8.1.11-1}$$

The symbols in this formula are shown in Clause 8.1.8 of the *Specifications*, but $\sum\psi_{2i}M_{2Qik}$ in M_{2l} is the sum of all the products of the factors for the quasi-permanent variable actions involved in the combination and the corresponding characteristic bending moments. The factors of quasi-permanent variable actions shall be taken in accordance with *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015).

2 Stress in longitudinal reinforcement of reinforced concrete composite flexural members, σ_{ss}

$$\sigma_{ss} = \sigma_{s1} + \sigma_{s2} = \frac{M_{1Gk}}{0.87A_s h_{01}} + \frac{0.5\left(1 + \frac{h_1}{h}\right)M_{2s}}{0.87A_s h_0} \leqslant 0.75 f_{sk} \tag{8.1.11-2}$$

If $M_{1Gk} < 0.35M_{1u}$, $h_1 = h$ is taken in Eq. (8.1.11-2). Here, M_{1u} is the design flexural resistance on cross-sections of precast members and is calculated by Eq. (5.2.2-1) or Eq. (5.2.3-2), in which the equal sign is used and $\gamma_0 M_d$ is substituted by M_{1u}.

In the above formula,

σ_{s1}—stress of longitudinal reinforcement of the precast member caused by the characteristic bending moment M_{1Gk};

σ_{s2}—stress of longitudinal reinforcement of the composite member caused by the bending moment M_{2s};

h_1—depth of the section of precast member;

h—depth of the section of composite member;

h_{01}—effective depth of section of precast member;

h_0—effective depth of section of composite member;

A_s—cross-sectional area of reinforcement in the tension zone of the precast member.

8.1.12 Deflection of composite flexural members at the serviceability limit state may be calculated based on the given stiffness by the method of structural mechanics.

8.1.13 Stiffness of composite flexural members under the frequent combination of actions may be calculated according to the following provisions:

1 If a reinforced concrete composite member is regarded as a unit, its stiffness obtained from Eq. (6.5.2-1) shall be multiplied by a reduction factor of 0.9. In the formula, the flexural stiffness of the full section is $B_0 = 0.95E_{c1}I_0$, and the flexural stiffness of the cracked section is $B_{cr} = E_{c1}I_{cr}$. Here, E_{c1} is the modulus of elasticity of concrete in precast members.

2 If a fully or Type A prestressed concrete member is regarded as a unit, $B_0 = 0.80E_{c1}I_0$ is used as its stiffness.

8.1.14 Long-term deflection of composite flexural members may be obtained from the product of the deflection determined based on the stiffness from Clause 8.1.13 of the *Specifications* and the factor for long-term deflection η_θ:

1 For concrete with a strength class lower than C40, $\eta_\theta = 1.80$.

2 For concrete with a strength class between C40 ~ C80, $\eta_\theta = 1.65 \sim 1.55$, in which intermediate values may be taken by linear interpolation.

Long-term deflections of composite flexural members under the frequent combination of vehicular load (excluding impact force) and pedestrian load in service shall not exceed the limits specified in Clause 6.5.3 of the *Specifications*.

Note: If the concrete strength classes of the precast elements and cast-in-place concrete layers are different, the above strength grade refers to the concrete strength class of the precast element.

8.1.15 For flexural prestressed concrete composite members, the camber induced by prestressing force may be calculated by the method of structural mechanics based on the stiffness of the precast element $E_{c1}I_0$. In the calculation, all the losses of prestress shall be subtracted from the stresses in the prestressing steels. To determine the camber induced by prestressing force at the service stage, the calculated result from the above calculation shall be multiplied by a factor of 1.75 for long-term deflection.

8.1.16 The pre-camber of precast elements of composite flexural members may be calculated according to the provisions of Clause 6.5.5 of the *Specifications*.

8.1.17 Stress analysis for prestressed concrete composite flexural members in persistent situations shall be carried out in accordance with Section 7.1 of the *Specifications* by considering the mechanical behaviors of the composite members.

8.2 Post-tensioned Concrete Anchorage Zones

8.2.1 For post-tensioned members, the resistances of their prestressed anchorage zones shall meet the following requirements:

1 Compressive resistance of the local zone ahead of the anchorage device shall comply with the provisions in Section 5.7 of the *Specifications*.

2 Tensile resistance at each tensile location in the general zone shall comply with the provision in the following formula:

$$\gamma_0 T_{(\cdot),d} \leq f_{sd} A_s \tag{8.2.1}$$

where:

$T_{(\cdot),d}$—design tensile force at each tensile location in the general zone. For anchorage zones at the end of a member, bursting force $T_{b,d}$ ahead of the anchorage device, spalling force $T_{s,d}$ and edge tension force $T_{et,d}$ may be calculated in accordance with Clauses 8.2.2 ~ 8.2.5 of the *Specifications* or by strut-and-tie model. For anchorage zones of triangular blisters, the design tension forces at the five tensile locations may be calculated in accordance with Clause 8.2.6 or by strut-and-tie model;

f_{sd}—design tensile strength of reinforcing steel;

A_s—area of reinforcing steel in the tie. In the calculation, the reinforcing steel refers to that arranged in accordance with Clauses 9.4.18 and 9.4.20 of the *Specifications*.

Notes: 1. For prestressed anchorage zones at the end of a beam, the transverse dimensions of the anchorage zone are taken as the width of the end section of the beam, and its longitudinal dimension is taken as the greater one of 1.0 ~ 1.2 times the width and 1.0 ~ 1.2 times the depth of the beam. For triangular anchor blisters, the transverse extent of the anchorage zone is taken as 3 times the width of the blister, and the longitudinal extent is taken as the length of the blister plus 2 times the thickness of the wall plate.

2. The transverse extent of local zones is taken as the local compression area ahead of the anchorage device, as shown in Figure 5.7.1, and the longitudinal extent is taken as 1.2 times the larger dimension of the bearing plate.

3. The general zone refers to the anchorage zone excluding the local zone.

8.2.2 The design bursting force $T_{b,d}$ acting ahead of anchorage devices at the end anchorage zones (Figure 8.2.2) should be calculated in accordance with the following provisions.

1 Design bursting force for a single anchorage device:

$$T_{b,d} = 0.25P_d\,(1+\gamma)^2\left[(1-\gamma)-\frac{a}{h}\right]+0.5P_d\,|\sin\alpha| \tag{8.2.2-1}$$

The horizontal distance from the bursting force to the end surface of the anchorage:

$$d_b = 0.5(h-2e)+e\sin\alpha \tag{8.2.2-2}$$

where:

P_d—design anchor force for prestressing, which is taken as 1.2 times the maximum jacking force;

a—width of bearing plate;

h—depth of section at the end of the anchorage;

e—eccentricity of the anchor force, that is, the distance from the anchor force to the centroid of the section;

γ —eccentricity ratio of anchor force on the section, $\gamma = 2e/h$;

α—angle of inclination of the jacking force, which is normally between -5°~ +20°. It is positive if the anchor force points toward the centroid of the section, and negative if the anchor force points away from the centroid of the section.

2 The design bursting force acting ahead of a group of closely spaced anchorages should be calculated by substituting the resultant anchor force as the P_d in Eq. (8.2.2-1).

3 The design bursting force acting ahead of a group of non-closely spaced anchorages should be determined by the maximum one among all bursting forces which are calculated for individual anchorage devices.

Notes: 1. For a group of anchorages, if their center-to-center spacing is smaller than 2 times the width of the bearing plates [Figure 8.2.2b)], it is regarded as closely spaced anchorages; otherwise, it is regarded as non-closely spaced anchorages [Figure 8.2.2c)].

2. The total width of all bearing plates for a group of closely spaced anchorages is taken as the edge distance between two exterior bearing plates [Figure 8.2.2b)].

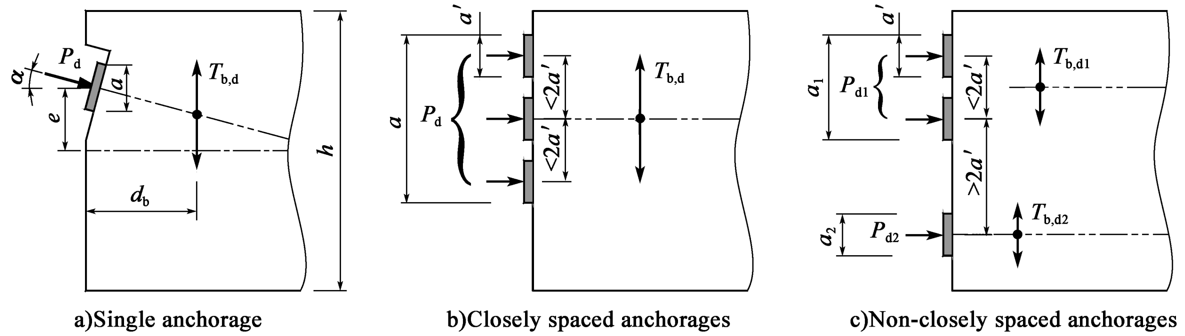


Figure 8.2.2　Bursting force acting ahead of anchorages at the end anchorage zone

8.2.3　The spalling force in the vicinity of an anchorage device due to the local compression of the bearing plate $T_{s,d}$ (Figure 8.2.3) should be calculated by Eq. (8.2.3):

$$T_{s,d} = 0.02\max\{P_{di}\} \tag{8.2.3}$$

where:

P_{di}—design value of the ith anchor force at the same end surface.

8.2.4　For a group of anchorages in which the center-to-center spacing between two anchor forces is larger than half the depth of the section at the end of the anchorages, the spalling force between the anchorages with larger spacing (Figure 8.2.4) should be calculated by Eq. (8.2.4) and should not be smaller than 0.02 times the maximum design anchor force.

$$T_{s,d} = 0.45\overline{P}_d \cdot \left(\frac{2s}{h} - 1\right) \tag{8.2.4}$$

where:

$\overline{P}_d$—average of design anchor force, that is, $\overline{P}_d = (P_{d1} + P_{d2})/2$;

s—center-to-center spacing between two anchor forces;

h—depth of section at the end of the anchorage.

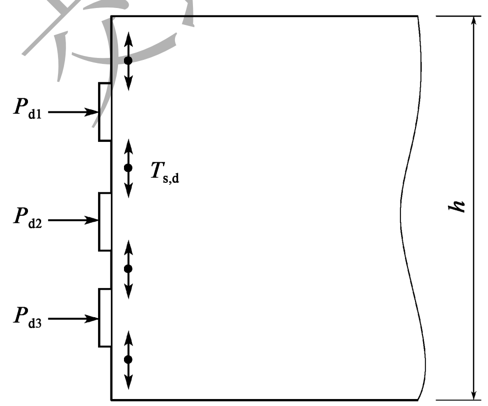


Figure 8.2.3　Spalling force in the vicinity of the anchorage

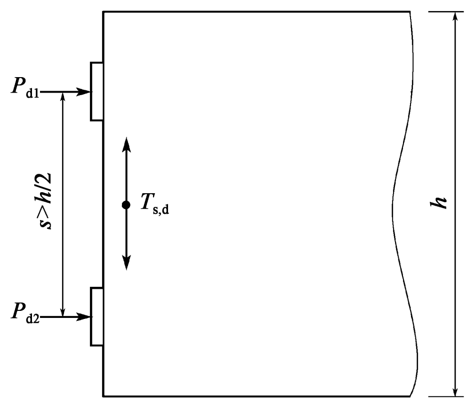


Figure 8.2.4　Spalling force between anchorages with large spacings

8.2.5　Design edge tension force in the end anchorage zone (Figure 8.2.5) should be calculated by Eq. (8.2.5):

$$T_{\mathrm{et,d}} = \begin{cases} 0 & \gamma \leqslant 1/3 \\ \dfrac{(3\gamma - 1)^2}{12\gamma} P_{\mathrm{d}} & \gamma > 1/3 \end{cases} \tag{8.2.5}$$

where:

γ—eccentricity ratio of anchor force on the section, $\gamma = 2e/h$, in which e and h are taken in accordance with Clause 8.2.2 of the *Specifications*.

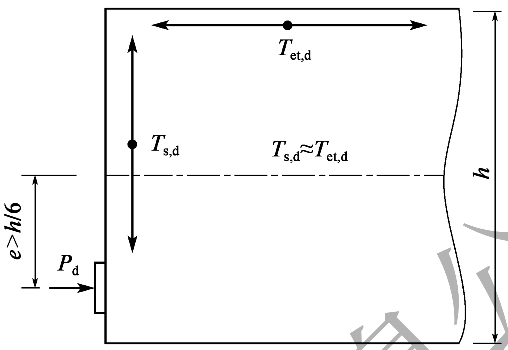


Figure 8.2.5 Edge tension force in the end anchorage zone

8.2.6 Design tension forces at the five tensile locations in the triangular anchor blister (Figure 8.2.6) should be calculated in accordance with the following provisions:

1 Design bursting force ahead of the anchorage device

$$T_{\mathrm{b,d}} = 0.25 P_{\mathrm{d}} \left(1 - \frac{a}{2d}\right) \tag{8.2.6-1}$$

where:

d—vertical distance from the center of the anchor force to the upper edge of the blister.

2 Design spalling force at the end surface of the blister

$$T_{\mathrm{s,d}} = 0.04 P_{\mathrm{d}} \tag{8.2.6-2}$$

3 Design tensioning force behind the anchorage

$$T_{\mathrm{tb,d}} = 0.2 P_{\mathrm{d}} \tag{8.2.6-3}$$

4 Design edge tension force due to local bending

$$T_{\mathrm{et,d}} = \frac{(2e - d)^2}{12e(e + d)} P_{\mathrm{d}} \tag{8.2.6-4}$$

where:

e—distance from the anchor force to the center of the wall plate.

5 Design tension force due to the action of radial force

$$T_{\mathrm{R,d}} = P_{\mathrm{d}} \alpha \tag{8.2.6-5}$$

where:

α—deviation angle due to sharp change of prestressing tendon profile (rad).

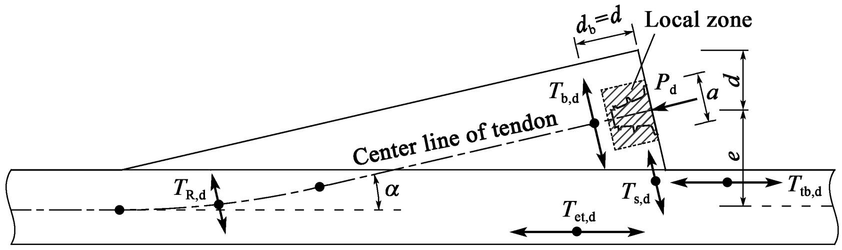


Figure 8.2.6 Tension effects of anchor blister in a post-tensioned member

8.3 Diaphragms at Supports

8.3.1 Mechanical behaviors of diaphragms at supports in the transverse direction of a bridge shall be analyzed. Isolated simplified models may be used with the assumption that loads within the span are transmitted from webs to diaphragms. The diaphragm with the span-to-depth ratio $B_w/h > 2$ may be analyzed in accordance with the provisions for reinforced concrete flexural members in Chapters 5 ~ 7 of the *Specifications*. The diaphragm with the span-to-depth ratio $B_w/h \leqslant 2$ may be analyzed as a disturbed region, where B_w is the distance between the center line of webs at the location of the diaphragm, and h is the depth of the diaphragm.

8.3.2 For diaphragms with the span-to-depth ratio $0.5 \leqslant B_w/h \leqslant 2$ in single-cell box beams, the transverse tensile resistance at the top of diaphragms may be calculated by the following provisions:

$$\gamma_0 T_{t,d} \leqslant f_{sd} A_s + f_{pd} A_p \tag{8.3.2-1}$$

$$T_{t,d} = [0.20 + (B_w/h - 0.5)(0.87 - s/B_w)] \cdot V_d \tag{8.3.2-2}$$

where:

$T_{t,d}$—design internal force in transverse ties at the top of the diaphragm, see Figure 8.3.2;

V_d—design vertical shear force transmitted from one web to the diaphragm. For the case of double supports, V_d is taken to be the design reaction force on a single support R_d; for the case of single support, V_d is taken to be half the design reaction force, i.e., $V_d = R_d/2$;

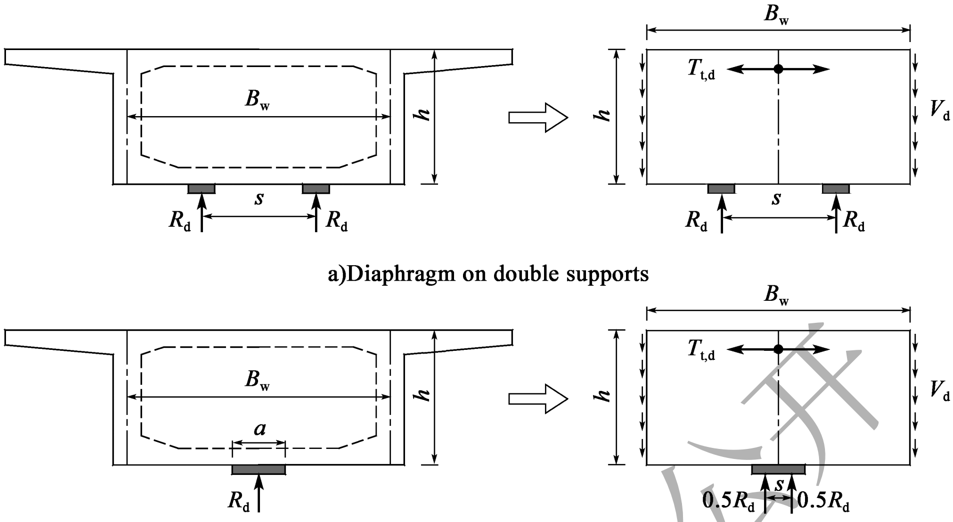
s—for diaphragms on double supports, s is the center-to-center spacing of the two supports; for diaphragms on single support, s is half of the width of the bearing plate of support, i.e., $s = 1/2\ a$;

h—depth of diaphragm, which is taken as the depth of the box beam at the support;

B_w—for vertical webs, B_w is the distance between the center lines of the webs; for inclined webs, B_w is the distance between the midpoints of the center lines of the webs;

f_{sd}, f_{pd}—design tensile strength of reinforcing steel and prestressing steel, respectively;

A_s, A_p—area of reinforcing steel and prestressing steel in the tie, respectively. In the longitudinal direction of the bridge, it shall include the reinforcement within the diaphragm and its each side extent with a length of the diaphragm thickness; in the direction of girder depth, it shall include the reinforcement within the thickness of top slab of the box beam.



b)Diaphragm on a single support

Figure 8.3.2 Transverse tensile force at the top of the diaphragm on supports

8.4 Cap Beams on Piers and Abutments

8.4.1 Cap beams and columns of piers or abutments should be calculated as rigid frames. The center-to-center distance between supports should be taken as the effective span of the cap beams.

8.4.2 Structural design for cap beams shall comply with the following provisions:

1 Cap beams with a span-to-depth ratio of $l/h > 5.0$ at its mid-span portion shall be analyzed in accordance with the provisions for general reinforced concrete members in Chapters 5 ~ 7 of the *Specifications*. Cap beams with a span-to-depth ratio of $2.5 < l/h \leqslant 5.0$ at its mid-span portion shall be checked for their resistance in accordance with Clauses 8.4.3 ~ 8.4.5 of the *Specifications*. Herein, l is the effective span of the cap beams, and h is the depth of the cap beams.

2 For the cantilever portions of cap beams (pier caps), the resistance shall be checked in accordance with Clauses 8.4.6 and 8.4.7 of the *Specifications*.

8.4.3 Flexural resistance on cross-section for a reinforced concrete cap beam shall satisfy the requirements in the following formulas:

$$\gamma_0 M_d \leqslant f_{sd} A_s z \tag{8.4.3-1}$$

$$z = \left(0.75 + 0.05\frac{l}{h}\right)(h_0 - 0.5x) \tag{8.4.3-2}$$

where:

M_d—maximum design bending moment of the cap beam;

f_{sd}—design tensile strength of longitudinal reinforcing steel;

A_s—cross-sectional area of reinforcing steel in the tension zone;

z—internal lever arm;

x—depth of compression zone in cross-section, calculated by Eq. (5.2.2-2);

h_0—effective depth of the cross-section.

8.4.4 Dimensions of sections resisting shear in a reinforced concrete cap beam shall satisfy the following requirement:

$$\gamma_0 V_d \leqslant 0.33 \times 10^{-4}\left(\frac{l}{h}+10.3\right)\sqrt{f_{cu,k}}\,bh_0 \tag{8.4.4}$$

where:

V_d—design shear force acting on the design section (kN);

b—cross-sectional width of the cap beam (mm);

h_0—effective depth of the cap beam (mm);

$f_{cu,k}$—characteristic compressive strength of concrete cube (MPa).

8.4.5 Shear resistance on the inclined section of a reinforced concrete cap beam shall satisfy the following requirements:

$$\gamma_0 V_d \leqslant 0.5 \times 10^{-4}\alpha_1\left(14-\frac{l}{h}\right)bh_0\sqrt{(2+0.6P)\sqrt{f_{cu,k}}\rho_{sv}f_{sv}} \tag{8.4.5}$$

where:

V_d—design shear force at the investigated section (kN);

α_1—coefficient considering the influence of the opposite signs of the bending moment in a continuous beam. In calculating shear resistance of beam portions near end supports, $\alpha_1 = 1.0$; In calculating for beam portions at interior supports and each node of rigid frames, $\alpha_1 = 0.9$;

P—percentage of longitudinal tension reinforcement in tension zone, $P = 100\rho$, $\rho = A_s/(bh_0)$; $P = 2.5$ is taken when $P > 2.5$;

ρ_{sv}—reinforcement ratio for stirrup, $\rho_{sv} = A_{sv}/bs_v$, herein, A_{sv} is the total cross-sectional area of all legs of stirrup in the same cross-section, s_v is the stirrup spacing; the reinforcement ratio for stirrup shall comply with the provisions in Clause 9.3.12 of the *Specifications*;

f_{sv}—tensile strength of stirrup (MPa);

b—width of section of the cap beam (mm);

h_0—effective depth of section of the cap beam (mm).

8.4.6 For the overhang portions of reinforced concrete cap beams subjected to vertical forces, they shall be designed to conform to the following provisions:

1 When the horizontal distance from the vertical force to the edge of the column (circular

column may be transformed into square column with a side length equal to 0.8 times the diameter) is larger than the section depth of the cap beam, the analysis shall be performed in accordance with the provisions for general reinforced concrete members in Chapters 5 ~ 7 of the *Specifications*.

2 When the horizontal distance from the vertical force to the edge of the column is smaller than or equal to the section depth of the cap beam, a strut-and-tie model may be used to calculate the tensile resistance of the ties at the upper edge of the overhang (Figure 8.4.6) in accordance with the following provisions:

$$\gamma_0 T_{t,d} \leq f_{sd} A_s + f_{pd} A_p \qquad (8.4.6\text{-}1)$$

$$T_{t,d} = \frac{x + b_c/2}{z} F_d \qquad (8.4.6\text{-}2)$$

where:

$T_{t,d}$—design internal force in the ties at the upper edge of the overhang of the cap beam;

f_{sd}, f_{pd}—tensile strength of reinforcing steel and prestressing steel, respectively;

A_s, A_p—cross-sectional area of reinforcing steel and prestressing steel in the tie, respectively;

F_d—vertical force on the overhang of the cap beam, which is taken as the fundamental combination of actions;

b_c—supporting width of the column, which is taken as the side length of the section for square columns, and 0.8 times the diameter for circular columns;

x—horizontal distance from the vertical force to the edge of the column;

z—internal lever arm in the cap beam, which may be taken as $z = 0.9h_0$;

h_0—effective depth of section of the cap beam.

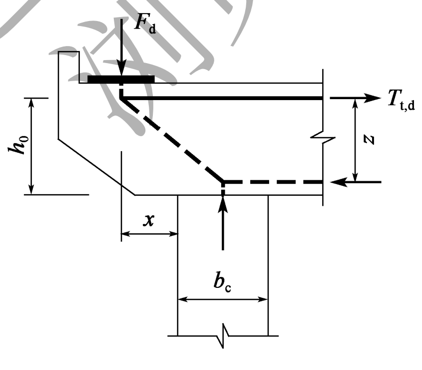


Figure 8.4.6 Strut-and-tie model for a short overhang of a cap beam

8.4.7 For caps or tops of single-column piers with double bearings, the tensile resistance in the transverse tensile section at their tops (Figure 8.4.7) may be calculated by strut-and-tie models in accordance with the following provisions:

$$\gamma_0 T_{t,d} \leq f_{sd} A_s \qquad (8.4.7\text{-}1)$$

$$T_{t,d} = 0.45 F_d \left(\frac{2s - b'}{h} \right) \qquad (8.4.7\text{-}2)$$

where:

$T_{t,d}$—internal force in the transverse tie at the pier cap or top;

F_d—design vertical force on the pier cap or top, for which the fundamental combination of actions is adopted;

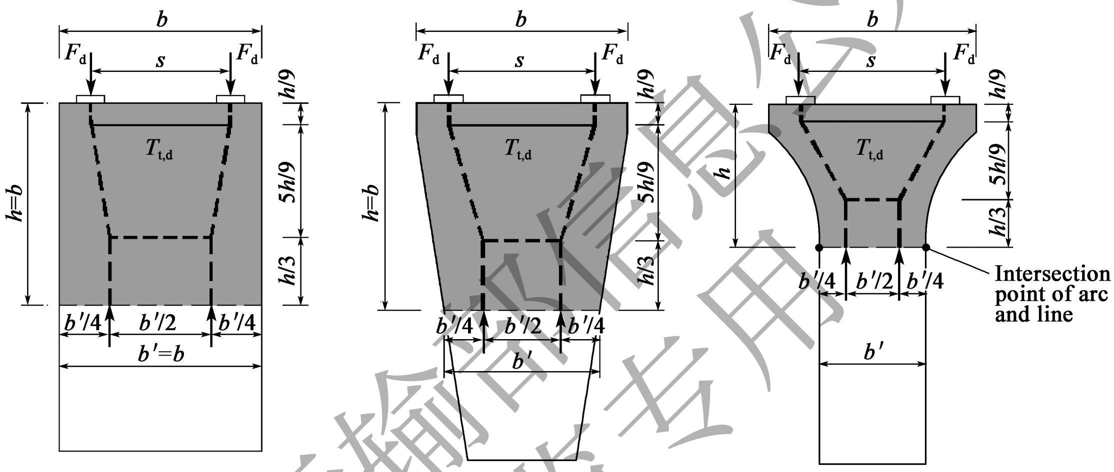
s—center-to-center distance between double bearings;

h—depth of pier cap section with variable transverse width, $h = b$ is taken when $h > b$, in which b is the transverse width on the top of pier cap;

b'—transverse width of pier cap or pier body at the height of h away from the pier top;

f_{sd}—design tensile strength of reinforcing steel;

A_s—cross-sectional area of reinforcing steel in the tie, which is calculated based on the reinforcing steel at the top of the cap beam in the range of $2h/9$ depth.



a)Rectangle b)Inverted trapezoid c)Vase shape

Figure 8.4.7 Strut-and-tie models for reinforcement design for three types of caps of single-column piers (tops)

8.4.8 When the central span portion has a span-to-depth ratio of $2.5 < l/h \leqslant 5.0$ in a reinforced concrete cap beam, its maximum crack width shall be calculated by the formula in Clause 6.4.3 of the *Specifications*, in which the C_3 coefficient is taken as $\frac{1}{3}\left(\frac{0.4l}{h}+1\right)$, and shall not exceed the limits in Clause 6.4.2 of the *Specifications*.

8.4.9 When the central span portion has a span-to-depth ratio of $l/h > 5.0$ in a reinforced concrete cap beam, its deflection should be checked in accordance with the provisions in Section 6.5 of the *Specifications*.

8.5 Pile Caps

8.5.1 In the analysis of pile caps, the vertical force of a single pile acting on the bottom of the pile cap may be calculated by the following formula (Figure 8.5.1):

$$N_{id} = \frac{F_d}{n} \pm \frac{M_{xd} y_i}{\sum y_i^2} \pm \frac{M_{yd} x_i}{\sum x_i^2} \tag{8.5.1}$$

where:

N_{id}—vertical force of the ith pile acting on the bottom of pile cap;

F_d—vertical force due to the combination of actions above the bottom of the pile cap;

M_{xd}, M_{yd}—bending moment due to the combination of actions above the bottom of the pile cap about the x-axis and y-axis through the centroid of the pile group, respectively;

n—total number of piles under the pile cap;

x_i, y_i—distance from the center of the ith row of piles to the y-axis and x-axis, respectively.

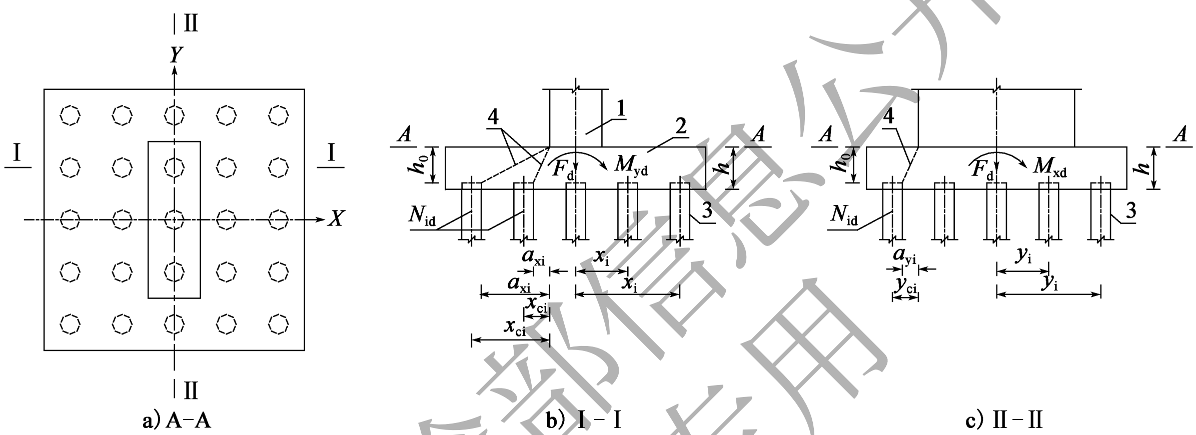


Figure 8.5.1 Analysis of pile caps

1-Pier; 2-Pile cap; 3-Pile; 4-Shear failure location

8.5.2 If the distance from the center of the exterior row of piles under the pile cap to the edge of the pier or abutment is larger than the depth of the pile cap, the flexural esistance on the cross-section (perpendicular to the x-axis and y-axis) of the pile cap may be calculated as for a cantilever beam in accordance with the provisions in Section 5.2 of the *Specifications*.

1 Effective width of the section of pile cap

1) When the center-to-center spacing of piles is not larger than 3 times the side length of the pile or 3 times the diameter of the pile, it is taken as the full width of the pile cap;

2) When the center-to-center spacing of piles is larger than 3 times the side length of the pile or 3 times the diameter of the pile, it is calculated by:

$$b_s = 2a + 3D(n-1) \tag{8.5.2-1}$$

where:

b_s—effective width of the section of pile cap;

a—distance from the center of the side pile parallel to the calculated section and the edge of the pile cap;

D—side length or diameter of the pile;

n—number of piles parallel to the calculated section.

2 Design bending moment at the pile cap section under investigation shall be calculated by the following formulas (Figure 8.5.1):

$$M_{xcd} = \sum N_{id} y_{ci} \qquad (8.5.2\text{-}1)$$

$$M_{ycd} = \sum N_{id} x_{ci} \qquad (8.5.2\text{-}1)$$

where:

M_{xd}, M_{yd}—design bending moment at the investigated section about the x-axis and y-axis due to the combination of vertical forces in each row of piles outside the investigated section;

N_{id}—design vertical force in the ith row of piles outside the investigated section, which is taken as the maximum design vertical force in one pile of the row multiplied by the number of piles in the row;

x_{ci}, y_{ci}—distance perpendicular to the y-axis and x-axis, from the center of the ith row of piles to the investigated section, respectively.

8.5.3 If the distance from the center of the exterior row of piles under the pile cap to the edge of the pier or abutment is larger than the depth of the pile cap, the shear resistance on the inclined section of the pile cap shall be checked in accordance with the following provisions (Figure 8.5.1):

$$\gamma_0 V_d \leqslant (0.9 \times 10^{-4}) \frac{(2 + 0.6P)\sqrt{f_{cu,k}}}{m} b_s h_0 \qquad (8.5.3)$$

where:

V_d—sum of the maximum design shear forces in all rows of the piles outside the calculated inclined section, which are resulted from the the design vertical force in the piles under the overhang section of the pile caps (kN). The design vertical force in each row of piles is obtained from the product of the maximum force among all the piles and the number of piles in the same row;

$f_{cu,k}$—characteristic compressive strength of 150 mm concrete cubes (MPa);

P—percentage of longitudinal tension reinforcement within the inclined section. $P = 100\rho$, $\rho = A_s/(bh_0)$; $P = 2.5$ is taken when $P > 2.5$. Here, A_s is the cross-sectional area of longitudinal tension reinforcement within the effective width of the pile cap section;

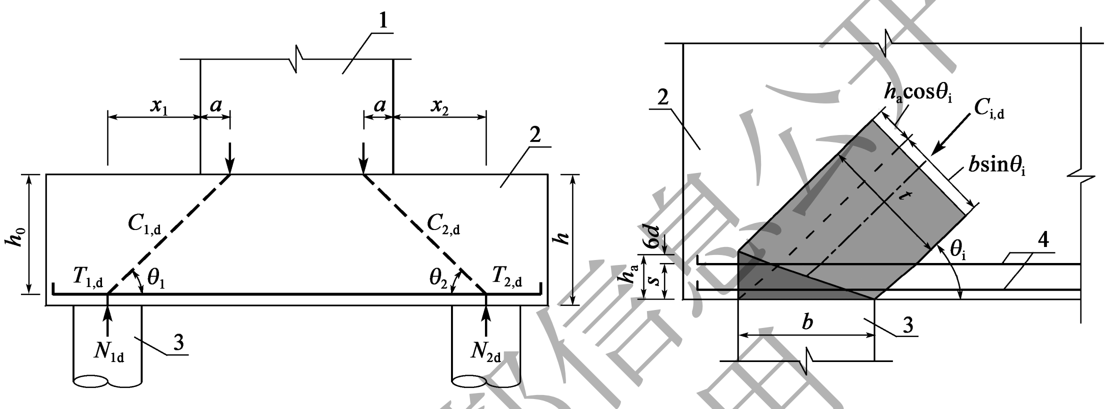
m—shear span to depth ratio, $m = a_{xi}/h_0$ or $m = a_{yi}/h_0$, in which $m = 0.5$ is taken when $m < 0.5$; a_{xi} and a_{yi} are the distance along the x-axis and y-axis from the edge of pier or abutment to the edge of the ith row of piles outside the calculated section, respectively. A circular pile may be converted to a square pile with a side length equal to 0.8 times the diameter of the circular pile;

b_s—effective width of pile cap (mm), refers to the provision in Clause 8.5.2 of the *Specifications* on the effective width in the calculation of shear resistance on cross-section;

h_0—effective depth of pile cap (mm).

If several inclined sections may be considered as potential failure surfaces in the same direction of the pile cap, the shear resistance shall be calculated for each inclined section.

8.5.4 If the distance from the center of the exterior row of the piles under the pile cap to the edge of the pier or the abutment is less than or equal to the depth of the pile cap, the resistance of the pile cap may be computed by the strut-and-tie model given in Appendix B of the *Specifications* (Figure 8.5.4).



a)Strut-and-tie model b)Effective depth of the strut

Figure 8.5.4 Analysis of pile cap by strut-and-tie model

1-Pier or abutment; 2-Pile cap; 3-Pile; 4-Reinforcement in the tie

1 The resistance of the diagonal strut may comply with the following provision:

$$\gamma_0 C_{i,d} \leqslant t b_s f_{ce,d} \tag{8.5.4-1}$$

$$f_{ce,d} = \frac{\beta_c f_{cd}}{0.8 + 170\varepsilon_1} \leqslant 0.85\beta_c f_{cd} \tag{8.5.4-2}$$

$$\varepsilon_1 = \frac{T_{i,d}}{A_s E_s} + \left(\frac{T_{i,d}}{A_s E_s} + 0.002\right)\cot^2\theta_i \tag{8.5.4-3}$$

$$t = b\sin\theta_i + h_a\cos\theta_i \tag{8.5.4-4}$$

$$h_a = s + 6d \tag{8.5.4-5}$$

where:

$C_{i,d}$—design internal force in the strut, including $C_{1,d} = N_{1d}/\sin\theta_1$ and $C_{2,d} = N_{2d}/\sin\theta_2$, in which N_{1d} and N_{2d} are the numbers of piles in the "1st" and "2nd" rows of piles under the overhang sections of pile cap multiplied by the maximum design vertical force of a single pile among the piles in the same row, respectively, where the design vertical force is calculated by Eq. (8.5.1). When calculating the resistance of struts by Eq. (8.5.4-1), $C_{i,d}$ is taken as the larger value between $C_{1,d}$ and $C_{2,d}$;

θ_i—angle between diagonal strut and tie, $\theta_1 = \tan^{-1}\dfrac{h_0}{a + x_1}$, $\theta_2 = \tan^{-1}\dfrac{h_0}{a + x_2}$, in which h_0 is the effective depth of the pile cap; a is the distance from the intersection of the center line of the strut and the top surface of the pile cap to the edge of the pier or abutment, and $a = 0.15h_0$ is adopted; x_1 and x_2 are the distances from the centers of piles to each edge of pier or abutment;

$f_{ce,d}$—design equivalent compressive strength of concrete strut, the parameter β_c in the calculation is taken from Appendix B of the *Specifications*;

t—effective depth of the strut;

b_s—effective width of the strut, which is taken in accordance with the provision in Clause 8.5.2 of the *Specifications* on effective width for the analysis of the flexural resistance of the cross-section;

b—effective width of the supporting surface of the pile, which is taken as the side length for a square section or 0.8 times the diameter for a circular section;

A_s—cross-sectional area of reinforcement in the tie within the effective width of the strut b_s (effective width of tie);

s—distance from the center of the top reinforcement in the tie to the bottom of the pile cap;

d—diameter of reinforcement in the tie. When reinforcements of different diameters are used, d is taken as the weighted average value.

2 The resistance of ties may comply with the following provision:

$$\gamma_0 T_{i,d} \leqslant f_{sd} A_s \tag{8.5.4-6}$$

where:

$T_{i,d}$—design internal force in the tie, which is taken as the large one between $T_{1,d}$ and $T_{2,d}$, where $T_{1,d} = N_{1d}/\tan\theta_1$ and $T_{2,d} = N_{2d}/\tan\theta_2$;

f_{sd}—design tensile strength of reinforcement in the tie;

A_s—cross-sectional area of reinforcement in the tie within the effective width b_s of the strut (effective width of the tie).

Within the full width of the pile cap perpendicular to the tie, the reinforcement in the tie shall be arranged in accordance with Item 2 in Clause 9.6.10 of the *Specifications*. The tension reinforcement ratio within the effective width b_s of the tie shall not be smaller than 0.15%.

8.5.5 Punching shear resistance shall be checked for a pile cap in accordance with the following provisions:

1 The truncated pyramid of failure due to the downward punching shear of the column, pier or abutment shall be formed by the connection lines from the edge of the column, pier or abutment to the corresponding top edge of the pile; the pile top is located at one effective depth h_0 beneath

the top surface of the pile cap. The angle between the inclined plane of the truncated pyramid and the horizontal plane shall not be smaller than 45°. If it is smaller than 45°, 45° is taken.

Punching shear resistance of a pile cap due to the downward punching shear of the column, pier or abutment shall comply with the following provisions:

$$\gamma_0 F_{ld} \leq 0.6 f_{td} h_0 [2\alpha_{px}(b_y + a_y) + 2\alpha_{py}(b_x + a_x)] \tag{8.5.5-1}$$

$$\alpha_{px} = \frac{1.2}{\lambda_x + 0.2} \tag{8.5.5-2}$$

$$\alpha_{py} = \frac{1.2}{\lambda_y + 0.2} \tag{8.5.5-3}$$

where:

F_{ld}—design punching shear force acting on the truncated pyramid, which may be taken as the design vertical force in the column, pier or abutment minus the design reaction of piles within the range of the pyramid;

b_x, b_y—side lengths of the loaded area of the column, pier or abutment;

a_x, a_y—punching shear span, which is the horizontal distance between the top edge and the bottom edge of the truncated pyramid surface, i.e., the horizontal distance from the edge of the column, pier or abutment to the edge of the pile, and shall not be larger than h_0 [Figure 8.5.5a)];

λ_x, λ_y—punching shear span-to-depth ratio, $\lambda_x = a_x/h_0$, $\lambda_y = a_y/h_0$. When $a_x < 0.2h_0$ or $a_y < 0.2h_0$, $a_x = 0.2h_0$ or $a_y = 0.2h_0$ is taken;

α_{px}, α_{py}—coefficient for punching shear resistance corresponding to the punching shear span-to-depth ratio λ_x and λ_y, respectively;

f_{td}—design tensile strength of concrete.

2 For corner and edge piles beyond the truncated pyramid of failure due to the downward punching shear of the column, pier or abutment, the resistance of a pile cap to the upward punching shear shall comply with the following provisions:

1) For corner piles,

$$\gamma_0 F_{ld} \leq 0.6 f_{td} h_0 \left[2\alpha'_{px}\left(b_y + \frac{a_y}{2}\right) + 2\alpha'_{py}\left(b_x + \frac{a_x}{2}\right)\right] \tag{8.5.5-4}$$

$$\alpha'_{px} = \frac{0.8}{\lambda_x + 0.2} \tag{8.5.5-5}$$

$$\alpha'_{py} = \frac{0.8}{\lambda_y + 0.2} \tag{8.5.5-6}$$

where:

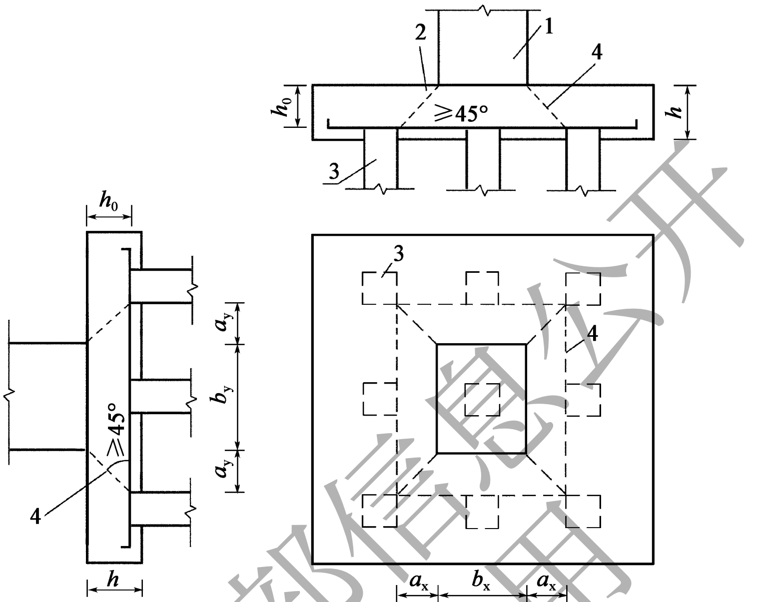
F_{ld}—design vertical force in the corner pile;

b_x, b_y—horizontal distance from the edge of the pile cap to the inner edge of the pile;

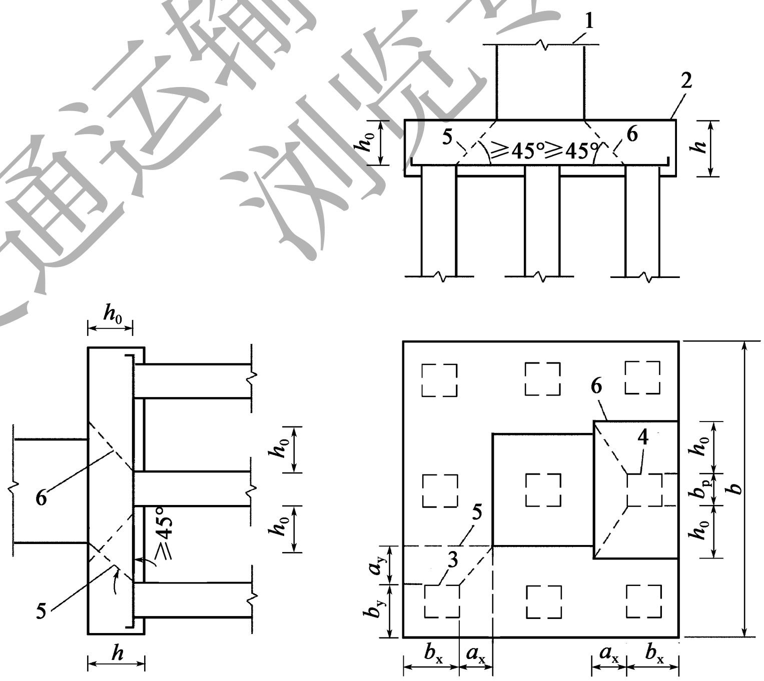
a_x, a_y—punching shear span, which is the horizontal distance from the edge of the pile to the edge of the column, pier or abutment, and shall not be larger than h_0[Figure 8.5.5b)];

λ_x, λ_y—punching shear span-to-depth ratio, $\lambda_x = a_x/h_0$, $\lambda_y = a_y/h_0$. When $a_x < 0.2h_0$ or $a_y < 0.2h_0$, $a_x = 0.2h_0$ or $a_y = 0.2h_0$ is taken;

α'_{px}, α'_{py}—coefficient for punching shear resistance corresponding to the punching shear span-to-depth ratio λ_x and λ_y, respectively.



a)Truncated pyramid due to downward punching shear of column, pier or abutment
1-Column, pier or abutment; 2-Pile cap; 3-Pile; 4-Truncated pyramid



b)Truncated pyramid due to upward punching shear of corner pile and edge pile
1-Column, pier or abutment; 2-Pile cap; 3-Corner pile; 4-Cdge pile; 5-Truncated pyramid due to upward punching shear of corner pile; 6-Truncated pyramid due to upward punching shear of edge pile

Figure 8.5.5 Truncated pyramids due to punching shear on pile caps

2) For edge piles, when $b_p + 2h_0 \leqslant b$ [for b see Figure 8.5.5b)],

$$\gamma_0 F_{ld} \leqslant 0.6 f_{td} h_0 [\alpha'_{px}(b_p + h_0) + 0.667 \cdot (2b_x + a_x)] \tag{8.5.5-7}$$

where:

F_{ld}—design vertical force in the edge pile;

b_x—horizontal distance from the edge of the pile cap to the inner edge of the pile;

b_p—side length of the square pile;

a_x—punching shear span, which is the horizontal distance from the edge of the pile to the edge of the corresponding column, pier or abutment, and shall not be larger than h_0.

In the above calculation, circular piles may be converted to square piles with the side length equal to 0.8 times the diameters of the circular piles.

Notes: For a pile cap with variable depth h_0 in Eq. (8.5.5-1) is taken as the effective depth for the section of the pile cap perpendicular to the edge of the column, pier or abutment; in Eqs. (8.5.5-4) and (8.5.5-7), h_0 is taken as the effective depth of the section at the edge of the pile cap.

8.5.6 In pile caps, the parts subjected to local loads shall be checked for their local compression resistance in accordance with Section 5.7 of the *Specifications*.

8.6 Hinges

8.6.1 For cylindrical hinges in line contact, the compressive resistance of the compressed surface should comply with the following provision:

$$\gamma_0 F_{hd} \leqslant \frac{7.14\,(\eta_s \beta f_{cd})^2 l}{E_c\left(\dfrac{1}{r_1} - \dfrac{1}{r_2}\right)} \tag{8.6.1-1}$$

$$\beta = \sqrt{\frac{A_b}{bl}} \tag{8.6.1-2}$$

The width b of the area for transmitting pressure (Figure 8.6.1) should be calculated by the following formula:

$$b = 2.74\sqrt{\frac{\gamma_0 F_{hd}}{E_c\left(\dfrac{1}{r_1} - \dfrac{1}{r_2}\right) l}} \tag{8.6.1-3}$$

where:

F_{hd}—compressive force of hinge acting on the compressed surface;

f_{cd}—design compressive strength of concrete;

A_b—calculated bottom area for local compression, determined by Figure 5.7.1;

η_s—correction factor for local compression of concrete, which is taken in accordance with the provision in Clause 5.7.1 of the *Specifications*;

l—length of cylindrical hinge;

E_c—modulus of elasticity of concrete;

r_1, r_2—diameters of the upper and lower cylinders, respectively; when the upper cylinder contacts with a plane, $\frac{1}{r_2}=0$ is taken.

γ_0—importance factor of structure.

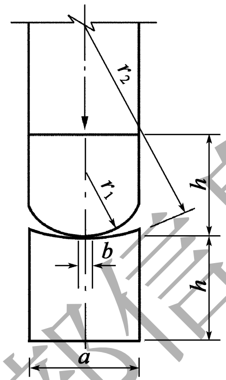


Figure 8.6.1 Concrete hinge

8.6.2 The transverse tensile resistance of hinges should comply with the following provision (Figure 8.6.1):

$$\gamma_0 F_{hd} \leqslant \frac{h}{0.425(a-b)} f_{sd} A_s \tag{8.6.2}$$

where: f_{sd}—design tensile strength of the transverse reinforcement in the hinge;

a—width of the hinge;

h—height of the hinge, which is taken to be 0.80 ~ 1.25 times of a;

b—width of the area for transmitting pressure in the hinge, which is computed by Eq. (8.6.1-3);

A_s—cross-sectional area of the transverse tension reinforcement in the hinge.

In the side direction of the hinge, reinforcement may be arranged by 0.4 times the cross-sectional area of the transverse reinforcement.

8.7 Bearings

8.7.1 Bridge bearings shall conform to the following requirements:

1 Bearings shall have the ability to effectively transmit structural self-weights, vehicular loads and other vertical actions sustained by superstructures to substructures, and ensure the safety of superstructures under the actions of horizontal loads, such as wind load and seismic action, etc.

2 Type and specification of the bearings shall be determined according to the forms of superstructures and substructures, design reaction force and horizontal force of bearings, and displacement of the beams at bearings.

3 Design reaction force R_{ck} at supports shall be calculated with the combination of characteristic vertical loads, in which the impact force shall be considered for the vehicular load.

4 Design horizontal force of bearings shall be calculated with the combination of characteristic horizontal actions.

5 The following factors shall be considered in the calculation for relative displacements of beams at bearings:

1) Displacement caused by temperature change, vehicular braking force, etc;

2) Displacement caused by the deflection angle of the beams;

3) Displacement of beams caused by prestressing force;

4) Displacement caused by shrinkage and creep of concrete;

5) Displacement caused by earthquake and other accidental actions.

8.7.2 Basic design data, product types, technical requirements, test methods, and inspection rules of laminated elastomeric bearings shall comply with the provisions of the current *Laminated Bearings for Highway Bridge* (JT/T 4).

8.7.3 The selection of laminated elastomeric bearings shall comply with the following provisions:

1 The effective compression area shall comply with the following provision:

$$A_e \geqslant \frac{R_{ck}}{\sigma_c} \tag{8.7.3-1}$$

where:

A_e—effective compression area of the bearing (area of steel plate for reinforcing and bearing);

R_{ck}—design reaction force of bearing, in which impact force should be considered for the vehicular load;

σ_c—average compressive stress limit on bearing at the service stage, which is taken in accordance with *Laminated Bearings for Highway Bridge* (JT/T 4).

2 The total elastomer thickness shall comply with the following provisions:

1) To accomodate the shear deformation, the total thickness of the elastomer shall conform to the following conditions:

When braking force is not considered,

$$t_e \geqslant 2\Delta_l \tag{8.7.3-2}$$

When braking force is considered,

$$t_e \geqslant 1.43\Delta_l \tag{8.7.3-3}$$

When laminated elastomeric bearings are arranged in the transverse direction of a bridge and parallel to the cross slopes of bed blocks or cap beams on piers or abutments, the total elastomer thickness of the bearings shall satisfy the following conditions:

When braking force is considered,

$$t_e \geqslant 2\sqrt{\Delta_l^2 + \Delta_t^2} \tag{8.7.3-4}$$

When braking force is not considered,

$$t_e \geqslant 1.43\sqrt{\Delta_l^2 + \Delta_t^2} \tag{8.7.3-5}$$

where:

t_e—total elastomer thickness of the bearing;

Δ_l—sum of the shear deformation induced by characteristic temperature change, shrinkage and creep of concrete and other actions, the shear deformation caused by characteristic longitudinal force (including characteristic braking force), and the shear deformation induced by the longitudinal component of design reaction force of bearings acting on the top surface of the bearings when the bearings are directly placed beneath the bottom of the cap beam with a profile grade not larger than 1%;

Δ_t—shear deformation caused by the component of the design reaction force of the bearing, in which the component is parallel to the cross slope of the bed block or cap beam on the pier or abutment, where the bearings are set parallel to the transverse direction of the bridge and the cross slope is not larger than 2%.

2) To guarantee the stability of the laminated elastomeric bearings in compression, the following conditions shall be conformed to:

For rectangular bearings,

$$\frac{l_a}{10} \leq t_e \leq \frac{l_a}{5} \tag{8.7.3-6}$$

For circular bearings,

$$\frac{d}{10} \leq t_e \leq \frac{d}{5} \tag{8.7.3-7}$$

where:

l_a—short side length of the rectangular bearing;

d—diameter of the circular bearing.

3 Average vertical compressive deformation of a laminated elastomeric bearing shall comply with the following provisions:

$$\delta_{c,m} = \frac{R_{ck} t_e}{A_e E_e} + \frac{R_{ck} t_e}{A_e E_b} \tag{8.7.3-8}$$

$$\theta \frac{l_a}{2} \leq \delta_{c,m} \leq 0.07 t_e \tag{8.7.3-9}$$

where:

$\delta_{c,m}$—average vertical compressive deformation of the bearing;

E_e—compressive modulus of elasticity of the bearing, which is taken in accordance with the current *Laminated Bearings for Highway Bridge* (JT/T 4);

E_b—bulk modulus of elastomer, which is taken in accordance with the current *Laminated Bearings for Highway Bridge* (JT/T 4);

L_a—short side length of the rectangular bearing or the diameter of the circular bearing;

θ—angle of inclination on the top surface of the bearing caused by the deflection difference of superstructure, or angle of profile grade on the top surface of the bearing when the bearing is directly placed beneath the beam soffit with a longitudinal slope of not more than 1% (rad).

4 Reinforcing steel plates in laminated elastomeric bearings shall comply with the following provisions, and their minimum thickness shall not be less than 2 mm.

$$t_s \geq \frac{K_p R_{ck} (t_{es,u} + t_{es,l})}{A_e \sigma_s} \tag{8.7.3-10}$$

where:

t_s—thickness of reinforcing steel plate in the bearing;

K_p—correction factor for stress, which is taken as 1.3;

$t_{es,u}$, $t_{es,l}$—thickness of elastomer on and beneath reinforcing steel plate, respectively;

σ_s—axial tensile stress limit of reinforcing steel plate, which may be taken as 0.65 times the yield strength of steel material.

The minimum distance between the edges of the reinforcing steel plate and the bearing shall not be less than 5 mm, and the top and bottom cover layers shall not be thinner than 2.5 mm.

8.7.4 The stability against sliding for laminated elastomeric bearings shall comply with the following provisions:

When vehicular braking force is not considered,

$$\mu R_{\mathrm{Gk}} \geqslant 1.4 G_{\mathrm{e}} A_{\mathrm{g}} \frac{\Delta_l}{t_{\mathrm{e}}} \tag{8.7.4-1}$$

When vehicular braking force is considered,

$$\mu R_{\mathrm{ck}} \geqslant 1.4 G_{\mathrm{e}} A_{\mathrm{g}} \frac{\Delta_l}{t_{\mathrm{e}}} + F_{\mathrm{bk}} \tag{8.7.4-2}$$

where:

R_{Gk}—reaction force of bearing due to structural self-weight;

R_{ck}—reaction force of bearing due to characteristic structural self-weight and 0.5 times characteristic vehicular load (including impact force);

μ—coefficient of friction between bearing and its contact surface, which is taken in accordance with the current *Laminated Bearings for Highway Bridge* (JT/T 4);

G_{e}—shear modulus of bearing, which is taken in accordance with the current *Laminated Bearings for Highway Bridge* (JT/T 4);

Δ_l—see Clause 8.7.3, in which the shear deformation caused by vehicular braking force is not included;

F_{bk}—characteristic braking force caused by the vehicular load;

A_{g}—gross area of bearing surface.

8.7.5 The coefficient of friction for Polytetrafluoroethylene (PTFE or Teflon) slide bearings shall comply with the following provisions:

When vehicular braking force is not considered,

$$\mu_{\mathrm{f}} R_{\mathrm{Gk}} \leqslant G_{\mathrm{e}} A_{\mathrm{g}} \tan\alpha \tag{8.7.5-1}$$

When vehicular braking force is considered,

$$\mu_{\mathrm{f}} R_{\mathrm{ck}} \leqslant G_{\mathrm{e}} A_{\mathrm{g}} \tan\alpha \tag{8.7.4-2}$$

where:

μ_{f}—coefficient of friction between PTFE and stainless plate, which is taken in accordance with the current *Laminated Bearings for Highway Bridge* (JT/T 4);

$\tan\alpha$—limit value for the tangent of shear angle of bearing, which is taken in accordance with the current *Laminated Bearings for Highway Bridge* (JT/T 4);

R_{ck}—reaction force of bearing due to characteristic structural self-weight and vehicular load (including impact force);

A_{g}—gross area of bearing surface.

8.7.6 Pot rubber bearings and spherical bearings with production approval shall conform to the technical requirements in *Pot Bearings for Highway Bridge* (JT/T 391) and *Spherical Bearings for Bridges* (GB/T 17955), respectively.

8.8 Deck expansion joints

8.8.1 Deck expansion joints (expansion joint in short) shall satisfy the following requirements:

1 Technical requirements for materials and finished products of expansion joints shall comply with relevant provisions in *General Technical Requirements of Expansion and Contraction Installation for Highway Bridge* (JT/T 327-2016).

2 When various types of expansion joints with production approval are used, their installation temperature may be selected according to the local temperature condition at bridge sites and construction season. The type specification and category of the expansion joints are selected based on the expansion and contraction values of the beam at the joint (opening and closing ranges of the joint gap) calculated in accordance with Clause 8.8.2 of the *Specifications*.

If expansion joints are designed by bridge designers, fatigue caused by impact actions and repetitive actions shall be considered for steel components subjected to vehicular loads.

3 According to the installation widths of expansion joints, the local structures around the joints shall be drawn down in details, including the dimensions of the openings (depth, top and bottom widths) required for the installation of the expansion joints, the embedded parts and their locations required for the connection of the expansion joints. Meanwhile, the following contents shall be shown on the drawings:

1) Category and strength class of materials for filling into the openings;

2) Temperature range for installation of the expansion joints. For expansion joints installed within the range, their normal operation can be guaranteed after installation;

3) Type and category of the expansion joints, and their maximum and minimum working widths (B_{max} and B_{min});

4) Installation width or delivery width of the expansion joints (the width after contraction for mat expansion joints, which may be temporarily fastened in the factory for delivery);

5) Matters need attention during the construction of the expansion joints.

8.8.2 Movement range of an expansion joint after installation may be calculated by considering the following factors:

1 Movement range due to temperature change may be calculated by the following formulas:

Expansion of a beam due to temperature rise Δl_{t}^{+}

$$\Delta l_{t}^{+} = \alpha_{c} l (T_{max} - T_{set,l}) \tag{8.8.2-1}$$

Contraction of a beam due to temperature drop Δl_{t}^{-}

$$\Delta l_{t}^{-} = \alpha_{c} l (T_{set,u} - T_{min}) \tag{8.8.2-2}$$

where:

T_{max}, T_{min}—highest and lowest effective local ambient temperature, respectively, which shall be taken in accordance with *General Specifications for Design of Highway Bridges and Culverts* (JTG D60-2015);

$T_{set,u}$, $T_{set,l}$—upper and lower values of the preset temperature range for installation;

l—length of beam used for the calculation of the movement range of an expansion joint, which is determined depending on the segmentation of the bridge length and the arrangement of bearings;

α_{c}—coefficient of linear thermal expansion of concrete in the beam, $\alpha_{c} = 0.00001$ shall be used.

2 Contraction of a beam due to the shrinkage of concrete, Δl_{s}^{-}, is calculated by Eq. (8.8.2-3):

$$\Delta l_{s}^{-} = \varepsilon_{cs}(t_{u}, t_{0}) l \tag{8.8.2-3}$$

where:

$\varepsilon_{cs}(t_{u}, t_{0})$—ultimate shrinkage strain. It is the shrinkage strain from the concrete age t_{0} of the beam when the installation of expansion joints is completed to the concrete age t_{u} at the end of shrinkage and it may be calculated in accordance with Appendix C in the *Specifications*.

3 Contraction of a beam due to the creep of concrete, Δl_{c}^{-}, is calculated by Eq. (8.8.2-4):

$$\Delta l_{c}^{-} = \frac{\sigma_{pc}}{E_{c}} \varphi(t_{u}, t_{0}) l \tag{8.8.2-4}$$

where:

σ_{pc}—normal compressive stress at the center of gravity of the cross-section due to prestressing force (after subtracting the losses of prestress at the corresponding stages), which may be taken to be the average on the mid-span and quarter-span sections for simply-supported beam; and may be taken to be the average on several typical sections for continuous beams;

E_c—modulus of elasticity of concrete in the beam, taken from Table 3.1.5;

$\varphi(t_u, t_0)$—ultimate creep coefficient. It is the creep coefficient from the concrete age t_0 of the beam when the installation of expansion joints is completed to the concrete age t_u at the end of creep, which may be calculated in accordance with Appendix C in the *Specifications*.

4 Opening range Δl_b^- or closing range Δl_b^+ of the joint gap caused by shear deformation of laminated elastomeric bearings due to braking force may be calculated by Δl_b^- or $\Delta l_b^+ = F_k t_e / G_e A_g$, in which F_k is the characteristic vehicular braking force assigned to bearings, t_e is the total elastomer thickness, G_e is the shear modulus of the elastomers in the bearing (taken from Clause 8.7.4 of the *Specifications*), and A_g is the gross area of the bearing surface.

5 Types of expansion joints shall be adopted according to the movement ranges of the beam.

1) Closing range of an expansion joint after installation C^+

$$C^+ = \beta(\Delta l_t^+ - \Delta l_b^+) \tag{8.8.2-5}$$

2) Opening range of an expansion joint after installation C^-

$$C^- = \beta(\Delta l_t^- + \Delta l_s^- + \Delta l_c^- + \Delta l_b^-) \tag{8.8.2-6}$$

3) Movement range of an expansion joint shall satisfy

$$C \geqslant C^+ + C^- \tag{8.8.2-7}$$

where:

β—amplification factor for movement range of the expansion joint, which may be taken as $\beta = 1.2 \sim 1.4$.

Notes: 1. Other factors that affect the movement ranges of expansion joints, such as seismic action, wind load and deflection of beams, shall be considered as appropriate.

2. When the installation temperature during construction is beyond the installation temperature range specified in the design, the movement range of expansion joints shall be calculated separately.

8.8.3 The opening range C^- and the closing range C^+ determined in accordance with Clause 8.8.2 of the *Specifications* may be used to calculate the installation widths (or delivery widths) of the expansion joint. The value may be taken to be either $[B_{min} + (C - C^-)]$ or $(B_{min} + C^+)$ or between these two values, in which C is the movement range of the selected expansion joint, and B_{min} is the minimum working width of the selected expansion joint.

9 Detailing Requirements

9.1 General

9.1.1 The thickness of concrete cover over reinforcing and prestressing steel shall satisfy the following requirements:

1 The thickness of concrete cover over the reinforcing steel is taken as the distance from the outer surface of the reinforcement to the surface of concrete, and shall not be less than the nominal diameter of the reinforcement. If bundled bars are used for the reinforcement, the thickness of the concrete cover shall not be less than the effective diameter of the bundled bars.

2 The thickness of concrete cover over the prestressing steel in pretensioned members is taken as the distance from the outer surface of reinforcement to the surface of concrete, and shall not be less than the nominal diameter of the reinforcement. The thickness of the concrete cover over the prestressing steel in post-tensioned members is taken as the distance from the outer surface of the tendon duct to the surface of the concrete, and shallnot be less than one-half of the duct diameter.

3 The thickness of the concrete cover over the extreme reinforcement shall not be less than that specified in Table 9.1.1.

Table 9.1.1 Minimum concrete cover c_{min} (mm)

Category of member	Beam, slab, tower, arch, superstructure of culvert		Pier, abutment, substructure of culvert		Pile cap, foundation	
Design life (Year)	100	50, 30	100	50, 30	100	50, 30
Class I-General	20	20	25	20	40	40

continued

Category of member	Beam, slab, tower, arch, superstructure of culvert		Pier, abutment, substructure of culvert		Pile cap, foundation	
Class II-Freeze-thaw	30	25	35	30	45	40
Class III-Coastal or marine chloride	35	30	45	40	65	60
Class IV-Deicing salt and other chloride	30	25	35	30	45	40
Class V-Salt crystallization	30	25	40	35	45	40
Class VI-Chemical corrosive	35	30	40	35	60	55
Class VII-Abrasion	35	30	45	40	65	60

Notes: 1. Values in this table are specified in accordance with the lowest action level of each environment category, the structures of lowest concrete strength class required in Clause 4.5.3 of the *Specifications* as well as of reinforcement and concrete without special protective measures against corrosion.

2. For concrete members cast in factory, the minimum concrete cover may be taken by reducing the corresponding values in this table by 5 mm, but shall not be less than 20 mm.

3. The minimum concrete cover for pile caps and foundations in this table is specified for the situations without bedding at the bottom of foundation pits or formworks on their sides. For the situations with bedding or formworks, the minimum concrete cover may be taken by reducing the corresponding values in this table by 20 mm, but shall not be less than 30 mm.

9.1.2 If the thickness of concrete cover over the longitudinal primary reinforcement is larger than 50 mm, effective measures and detailing should be taken for the concrete cover. If wire fabrics are provided in concrete cover to prevent cracking and spalling, the diameter of reinforcement should not be less than 6 mm, and the spacing should not be larger than 100 mm, and the thickness of concrete cover over the wire fabrics should not be less than 25 mm.

9.1.3 The diameter of a single bar in bundled bars shall not be larger than 36 mm. The number of bars in one bundle shall not exceed three when the diameter of a single bar is not larger than 28 mm, and two when the diameter is larger than 28 mm. The effective diameter of a unit of bundled bars is $d_e = \sqrt{n}d$, in which n is the number of bars in one bundle, and d is the diameter of a single bar.

When the diameter of a single bar or the effective diameter of a unit of bundled bars is larger than 36 mm, wire fabric inside the surface should be provided for bundled bars in tension zones. In the direction along the length of bundled bars, the bar diameter of the wire fabric shall not be less than 10 mm, and their spacing shall not be larger than 100 mm. In the direction perpendicular to the length of bundled bars, the bar diameter of the wire fabric shall not be less than 6 mm, and their spacing shall not be larger than 100 mm. The extent of the arrangement of the above wire fabric shall exceed that of the bundled bars on each side by not less than 5 times the bar diameter or the effective diameter of the bundled bars.

9.1.4 If the strength of reinforcement is fully considered in the analysis, the minimum development length shall comply with the provisions in Table 9. 1. 4.

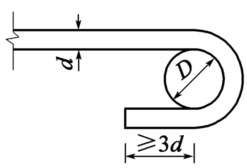
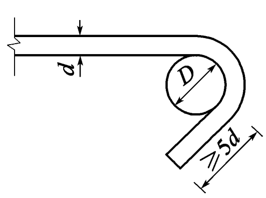
Table 9. 1. 4 Minimum development length of reinforcement l_a

Type of reinforcement		HPB300				HRB400, HRBF400, RRB400			HRB500		
Concrete strength class		C25	C30	C35	≥C40	C30	C35	≥C40	C30	C35	≥C40
Compression reinforcement (straight)		45d	40d	38d	35d	30d	28d	25d	35d	33d	30d
Tension reinforcement	Straight	—	—	—	—	35d	33d	30d	45d	43d	40d
	Hooked	40d	35d	33d	30d	30d	28d	25d	35d	33d	30d

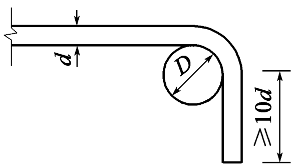
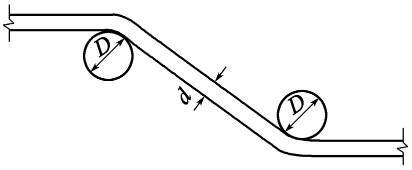
Notes: 1. d is the nominal diameter of reinforcement.
2. For bundled bars in tension with the effective diameter $d_e \leq 28$ mm and bundled bars in compression, the development lengths shall be determined in accordance with the values in the table based on the effective diameter, and individual bars in a bundle may be cut off at the same endpoint of the anchoring. For bundled bars in tension with the effective diameter $d_e > 28$ mm, cut-off for individual bars in a bundle shall be begun from the starting point of the anchoring and terminated at different points with a stagger of 1. 3 times the specified development length of a single reinforcement in the table. Namely, beginning from the starting point of the anchoring, the cut-off extent for the first bar is 1. 3 times the development length of a single reinforcement, 2. 6 times the development length of a single reinforcement for the second bar, and 3. 9 times the development length of a single reinforcement for the third bar.
3. If epoxy-coated reinforcement is used, the minimum development length for tension reinforcement shall be increased by 25%.
4. If the hardening of concrete is liable to be disturbed, the development length shall be increased by 25%.
5. If hooked tension reinforcement is used, the development length is the projected length including the hook.

9. 1. 5 Hooks at the free end of tension reinforcement and intermediate bends of reinforcement shall comply with the provisions in Table 9. 1. 5.

Table 9. 1. 5 End hooks of tension reinforcement and intermediate bends of reinforcement

Location of bend	Bend angle	Shape	Reinforcement	Diameter of bend D	Length of straight extension
Hook at the free end	180°		HPB300	≥2.5d	≥3d
	135°		HRB400 HRB500 HRBF400 RRB400	≥5d	≥5d

continued

Location of bend	Bend angle	Shape	Reinforcement	Diameter of bend D	Length of straight extension
Hook at the free end	90°		HRB400 HRB500 HRBF400 RRB400	$\geqslant 5d$	$\geqslant 10d$
Intermediate bend	$\leqslant$ 90°		All	$\geqslant 20d$	—

Notes: If epoxy-coated reinforcement is used, besides complying with the provisions in this table, the inside diameter of bend D shall not be less than $5d$ for the reinforcement with a diameter $d \leqslant 20$ mm. The inside diameter of bend D shall not be less than $6d$, and the length of straight extension shall not be less than $5d$ for $d > 20$ mm.

9.1.6 Stirrups shall have hooks at the free end, and the bend angle may be 135°. The bend diameter of the hooks shall be larger than the diameter of the primary reinforcement to be hooked, the bend diameter for HPB300 reinforcement shall not be less than 2.5 times the diameter of the stirrups, and the bend diameter for HRB400 reinforcement shall not be less than 5 times the diameter of the stirrups. The length of straight extensions for hooks shall not be less than 5 times the diameter of the stirrups for general structures, and 10 times the diameter of the stirrups for earthquake-resistant structures.

9.1.7 Connection of reinforcement should be placed at the regions subjected to small forces and should be staggered. Welded and mechanical splices (compression coupling sleeves, upset straight thread couplers) should be used. If they are difficult to use in construction or restrained by detailing conditions, lap splices may be used except for the primary reinforcement in axially loaded tension members or tension members with small eccentricities. The diameter of reinforcement for lap splices should not be larger than 28 mm; for the compression reinforcement in axially or eccentrically loaded compression members, it should not be larger than 32 mm.

9.1.8 Welded splices shall conform to the following requirements:

1 Flash butt welding should be employed for welded splices. If there is a lack of conditions for flash butt welding, other type weldings may be employed, including arc welding (welding with assisted bars or lap welding), electro slag welding, and gas pressure

welding, but they shall satisfy the following requirements:

1) Arc welding shall be performed from two sides, unless only welding from one side has to be made. The weld length of welded joints shall not be less than 5 times the diameter of reinforcement for welding from two sides, and 10 times of the diameter of reinforcement for welding from one side.

2) Assisted bars for welding shall have the same strength grade as the reinforcement to be welded, and the total cross-sectional area shall not be less than that of the welded reinforcement.

3) When lap welding is employed, the end of each reinforcement shall be bent to one side in advance, and the axes of the two reinforcements shall be consistent.

2 The center-to-center distance l of welded joints of a reinforcement shall be larger than 35 times the diameter of reinforcement and 500 mm (Figure 9.1.8). Within the range of l, the percentage of the cross-sectional area of the primary reinforcement with welded joints in tension zones to that of all primary reinforcement should not exceed 50%, and the percentage is not limited for the reinforcement with welded joints in compression zones and in prefabricated members.

3 The transverse clear distance of reinforcement at the welded joints with assisted bars or lap welding shall not be less than the diameter of the reinforcement, and shall not be less than 25 mm. Meanwhile, the clear distance of non-welded reinforcement shall still comply with the provisions in Clause 9.3.3.

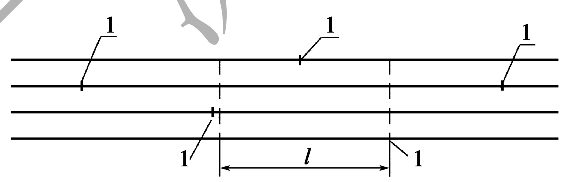


Figure 9.1.8 Layout of welded joints

1-Center of weld joint (The cross-sectional area of reinforcement at the joint within the l section shown in the figure is calculated with two bars)

9.1.9 Lap splices of reinforcement shall conform to the following requirements:

1 The length of lap for tension lap splices shall not be less than the values specified in Table 9.1.9. The length of lap for compression lap splices shall not be less than 0.7 times that for tension lap splices specified in Table 9.1.9.

Table 9.1.9 Length of lap for tension lap splices

Type of reinforcement	HPB300		HRB400, HRBF400, RRB400	HRB500
Concrete strength class	C25	≥C30	≥C30	≥C30
Length of lap (mm)	$40d$	$35d$	$45d$	$50d$

Notes: 1. d is the nominal diameter of reinforcement (mm). For deformed bars with $d > 25$ mm, the length of lap for tension reinforcement shall be taken as the values in this table plus $5d$. For deformed bars with $d < 25$ mm, the length of lap may be taken as the values in this table minus $5d$.

2. If the hardening of concrete is liable to be disturbed, the length of lap shall be increased by $5d$.

3. In any case, the length of lap for tension reinforcement shall not be less than 300 mm; the length of lap for compression reinforcement shall not be less than 200 mm.

4. The length of lap for lap splices of epoxy-coated reinforcement for tension reinforcement is taken as 1.5 times the values in this table.

5. Within tensile sections, the ends of HPB300 reinforcement for lap splice shall be hooked, and the ends of HRB400, HRB500, HRBF400 and RRB400 reinforcement may not be hooked.

2 The center-to-center distance l of welded joints of a reinforcement shall be larger than 1.3 times the length of lap l_s (Figure 9.1.9-1). Within the range of l, the percentage of the cross-sectional area of the primary reinforcement with welded joints in tension zones to that of all primary reinforcement should not exceed 25%, and the percentage for the reinforcement with welded joints in compression zones should not exceed 50%. If the above limits are exceeded, the specified values in Table 9.1.9 shall be multiplied by the following coefficients: 1.4 when the cross-sectional area of tension reinforcement with lap splices is larger than 25% but not larger than 50%, 1.6 when being larger than 50%; 1.4 when the cross-sectional area of compression reinforcement with lap splices is larger than 50% (the length of lap for lap splices of compression reinforcement is still 0.7 times the length of lap for lap splices of tension reinforcement).

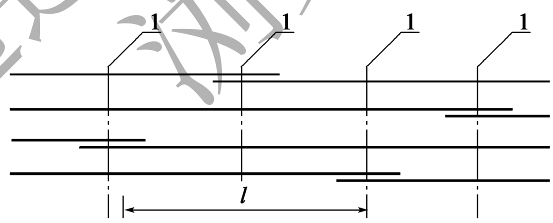


Figure 9.1.9-1 Lap splices for primary reinforcement

1-Center of lap length of lap splices (the cross-sectional area of reinforcement with lap splice within section l is counted as two bars)

3 The transverse clear distance of reinforcement at lap splices shall not be less than the diameter of the reinforcement and shall not be less than 25 mm. Meanwhile, the clear distance of reinforcement beyond the splices shall still comply with the provisions in Clause 9.3.3.

4 For a lap splice of a bundled-bars, all the bars shall be spliced in stagger individually, in which the stagger distance shall be 1.3 times the length of lap of a single bar specified in

Table 9.1.9. Then, an entire bar with a length of $1.3(n+1)l_s$ shall be used for lap splicing of the bundled-bars, where n is the number of individual bars within a bundle, and l_s is the length of lap of an individual bar.

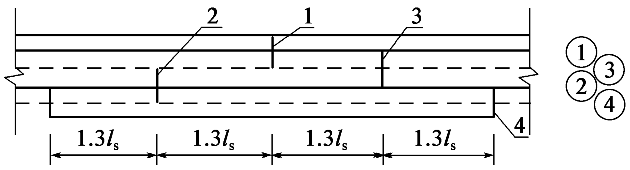


Figure 9.1.9-2 Lap splice of bundled bar

1, 2, 3-Individual bars within the bundle; 4-Entire length bar

9.1.10 Mechanical splices of reinforcement are applicable for the connection between HRB400, HRB500, HRBF400 and RRB400 deformed bars. Mechanical splices shall comply with the provisions in *Technical Specification for Mechanical Splicing of Steel Reinforcing Bars*.

The minimum concrete cover over the mechanical splices of reinforcement should comply with the provisions given for the thickness of concrete cover over the primary reinforcement specified in Table 9.1.1, and shall not be less than 20 mm.

The transverse clear distance between splicing elements or between splicing elements and reinforcement shall not be less than 25 mm. Meanwhile, the clear distance of reinforcement beyond the splices shall still comply with the provisions in Clauses 9.3.3 and 9.6.1 of the *Specifications*.

9.1.11 Cold-swaged coupling sleeves and upset straight-threaded couplers for reinforcement shall comply with relevant provisions in *Specifications for Hydraulic Concrete Construction* (SL 677—2014) and *Couplers for Rebar Mechanical Splicing* (JG/T 163-2013).

9.1.12 The minimum reinforcement percentage for longitudinal primary reinforcement in reinforced concrete members shall conform to the following requirements:

1 The reinforcement percentage for all longitudinal reinforcement in axially or eccentrically loaded compression members shall not be less than 0.5, and it shall not be less than 0.6 when the concrete with a strength class of C50 or higher is used. Meanwhile, the reinforcement percentage on one side shall not be less than 0.2. In tension members loaded with large eccentricities, the reinforcement percentage for the compression reinforcement in the compression zones required according to calculation results shall not be less than 0.2.

2 The reinforcement percentage for tension reinforcement on one side of flexural members, eccentrically and axially loaded tension members shall not be less than $45 f_{td}/f_{sd}$, and meanwhile, shall not be less than 0.2.

3 The reinforcement percentage for all longitudinal reinforcement in an axially or eccentrically loaded compression member as well as that for longitudinal reinforcement on one side (including compression reinforcement in tension member loaded with large eccentricities) shall be calculated by gross area of the section of the member. The reinforcement percentage for tension reinforcement on one side of axially loaded tension members or tension members loaded with small eccentricities shall be calculated by gross cross-sectional areas of the members. The reinforcement percentage for tension reinforcement on one side of flexural members or tension members loaded with large eccentricities shall be $100A_s/(bh_0)$, in which A_s is the cross-sectional area of tension reinforcement, b is the web width (the sum of web widths for a box section), and h_0 is the effective depth. When arranging the reinforcement along the circumference of a member section, "compression reinforcement on one side" or "tension reinforcement on one side" refers to the longitudinal reinforcement arranged on one of the two opposite sides along the load direction.

9.1.13 The minimum reinforcement ratio for prestressed concrete flexural members shall satisfy the following conditions:

$$\frac{M_{ud}}{M_{cr}} \geqslant 1.0 \tag{9.1.13}$$

where:

M_{ud}—design flexural resistance of the cross-section of the flexural member, which is calculated according to the right-hand side of the relevant formulas in Section 5.2 of the *Specifications*;

M_{cr}—cracking moment on the cross-section of the flexural member, which is calculated by Eq. (6.5.2-7).

The cross-sectional area of reinforcing steel in tension in partially prestressed concrete flexural members shall not be less than $0.003bh_0$.

9.2 Slabs

9.2.1 The thickness of the top and bottom flanges in voided slab bridges shall not be less than 80 mm. The ends of holes in voided slabs shall be filled. The thickness of sidewalk slabs shall not be less than 80 mm for cast-in-place concrete slabs and 60 mm for precast concrete slabs.

9.2.2 The diameter of primary reinforcement in deck slabs shall not be less than 10 mm. The diameter of primary reinforcement in sidewalk slabs shall not be less than 8 mm. At the mid-span of simply supported slabs and the support regions of continuous slabs, the spacing of primary reinforcement shall not be larger than 200 mm, and the minimum clear spacing and layer spacing shall comply with the provision in Clause 9.3.3 of the *Specifications*.

9.2.3 Primary reinforcement in deck slabs may be bent up at 1/4 ~ 1/6 design span along the longitudinal axis through the center of the slab thickness and with an angle of 30° ~ 45°. For the primary reinforcement through supports without being bent up, the number per meter of slab width shall not be less than 3, and the area shall not be less than 1/4 cross-sectional area of the primary reinforcement.

9.2.4 Distribution reinforcement perpendicular to primary reinforcement shall be provided in deck slabs. Distribution reinforcement shall be arranged on the inside layer of the primary reinforcement, the diameter shall not be less than 8 mm, the spacing shall not be larger than 200 mm, and the cross-sectional area should not be less than 0.1% that of the slab. At the bending location of primary reinforcement, distribution reinforcement shall be arranged. The diameter of distribution reinforcement in sidewalk slabs shall not be less than 6 mm, and the spacing shall not be larger than 200 mm.

9.2.5 In arranging the reinforcement in a two-way slab supported on all sides, the slab may be divided into three parts along the longitudinal and transverse directions, respectively. The widths of the edge parts are 1/4 the slab width at the short side. The reinforcement in the middle parts shall be arranged with the calculated amount, and the reinforcement in the edge parts shall be arranged with half the amount of the reinforcement for the middle parts. The spacing of the reinforcement shall not be larger than 250 mm and shall not be larger than twice the slab thickness.

9.2.6 Reinforcement in skewed slabs shall be arranged in accordance with the following provisions (Figure 9.2.6):

1 When the skew angle of monolithic skewed slabs (the angle between the line normal to the axis of a support and the longitudinal axis of the bridge) is not larger than 15°, the primary reinforcement may be arranged parallel to the longitudinal axis of the bridge. When the skew angle of monolithic skewed slabs is larger than 15°, the primary reinforcement should be arranged perpendicular to the axes of supports of the slabs. Meanwhile, no less than three reinforcing bars shall be arranged adjacent and parallel to the free edges of the slabs and be tied with stirrups. At the obtuse corner close to the top of the slab, strengthening reinforcement shall be arranged perpendicular to the bisector of the obtuse corner; at the obtuse corner close to the bottom of the slab, strengthening reinforcement shall be arranged parallel to the bisector of the obtuse corner. The strengthening reinforcement should have a diameter not less than 12 mm, a spacing between 100 ~ 150 mm, and be arranged in a fanned area along the length of 1.0 ~ 1.5 m

at the two sides of the obtuse corner.

2 Distribution reinforcement in skewed slabs should be arranged perpendicular to the primary reinforcement. Their diameter, spacing and quantity may conform to Clause 9.2.4 of the *Specifications*. Additional distribution reinforcement parallel to the axis of support should be arranged in the vicinity of the support of the skewed slab, or the distribution reinforcement should be fanned out towards the support to have a transition parallel to the axis of the support.

3 Primary reinforcement in precast skewed slabs may be parallel to the longitudinal axis of the bridge, and strengthening reinforcement at the obtuse corners and distribution reinforcement should be arranged in accordance with the requirements of Items 1 and 2 of this clause.

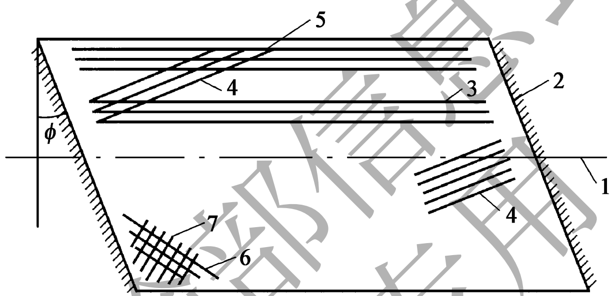


Figure 9.2.6 Layout of reinforcement in a skewed bridge

1-Longitudinal axis of the bridge; 2-Axis of support; 3-Reinforcement along the longitudinal axis of the bridge; 4- Reinforcment perpendicular to the axis of support; 5-Reinforcement adjacent to the free edge; 6-Reinforcement perpendicular to the bisector of obtuse angle; 7-Reinforcement parallel to the bisector of obtuse angle

9.2.7 For composite slabs with precast slabs and cast-in-place concrete layer, the top surfaces of the precast slabs shall be roughened to an amplitude no less than 6 mm. If vertical reinforcement for composition is arranged at the interface, the reinforcement shall be embedded into the precast slabs and the cast-in-place layers. The embedment depth shall not be less than 10 times the diameter of the reinforcement, and the longitudinal spacing of the reinforcement shall not be larger than 500 mm.

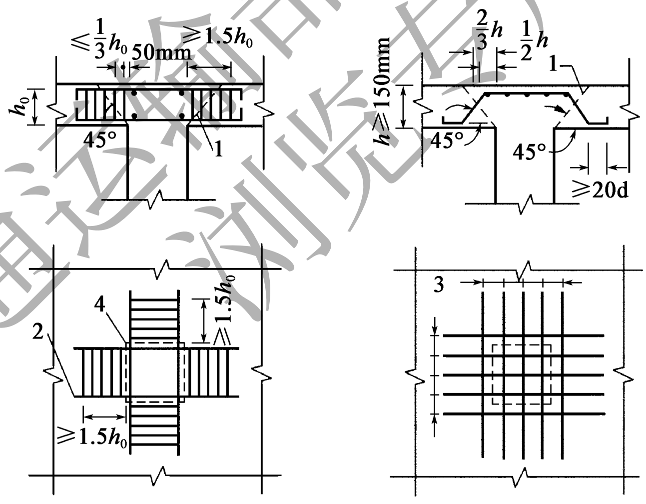
9.2.8 If longitudinal shear keys are used for the prefabricated slabs, the width of the upper opening of the shear keys shall meet the requirement for the operation of the immersion vibrator during construction, and the depth of the shear keys should be 2/3 the thickness of the precast slabs. Reinforcement extended into shear keys shall be pre-embedded in the precast slabs. Cast-in-place concrete layers shall be provided on the top surfaces of the slabs jointed by shear keys, and the thickness of the cast-in-place concrete layers should not be less than 80 mm.

9.2.9 Slabs supported by single-column piers, as well as their stirrups or bent bars required according to punching shear analysis shall comply with the following provisions:

1 The slab thickness shall not be less than 150 mm.

2 The diameter of stirrups shall not be less than 8 mm, and their spacing shall not be larger than $1/3h_0$. Closed stirrups shall be used to hoop the reinforcement in the rebar frame. The stirrups required by the calculation shall be arranged within the truncated pyramids of punching shear failure. Moreover, stirrups with equal diameters and spacing shall be arranged within the extent no less than $0.5h_0$ beyond the inclined surfaces of punching shear failure [the extent for placing the stirrups at each side shall be larger than or equal to $1.5h_0$, see Figure 9.2.9a)].

3 The diameter of bent bars shall not be less than 12 mm, the bend angle may be 30°~45° depending on the slab thickness, and the number for each direction shall not be less than 5. The inclined segments of bent bars shall intersect with the inclined surfaces of punching shear failure, and the points of intersection shall be located within the extent of $\left(\frac{1}{2}\sim\frac{2}{3}\right)h$ beyond the perimeter of the concentrated reaction area [Figure 9.2.9b)].



a)Stirrups b)Bent bars

gure 9.2.9 Layouts of reinforcement against punching shear in the slab over the single-column pier

1-Inclined surface of the truncated pyramid for punching shear failure; 2-Reinforcement in the rebar frame; 3-Bent bar; 4-Perimeter of the concentrated reaction area

9.3 Beams

9.3.1 The arrangement of diaphragms in concrete superstructures shall satisfy the following requirements:

1 In prefabricated T-beam bridges, diaphragms shall be provided at the ends of the span and within the span. When the rigid connection is employed between beams in the transverse direction, the spacing between diaphragms shall not be larger than 10 m.

2 In prefabricated composite box girder bridges, end diaphragms shall be provided, and intermediate diaphragms should be provided according to the practical situation of the structures.

3 For box girder bridges, end diaphragms shall be provided inside the boxes. Diaphragms within the span shall be provided for curved box girders with an inner radius less than 240 m. Their spacing shall not be larger than 10 m for reinforced concrete box girder bridges, and shall be determined through structural analysis for prestressed concrete box girder bridges. Diaphragms within cantilever spans shall also be provided for cantilever box girder bridges with a cantilever span of 50 m or above. If conditions permit, manholes for inspection shall be provided in the diaphragms.

9.3.2 Dimensions of beams shall satisfy the following requirements:

1 The depth of the cantilever flange ends in precast T-beams or box girders shall not be less than 100 mm. For precast T-beams jointed together transversely by cast-in-place concrete monolithically, or for decks of box girders prestressed by transverse prestressing steel, the depth of the cantilever flange ends shall not be less than 140 mm. For T- and I-beams, the depth of flanges at the connection with webs shall not be less than 1/10 of the beam depth. When there is a haunch at this location, the additional depth of the haunch may be included in the flange depth; if $\tan\alpha > 1/3$ for the bottom slope of the haunch, 1/3 is used.

2 Haunches shall be provided at the connections between top flanges and webs of the box girder; fillets shall be provided at the connections between bottom flanges and webs, and haunches may be provided if necessary. The depth at the middle of the top or bottom flanges of box girders shall not be less than 1/30 of the clear span of the flanges, and shall not be less than 200 mm.

3 The web width of T-beams, I-beams or box girders shall not be less than 160 mm. The web depth between the upper and lower haunches shall not be larger than 20 times the web width when vertical prestressing steel is used in the webs, and shall not be larger than 15 times the web width when vertical prestressing steel is not used in the webs. When the web width is variable, the length of the transition segment should not be less than 12 times the

difference of the web width. When T-beams, I-beams or box girders are subjected to torsion, the average web width shall also conform to the requirement in the note of Clause 5.5.5 of the *Specifications*.

4 For continuous beams with haunches in the longitudinal direction, the ratio of depth to length of the haunches should not be larger than 1/6.

9.3.3 In the design of the clear spacing of reinforcement in flexural members, it shall be taken into account that a vibrator can be inserted easily during concreting.

The transverse clear spacing of primary reinforcement and the vertical clear spacing between reinforcement layers shall not be less than 30 mm and the diameter of the reinforcement, when the number of reinforcement layers is equal to or less than 3. They shall not be less than 40 mm and 1.25 times the diameter of the reinforcement, when the number of reinforcement layers is more than 3. For bundled bars, the effective diameter shall be used.

9.3.4 Tension reinforcement in top flanges of T-beams or box girders to carrying the local loads shall comply with the provisions in Clause 9.2.2 of the *Specifications*. Distribution reinforcement shall be arranged perpendicular to the tension reinforcement, and may be arranged in accordance with the provisions in Clause 9.2.4 of the *Specifications*.

For tension reinforcement in top flanges of box girders to carrying the local loads, a portion of the reinforcement may be bent up near the web, extended to the cantilever end through the web, and hooked at the end. The number of reinforcement without bending up shall not be less than 3 per meter, and they shall be extended to the cantilever flange ends. For the cantilever flange with a length l_c larger than 2.5 m obtained by the provision in Clause 4.2.5 of the *Specifications*, the cross-sectional area of the above reinforcement without bending up shall also not be less than 60% that of reinforcement subjected to the negative moment at the base of the cantilever.

9.3.5 In the top and bottom layers of the bottom flanges of box girders, construction reinforcement shall be arranged both parallel and perpendicular to the bridge span. The cross-sectional area of the reinforcement shall not be less than 0.4% that of the reinforced bottom flange for reinforced concrete bridges, and shall not be less than 0.3% that of the reinforced bottom flange for prestressed concrete bridges. The above reinforcement may also be used as primary reinforcement. The reinforcement may be arranged by segments when the depth of the bottom flange varies. The diameter of the reinforcement should not be less than 10 mm, and the spacing should not be larger than 300 mm.

9.3.6 Primary reinforcement in reinforced concrete T-beams or box girders shall be arranged within the effective width of flanges specified in Clause 4.3.3 or 4.3.4 of the *Specifications*. Beyond this width, additional reinforcement with a cross-sectional area no less than 0.4% that of the exceeding part should be provided. Prestressing steel in prestressed concrete T-beams or box girders should mostly be arranged within the effective width.

9.3.7 At both sides of webs in T-beams, I-beams or box girders, longitudinal reinforcement with a diameter of 6 ~ 8 mm shall be provided. The cross-sectional area of the reinforcement in each web should be $(0.001 \sim 0.002)bh$, where b is the web width, and h is the beam depth. Their spacing in tension zones shall not be larger than the web width and 200 mm, and shall not be larger than 300 mm in compression zones. In the zones subjected to large shear force in the vicinity of supports and the anchorage zones of prestressed concrete beams, the cross-sectional area of longitudinal reinforcement at both sides of webs shall be increased, and the spacing of longitudinal reinforcement should be 100 ~ 150 mm.

9.3.8 Longitudinal tension reinforcement in reinforced concrete beams should not be cut off at tension zones. Whenever cut-off is required, the position shall be at least extended $(l_a + h_0)$ from the section that fully develops the strength of reinforcement according to the analysis of flexural resistance on the cross-section (Figure 9.3.8), herein, l_a is the minimum development length of the tension reinforcement, h_0 is the effective depth of the beam section. Meanwhile, the tension reinforcement shall be extended at least $20d$ ($25d$ for epoxy-coated reinforcement) from the section where it is not required according to the analysis of flexural resistance on cross-section, herein, d is the nominal diameter of the reinforcement. If longitudinal compression reinforcement is cut off within the span, it shall be extended at least $15d$ ($20d$ for epoxy-coated reinforcement) beyond the section where the reinforcement is not required according to the analysis.

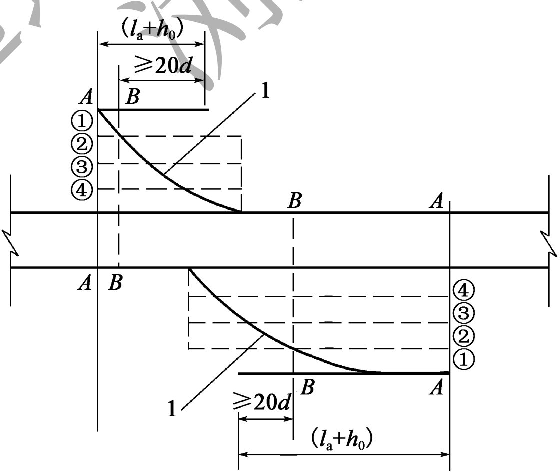


Figure 9.3.8 Extension of longitudinal tension reinforcement cut-off

A-A-section that fully develops the strength of reinforcement ①, ②, ③, ④; *B-B*-section where reinforcement ① is not required according to the analysis; ①,②,③,④-numbering of reinforcement; 1-bending moment diagram

9.3.9 At least two and no less than 1/5 the total number of bottom primary tension reinforcement shall pass through the end supports of a reinforced concrete beam. Reinforcement at both sides shall extend outside the end supports, be bent into the right angle, extend to the top along the beam depth, and be connected with the top longitudinal reinforcement in the rebar frame. For other reinforcement without bending up between both sides, the extension length beyond the sections at the supports shall not be less than 10 times the diameter of reinforcement (12.5 times the diameter for epoxy-coated reinforcement); and semicircular hooks shall be provided for HPB300 reinforcement.

9.3.10 When bent bars are provided in reinforced concrete beams, the bend angle should be 45°. The bending point of bent bars in tension zones shall be set at the position no less than $h_0/2$ beyond the section that fully develops the strength of the reinforcement according to the analysis of flexural resistance on the cross-section, herein, h_0 is the effective depth of the beam. Bent bars may be bent ahead of the section where the reinforcement is not required according to the analysis of flexural resistance on the cross-section, but the point of intersection between bent bars and the center line of the beam shall be located outside the section where the reinforcement is not required according to the analysis (Figure 9.3.10). Embedment length shall be reserved at the ends of the bent bars; it shall not be less than 20 times the diameter of reinforcement in tension zones, 10 times the diameter of reinforcement in compression zones, and shall be increased by 25% for epoxy-coated reinforcement. Semicircular hooks shall be provided for HPB300 reinforcement.

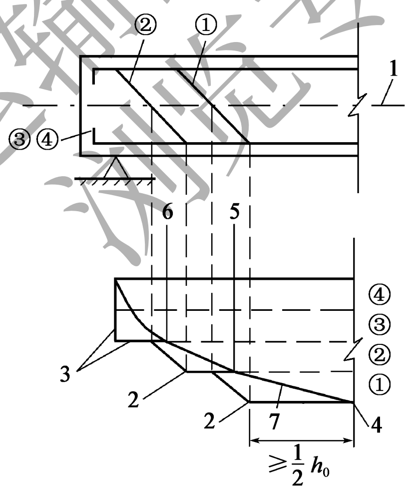


Figure 9.3.10 Bending points of bent bar

1-Center line of beam; 2-Bending point of reinforcement in tension zone; 3-Diagram of flexural resistance on the cross-section; 4-Section that fully develops the strength of reinforcement ①~④; 5-Section where reinforcement ① is not required according to the analysis (the section that fully develops the strength of reinforcement ②~④); 6-Section where reinforcement ② is not required according to the analysis (the section that fully develops the strength of reinforcement ③~④); 7-Bending moment diagram; ①, ②, ③, ④-numbering of reinforcement

The top bending points of the first row of bent bars near supports shall be located at the central sections of supports in simply-supported beams or end supports in continuous beams, and they shall be located at the edge of the diaphragm towards the span side in interior supports for cantilever or continuous beams. The top bending points of each sequential row (towards the span) of bent bars shall be located at or within the bottom bending points of the former row (towards the support) of the bent bars.

Suspended bars shall not be used as bent bars.

9.3.11 When multi-layer welded reinforcement is used in reinforced concrete beams, the following requirements should be satisfied:

1 Side welds are adopted in multi-layer welded reinforcement to form a frame (Figure 9.3.11). Side welds are provided at the bending points of bent bars, and short welds are provided appropriately at the middle of the straight segment.

2 In addition to the bent longitudinal bars, special bent steel bars may also be used in the welded rebar frame.

3 Double-sided welds should be adopted for the welding between inclined bars and longitudinal bars, their lengths shall be 5 times the bar diameter, and the short welds between longitudinal bars shall be 2.5 times the bar diameter. When single-sided welds are necessary, their lengths shall be doubled.

4 The number of layers of reinforcement in a welded rebar frame shall not be larger than 6, and the diameter of an individual reinforcement shall not be larger than 32 mm.

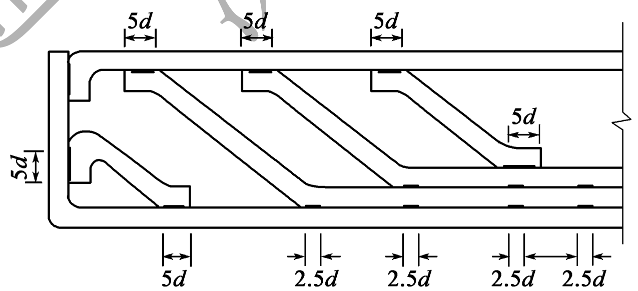


Figure 9.3.11 Welded rebar frame

9.3.12 Stirrups in beams shall conform to the following requirements:

1 Stirrups with a diameter no less than 8 mm and no less than 1/4 the diameter of primary reinforcement shall be provided in reinforced concrete beams, and their reinforcement ratio ρ_{sv} is referred to Clause 5.2.9 of the *Specifications*. The reinforcement ratio shall be no less than 0.14% for HPB300 reinforcement and 0.11% for HRB400 reinforcement.

2 If longitudinal compression reinforcement required by the load-carrying analysis is provided in girders or girder segments near interior supports in the negative moment zone for continuous or cantilever beams, closed stirrups shall be used. Meanwhile, the spacing between any longitudinal compression reinforcement in one row and the longitudinal reinforcement at the corner of the stirrup shall not be greater than the larger one of 150 mm and 15 times the diameter of the stirrup. Otherwise, composite stirrups and tie bars shall be provided (see Figure 9.6.1). Hooked joints of adjacent stirrups shall be arranged in stagger along the longitudinal direction.

3 The stirrup spacing shall not be larger than 1/2 the beam depth and 400 mm. If the reinforcement to be hooked is longitudinal compression reinforcement according to the load-carrying requirement, the stirrup spacing shall not be larger than 15 times the diameter of the reinforcement to be hooked, and also shall not be larger than 400 mm. For the lap spliced reinforcement in tension, the stirrup spacing within the range of lap splices for reinforcement shall not be larger than 5 times the diameter of the primary reinforcement, and also shall not be larger than 100 mm; for the lap spliced reinforcement in compression, the spacing shall not be larger than 10 times the diameter of the primary reinforcement, and also shall not be larger than 200 mm. Within the range of a length no less than the beam depth from the support centers towards the span, the of stirrup spacing should not be larger than 100 mm.

4 The first stirrup near the beam end shall be arranged at the location with a concrete cover thickness from the end surface. Within the junction between beams or between beam and column, the distance between the interface and the stirrups that are close to the interface should not be larger than 50 mm.

9.3.13 Stirrups and longitudinal reinforcement in members subjected to combined flexure, shear and torsion shall also conform to the following requirements:

1 Closed stirrups with 135° end hooks shall be used. Hooks shall be tied tightly with longitudinal reinforcement, and hooked joints of adjacent stirrups shall be arranged in stagger in the longitudinal direction.

2 Longitudinal reinforcement subjected to torsion shall be arranged uniformly and symmetrically along the perimeter of the section, and the spacing shall not be larger than 300 mm. Longitudinal reinforcement shall be arranged at the four corners of the basic element with a rectangular section, and their ends shall have the minimum development length for tension reinforcement as specified in Clause 9.1.4 of the *Specifications*.

3 The reinforcement ratio for stirrups, ρ_{sv}, shall not be less than $\left[(2\beta_t-1)\left(0.055\frac{f_{cd}}{f_{sv}}-c\right)+c\right]$, where β_t is calculated in accordance with Clause 5.5.4 of the *Specifications*, c is taken as 0.0014 when using HPB300 reinforcement and 0.0011 when using HRB400 reinforcement. For members subjected to pure torsion (flanges of a girder), ρ_{sv} shall not be less than $0.055f_{cd}/f_{sv}$.

4 The longitudinal reinforcement ratio shall not be less than the sum of the minimum reinforcement ratios for longitudinal primary reinforcement in flexural members and in torsion members. For flexural members, the minimum reinforcement ratio for longitudinal primary reinforcement shall be taken in accordance with Clause 9.1.12 of the *Specifications*. For torsion members, the minimum reinforcement ratio for longitudinal primary reinforcement $[A_{st,min}/(bh)]$ shall be taken as $0.08(2\beta_t-1)f_{cd}/f_{sd}$ to resist shear and torsion, and $0.08f_{cd}/f_{sd}$ to resist pure torsion, where $A_{st,min}$ is the minimum cross-sectional area of all longitudinal reinforcement in a member subjected to pure torsion, h is the long side length of the basic element with rectangular section, b is the short side length, and f_{sd} is the design tensile strength of the longitudinal reinforcement.

9.3.14 For the curved soffit of a girder, longitudinal tension reinforcement close to the concave surface shall be fastened with stirrups. The stirrup spacing shall not be larger than 10 times the diameter of the hooked primary reinforcement, and the diameter of the stirrups shall not be less than 8 mm. The cross-sectional area of each single-leg stirrup may be calculated by the following formulas:

$$A_{sv1}\geqslant mA_s\frac{s_v}{2r} \tag{9.3.14-1}$$

$$r=\frac{l}{2}\left(\frac{1}{4\beta}+\beta\right) \tag{9.3.14-2}$$

where:

A_{sv1}—cross-sectional area of each single leg stirrup;

m—ratio of design tensile strength of primary reinforcement to design tensile strength of stirrup;

A_s—cross-sectional area of primary reinforcement hooked by one stirrup (two legs);

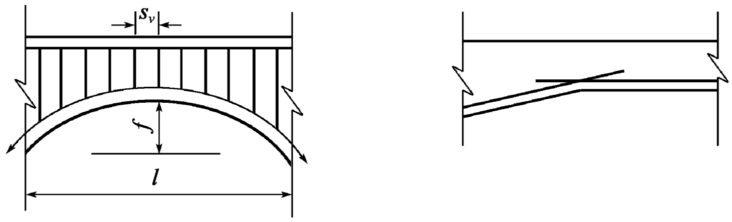
r—radius of the circular curve of the concave surface, which may be approximately calculated by Eq. (9.3.14-2) for other curves;

s_v—stirrup spacing[Figure 9.3.14a)];

l—chord length of the curve [Figure 9.3.14a)];

β—ratio of rise f to chord length l of the curve.

For primary reinforcement arranged in a cross way at the corner, an additional development length shall be extended separately from the intersection point at the corner [Figure 9.3.14b)], in which the longitudinal tension reinforcement shall be extended to the opposite side and anchored in the compression zone. The extent of the compression zone may be determined according to the calculated physical depth of the compression zone.



a)Stirrups in concave curved beam b)Primary reinforcement crossed at corner

Figure 9.3.14 Layouts of stirrups in concave curved girder and primary reinforcement crossed at corner

9.3.15 Top flanges of adjacent precast T-beams should be connected monolithically with cast-in-place concrete in the transverse direction, where a loop connection may be used for the transverse primary reinforcement. Diaphragms in precast T-beams should be connected monolithically with cast-in-place concrete.

The concrete age difference between precast beams and cast-in-place connections shall not exceed 3 months.

9.3.16 In composite beams, the depth of the cast-in-place concrete layer at the junction with precast beams should not be less than 150 mm. The top surface of the precast beams shall be roughened to an amplitude no less than 6 mm.

9.3.17 Stirrups of precast beams in composite beams shall inserted into the cast-in-place deck slabs, and the insertion length shall not be less than 10 times the diameter of the stirrups.

9.4 Prestressed Concrete Superstructures

9.4.1 When vertical prestressing steel is arranged in prestressed concrete beams, their longitudinal spacing should be 500 ~ 1000 mm.

Stirrups shall be provided in webs of prestressed concrete beams, and the diameter shall be no less than 10 mm for T- and I-beams and 12 mm for box girders. They shall be deformed bars, and their spacing should not be larger than 200 mm. Within the extent no less than the beam depth from the centers of supports, closed stirrups shall be used and the spacing shall not be larger than 120 mm.

In horseshoe-shaped portion at the bottom parts of T- and I-beams, closed stirrups with a diameter no less than 8 mm shall be provided, and the spacing shall not be larger than 200 mm.

9.4.2 A mixed reinforcement arrangement shall be used for partially prestressed concrete beams. Deformed bars with small diameters should be used as reinforcing steel at the edges of tension zones and arranged with small spacing.

9.4.3 Steel strands or spiral ribbed steel wires should be used as prestressing steel in pretensioned members. When plain wires are used as prestressing steel, proper measures shall be taken to ensure that the wires are reliably anchored in concrete.

9.4.4 In pretensioned members, the clear spacing of prestressing steel strands shall not be less than 1.5 times their nominal diameter, and shall not be less than 25 mm for 1 ×7 steel strands. The clear spacing of prestressing steel wires shall not be less than 15 mm.

9.4.5 In pretensioned members, spiral reinforcement with a length no less than 150 mm shall be arranged at the ends of the single prestressing tendon; for multiple prestressing tendons, 3 ~ 5 sheets of wire fabric shall be arranged at the ends of the members within the range of 10 times the diameter of the prestressing steel.

9.4.6 At the anchorage zones of post-tensioned members, bearing plates with trumpet pipes shall be used ahead of anchorage devices. Confinement reinforcement shall be provided ahead of bearing plates, and the volumetric ratio (see Clause 5.7.2 of the *Specifications*) shall not be less than 0.5%.

9.4.7 A part of prestressing steel in post-tensioned concrete beams (including cast-in-place segments of end spans in continuous beams) shall be bent up in pairs symmetrically along the transverse direction at the segments near the end supports. The prestressing steel should be anchored uniformly at the end planes of the beams; meanwhile, the webs of the beams may be widened along the longitudinal direction. In the vicinity of the beam ends, longitudinal reinforcement and stirrups should be closely spaced in accordance with the requirements in Clauses 9.3.7 and 9.4.1 of the *Specifications*.

9.4.8 For members with curved configuration and arranged with curved tendons, the minimum concrete cover over their ducts inside and outside the curvature shall be calculated by the following formula:

1 Radial direction toward the center of curve inside the curve plane

$$C_{in} \geqslant \frac{P_d}{0.266 r \sqrt{f'_{cu}}} - \frac{d_s}{2} \tag{9.4.8-1}$$

where:

C_{in}—minimum concrete cover inside the curvature (mm);

P_d—design jacking force of prestressing steel (N), which may be taken as 1.2 times the jacking force after subtracting the losses of prestress due to anchor set, the retraction of reinforcement, and the duct friction at the calculated section;

r—radius of curvature of duct (mm), which may be calculated by Eq. (9.3.14-2);

f'_{cu}—compressive strength of 150 mm concrete cubes during the jacking of prestressing steel (MPa);

d_s—outer diameter of duct (mm).

When the thickness of the concrete cover calculated by Eq. (9.4.8-1) is large, the minimum concrete cover specified in Clause 9.1.1 of the *Specifications* may be used, but stirrups shall be provided inside the curvature for curved duct segments. The cross-sectional area of one leg stirrup should be calculated by the formula:

$$A_{sv1} \geqslant \frac{P_d s_v}{2 r f_{sv}} \tag{9.4.8-2}$$

where:

A_{sv1}—cross-sectional area of one leg stirrup (mm^2);

s_v—stirrup spacing (mm);

f_{sv}—design tensile strength of stirrup (MPa), which is taken from Table 3.2.3-1.

2 Radial direction away from the center of curve outside the curve plane:

$$C_{out} \geqslant \frac{P_d}{0.266 \pi r \sqrt{f'_{cu}}} - \frac{d_s}{2} \tag{9.4.8-3}$$

where:

C_{out}—minimum concrete cover outside the curvature (mm).

3 When the thickness of the concrete cover calculated by the above formulas is smaller than that specified in Clause 9.1.1 of the *Specifications*, the thickness of the concrete cover for the corresponding environmental condition shall be taken in accordance with the provisions in Clause 9.1.1 of the *Specifications*.

9.4.9 For post-tensioned members, the arrangement of prestressing tendon ducts shall comply with the following provisions:

1 The clear spacing between straight ducts shall not be less than 40 mm, and should not be less than 0.6 times the diameter of the ducts. For embedded ducts with metal corrugated ducts, plastic corrugated ducts or sheet metal ducts, two ducts may be stacked in the vertical direction of the straight ducts.

2 The minimum clear spacing between adjacent ducts inside the curvature of curved tendon ducts shall be calculated by Item 1 of Clause 9.4.8 of the *Specifications*, in which P_d and r are the design jacking force and the radius of curvature of the prestressing tendon in one of the two adjacent ducts with a larger diameter, respectively; C_{in} is the clear spacing between the outer edges of the two adjacent ducts inside the curvature. When the above-calculated value is smaller than the clear spacing between the outer edges of the corresponding straight ducts, the latter shall be used.

The minimum clear spacing between adjacent ducts outside the curvature for curved tendon ducts shall be calculated by Item 2 of Clause 9.4.8 of the *Specifications*, in which C_{out} is the clear spacing between the outer edges of the two adjacent ducts outside the curvature.

3 The cross-sectional area within the inner diameter of the ducts shall not be less than twice the cross-sectional area of the prestressing tendons.

4 When the pre-camber is required to be set according to the analysis, the reserved ducts shall also be cambered simultaneously.

9.4.10 The curvature radius of the curved prestressing steel in post-tensioned members shall comply with the following provisions:

1 When the diameter of wires in bundled steel wires or bundled steel strands is smaller than or equal to 5 mm, the radius of curvature should not be less than 4 m. When the diameter of wires is larger than 5 mm, it should not be less than 6 m.

2 When the diameter of prestressing threaded bars is smaller than or equal to 25 mm, the radius of curvature should not be less than 12 m. When the diameter is larger than 25 mm, it should not be less than 15 m.

9.4.11 The minimum straight extension of curved bundled steel wires or bundled steel strands ahead of anchorages for post-tensioned members should be taken to be 0.80 ~ 1.50 m.

9.4.12 The cement grout used for grouting prestressing tendon ducts shall have a compressive strength of not less than 50 MPa when tested on 40 mm × 40 mm × 160 mm specimens after 28 days of standard curing in accordance with the provisions of the current *Test Method of Cement Mortar Strength (ISO Method)* (GB/T 17671—2021). To reduce shrinkage, a proper amount of expansive agents determined through testing may be added.

9.4.13 After the completion of prestressing, construction reinforcement shall be provided in the vicinity of anchorage devices to connect with the bars in the beams, and then the anchorages are sealed by concrete casting. The strength class of the concrete for sealing the anchorages shall not be lower than 80% of that of the members, and also shall not be lower than C30.

9.4.14 During the selection of the prestressing system and the arrangement of prestressing steel for continuous prestressed concrete beams, measures shall be taken to reduce the losses of prestress due to friction.

9.4.15 Along the full length of continuous beams, the number of prestressing steel should not be abruptly increased or reduced at a section or a segment. In the transition region between positive and negative moments of the beams, long lengths for overlapping may be provided for prestressing steel, which should be arranged dispersedly.

At the interior supports of continuous beams, reinforcing steel along the length of the bridge shall be provided in webs and their bottom flanges.

9.4.16 When prestressing steel needs to be anchored in the middle of the members, the anchoring points should be set near the gravity axis of the section or in the compression zones under external loads. If the section of the girder is weakened due to anchorage, reinforcing steel shall be used for strengthening. When prestressing steel in top or bottom flanges of the box girders are extended outside the flanges, they shall be anchored in the specially designed blisters. At the same time, the prestressing steel should have a large radius of curvature, and stirrups should be provided in accordance with Clause 9.4.8 of the *Specifications*.

9.4.17 Precast segmental prestressed concrete structures shall meet the following detailing requirements:

1 Wire fabric of steel bars with a diameter of not less than 10 mm shall be provided at the ends of precast segments.

2 Glued joints or cast-in-place wet joints should be adopted for the joints between precast segments. The epoxy may be used for glued joints, and fine aggregate concrete may be used for cast-in-place wet joints. The thickness of the epoxy shall be even, the epoxied joints shall be compressed until the epoxy has hardened. The width of joints with fine aggregate concrete shall not be less than 60 mm, and the concrete strength class shall not be lower than that of the precast segments.

3 Shear keys shall be provided at the joints between precast segments. Shear keys in webs, top slabs, bottom slabs and haunches should be used in accordance with Figure 9.4.17-1.

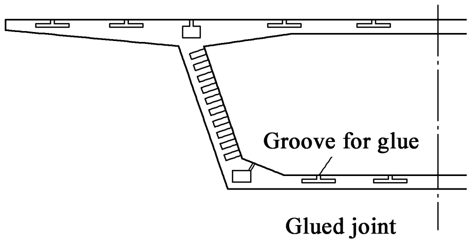


Figure 9.4.17-1 Arrangement of shear keys

The dimensions of shear keys shall conform to the following provisions (Figure 9.4.17-2):

1) The shear keys in webs should be arranged in a range not smaller than 75% of the girder depth. The transverse width of the shear keys should be 75% of the width of the webs.

2) Shear keys shall have sections of trapezoid (with an inclination angle close to 45°) or trapezoid with round fillet. The amplitude of shear keys shall be larger than twice the diameter of the top size of aggregate in the concrete and shall not be less than 35 mm. The ratio of the amplitude of shear keys to their mean width may be taken as 1:2.

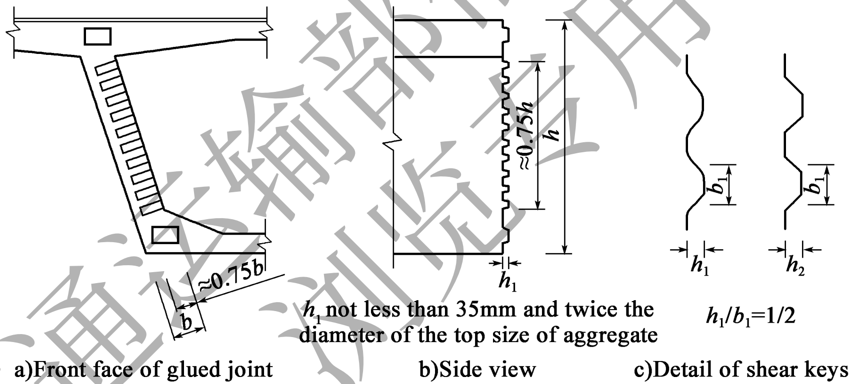


Figure 9.4.17-2 Dimensions of shear keys

9.4.18 At the end anchorage zones of post-tensioned members, reinforcing steel shall be applied in accordance with the following requirements:

1 Confinement reinforcement shall be provided in local zones ahead of anchorage devices. If flat bearing plates are used, wire fabric with no less than four layers or spiral reinforcement with no less than four turns shall be provided. If bearing plates with trumpet pipes are used, spiral reinforcement shall be provided, and the length of their turns shall be no less than the length of the trumpet pipe.

2 Closed stirrups shall be provided in general zones to resist transverse bursting force. Their spacing shall not be larger than 120 mm.

3 Reinforcement to resist surface spalling force shall be provided at the sections of the girder ends. When the anchorages with large eccentricity are used, the reinforcement at the side surfaces of the anchorages should be bent up and extended to the longitudinal tensile edges.

9.4.19 The main geometric parameters of the post-tensioned anchorage blisters include (Figure 9.4.19): radius of curvature of the tendon (R), angle of inclination of the tendon (α), dimensions of the anchorage plane (S_1 and S_2) and length of the blister (L). The radius of curvature of the prestressing tendon should be taken by referring to Clauses 9.4.10 and 9.4.16 of the *Specifications*. The deviation angle of the prestressing tendon in the blister should not be larger than 15°. The dimensions of the anchorage plane shall be determined according to the requirements for the placement of anchorage devices, the space for jacking, etc. The angle between the anchorage plane and the inclined plane of the blister should not be less than 90°. The length of the blister may be determined according to the geometrical relationship.

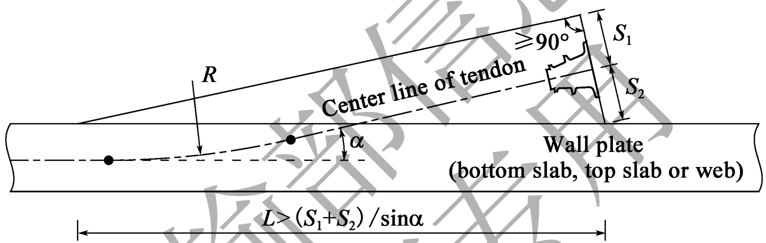


Figure 9.4.19 Elevation of an anchorage blister

9.4.20 Anchorage zones of post-tensioned blisters shall be reinforced based on analysis results. Detailing of reinforcing steel shall conform to the following requirements (Figure 9.4.20):

1 Stirrups to resist transverse bursting force shall be provided ahead of the anchorage of a blister. Their spacing should not be larger than 150 mm. The longitudinal distribution range should not be smaller than 1.2 times the height of the blister. Detailing requirements of confinement reinforcement for local compression ahead of the anchorage are the same as those in Item 1 of Clause 9.4.18 of the *Specifications*.

2 Stirrups shall be provided in the anchorage plane of the blister and extended to the outer edge of the wall plate.

3 Longitudinal reinforcement to resist tension ahead of the anchorage shall be provided inside the edge of the wall plate. When longitudinal strengthening reinforcement is required, the length should not be smaller than 1.5 m (centered at the intersecting line between the anchorage plane and the wall plate), and their transverse distribution range should be

within 1.5 times the width of the bearing plate at both sides of the center line of the tendon.

4 Longitudinal reinforcement to resist local lateral flexure of the edge shall be provided on the outer edge of the wall plate. When longitudinal strengthening reinforcement is required, the length should not be less than 1.5 m (centered at the position equal to the thickness of the wall plate ahead of the anchorage plane), and their transverse distribution range should be within 1.5 times the width of bearing plate at both sides of the center line of the tendon.

5 In the action zone of radial force of prestressing steel, vertical stirrups and U-shaped bursting steel along tendon ducts shall be provided and hooked with longitudinal reinforcement inside the wall plate, and their longitudinal distribution range should be taken as the full length of the segment with the curved tendon.

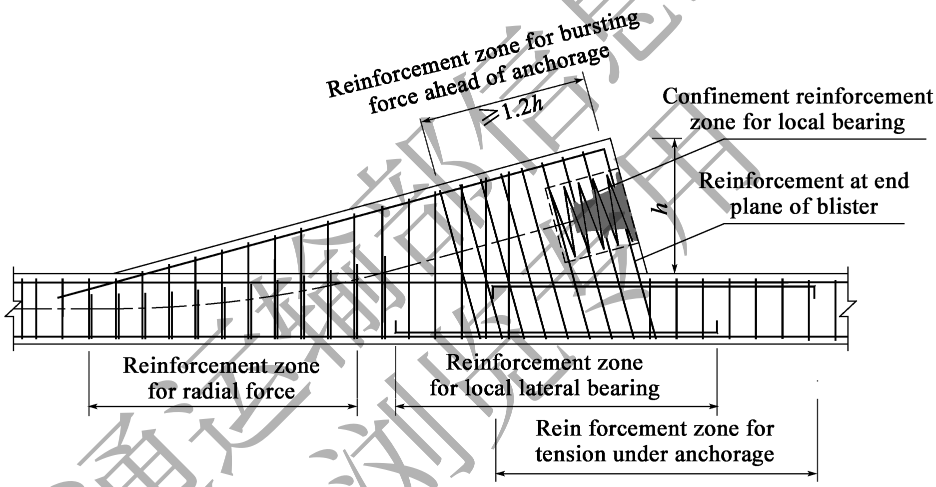


Figure 9.4.20 Arrangement of reinforcing steel in anchorage zone of blister

9.4.21 Spaces for maintenance and replacement of external prestressing systems as well as access for equipment shall be provided in externally prestressed concrete bridges.

9.4.22 For externally prestressed concrete bridges, the proportions of internal and external prestressing steel shall be determined and corrosion protection measures for external prestressing cables shall be selected, according to construction methods, design life of structures and environmental categories of bridge locations.

9.4.23 The minimum deviation radius for external prestressing strands shall comply with the requirements in Clause 9.4.10 of the *Specifications*.

9.4.24 Deviators for external prestressing tendons should be selected according to the stress requirements, which can be block type, transverse rib type, vertical rib type and crossbeam type, as illustrated in Figure 9.4.24.

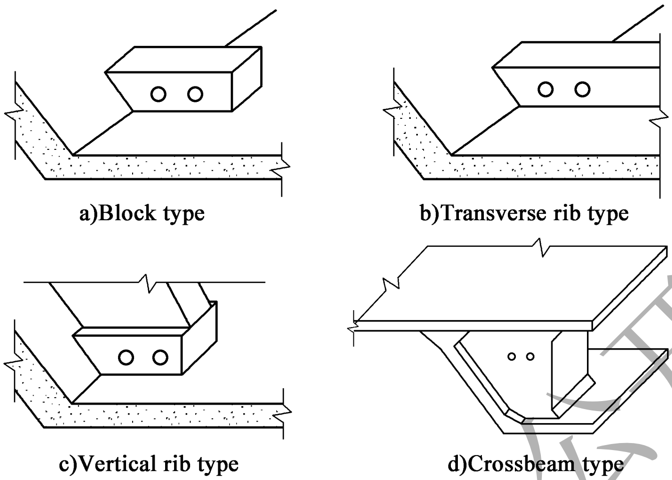


Figure 9.4.24 Deviator structures

9.4.25 Inner and outer closed hoops shall be provided in block-type deviators (Figure 9.4.25), in which an inner closed hoop encloses a single deviator, while an outer closed hoop encloses all deviators along the perimeter of the deviator structure.

The diameter of the inner closed hoop should not be less than 20 mm. The inner and outer closed hoops are arranged longitudinally along the deviators, and their longitudinal spacing should not be larger than 100 mm.

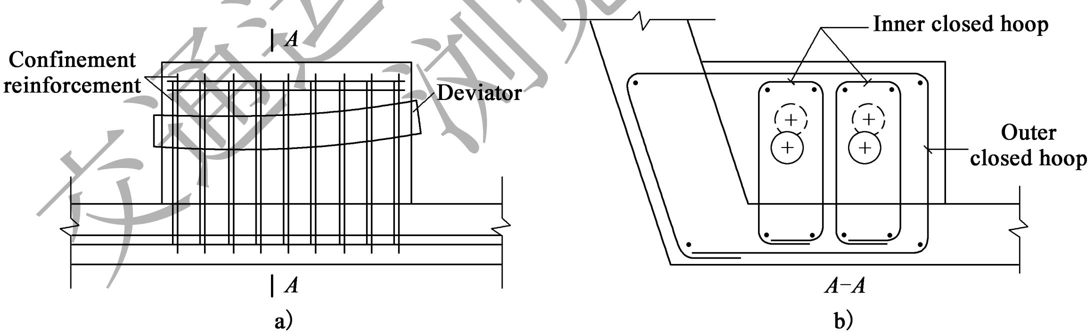


Figure 9.4.25 Reinforcement in block-type deviators

9.4.26 The free length of external prestressing tendons should not be larger than 8.0 m. The turning angle shall be set on the tendons between their free segments and connected anchor segments.

9.4.27 The longitudinal thickness of the crossbeams for anchorages shall be determined by the placement depth of the anchorages and the required length for tendon deviation, and it should not be less than 1000 mm. The dimensions of the anchorage plane of the crossbeams shall be determined by the requirements for the dimensions of anchorage placement and jacking space.

9.5 Arch Bridges

9.5.1 The rise-to-span ratio of reinforced concrete arches should be 1/4.5 ~ 1/8. Spans of spandrel structures of open-spandrel arch bridges shall be determined according to the stress states of the main arches. The arch axis coefficient for catenary arches should be 2.814 ~ 1.167.

9.5.2 The spandrel structures of open-spandrel arch bridges shall be able to adapt to the deformation of the arches. Their detailing shall conform to the following requirements:

1 Slabs or beams in spandrel structures should be simply supported. Their bearings may be elastomeric bearings with elastic restraints. Sliding bearings and expansion joints shall be provided at both ends of the bridge span.

2 Lateral bracings may be provided for spandrel columns if necessary.

3 Reinforcement in spandrel columns shall be arranged according to the load-carrying requirement of the structures and shall have enough development length.

4 Bearing beams of full length in the transverse direction shall be provided at the bottom of columns on barrel arches, and their depth should not be less than 1/5 of clear spacing between columns. Diaphragms in boxes shall be provided under spandrel columns or wall piers for box arch rings.

9.5.3 Primary reinforcement in arch rings or arch ribs of hingeless arches shall be extended into piers or abutments for anchoring. The development length not only shall satisfy the minimum development length specified in Table 9.1.4, but also shall conform to the following requirements:

1 For rectangular sections, it shall not be less than 1.5 times the sectional depth of the arch at the arch springings.

2 For T-sections, I-sections or box sections, it shall not be less than half of the sectional depth of the arch at the arch springings.

For three-hinged or two-hinged arches, wire fabric no less than three layers shall be provided in both piers or abutments and arch ribs where the hinges are located.

9.5.4 Lateral bracings shall be provided between the arch ribs for braced arches. At the locations under spandrel columns, lateral bracings must be provided between arch ribs. The depth of the lateral bracings may be 0.8 ~ 1.0 times that of the arch ribs, and the width may be 0.6 ~ 0.8 times the depth of arch ribs. Longitudinal reinforcement with a diameter no less than 16 mm shall be provided at the four corners of the lateral bracings, and stirrups with a diameter no less than 8 mm shall also be provided, and their spacing shall not be larger than the smaller dimension of the lateral bracings or 400 mm.

9.5.5 Lateral bracing systems shall be provided for half-through arches and tied arches.

9.5.6 Lateral bracing systems shall be provided for truss arches. Lateral stud bracings shall be provided at solid segments of the arch crown and at each node of top and bottom chords; the lateral stud bracing at the node of the first top chord from the bridge end shall be strengthened; vertical cross bracings shall be provided at the bridge ends; horizontal cross bracings shall be provided between two truss discs at the side panels of the bridge ends; vertical and horizontal cross bracings shall be provided at other locations of spans if the span is large; lateral stud bracings shall be provided at the nodes in the horizontal or vertical planes where cross bracings are arranged.

9.5.7 Multi-cell or single-cell box sections may be used for the top or bottom chords and diagonal or vertical web members in composite truss arch bridges. The nodes of the members shall be connected with lateral stud bracings. The arch shall be solid at the crown section.

The ratio of the end span to the main span in the composite truss arch bridges should be 0.2 ~ 0.4. A parabola may be used as the axis of the bottom chords. The interrupt points of top chords should be set at the location with a length 0.25 ~ 0.30 times the main span away from the arch crown.

9.5.8 The short sides of members in the lateral stud bracings, K-bracings and cross bracings in arch bridges should not be smaller than 1/15 of the length between supporting points or points of interaction. Longitudinal reinforcement with a diameter not less than 16 mm and stirrups with a diameter not less than 8 mm shall be provided for the members. Chamfers with inclined reinforcement shall be set at the connections between arch ribs and lateral stud bracings, K-bracings or cross bracings.

9.5.9 When members (including K-bracing and cross bracing) in truss arches or truss-type composite arches are intersected in the same plane, the adjoining edges of the intersected members shall have the transition length with arc or polyline shape, and the starting points of the transition lines at both sides of one member should approach to the same section. Enveloping reinforcement shall be arranged along the edges of the transition segment, and shall have enough development length inside the members. The primary reinforcement in each intersected member shall be extended over the center of the joint along the length direction of the members and shall have enough development length.

Stirrups near the joints shall be closely spaced.

9.5.10 When the span of a rigid-frame arch bridge is smaller than 25 m, the bridge structure may have only main arch legs but no secondary inclined legs. When the span is within 25 ~ 70 m, one secondary inclined leg should be added at each side of the arch. When the span is larger than 70 m, one more secondary inclined leg should be added. The length of the solid arch segments in a rigid frame arch may be 0.4 ~ 0.5 times the design span. The lateral central distance between the adjacent two arch discs should be in the range of 2.0 ~ 3.5 m, and lateral bracings should be provided between arch discs at every 3 ~ 5 m in the longitudinal direction.

9.5.11 For truss arch bridges or rigid frame arch bridges built on soft soil ground or in cold regions, the primary reinforcement in bottom chords near arch springings should be added properly, and their stirrups should also be closely spaced.

9.5.12 In multi-span arch bridges, robust pier or other measures that can resist the thrust from the arch at one side of the pier shall be provided depending on the requirements. Such a pier should be arranged for every 3 ~ 5 spans.

9.5.13 For arch bridges with flexible hangers, continuous longitudinal girders should be provided in the deck system.

9.6 Columns, Piers and Abutments, Pile Caps

9.6.1 For axially loaded members with reinforcing bars (or spiral reinforcement) excluding the driving piles, drilled/bored piles, the arrangement of primary reinforcement shall comply with the following provisions (Figure 9.6.1-1):

1 The diameter of longitudinal primary reinforcement shall not be less than 12 mm. The clear spacing shall not be less than 50 mm and also shall not be larger than 350 mm. The minimum clear spacing of longitudinal reinforcement in precast members by horizontal casting may conform to the provision of Clause 9.3.3 in the *Specifications*. The minimum reinforcement percentage for members shall conform to the provision of Clause 9.1.12 in the *Specifications*. The total longitudinal reinforcement ratio in members should not exceed 5%.

2 Longitudinal primary reinforcement shall be extended into foundation and cap beams, and the

extended length shall not be smaller than the development length specified in Table 9.1.4.

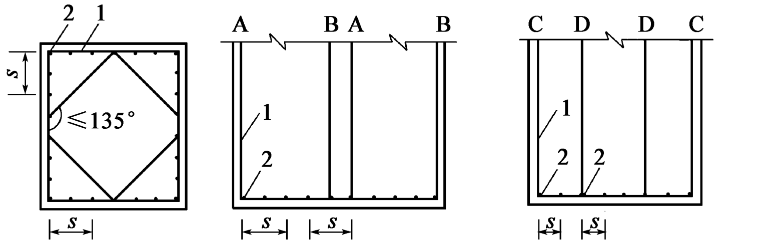


Figure 9.6.1-1 Arrangement of stirrups inside columns

1-Stirrup; 2-Corner reinforcement; A, B, C, D-Numbering of stirrup

Note: In Figures a) and b), the arrangement of Group (A and B) or Group (C and D) may be selected according to the actual situation.

3 Stirrups shall be of closed type, and their diameter shall not be smaller than 1/4 of that of longitudinal reinforcement and also shall not be less than 8 mm.

4 The stirrup spacing shall not be larger than 15 times that of longitudinal primary reinforcement, the short side of the members (0.8 times the diameter for circular sections) and 400 mm. The stirrup spacing within the splicing range of longitudinal primary reinforcement shall conform to the provision in Clause 9.3.12 of the *Specifications*.

When the cross-sectional area of the longitudinal reinforcement is larger than 3% of that of the concrete section, the stirrup spacing shall not be larger than 10 times the diameter of the longitudinal reinforcement and also shall not be larger than 200 mm.

5 Longitudinal primary reinforcement in members shall be arranged within the distance s, which is away from the center of corner reinforcement and shall not be larger than 150 mm or 15 times the diameter of stirrups (the larger value is taken). If the longitudinal primary reinforcement is arranged beyond this range, composite stirrups and tie reinforcement shall be provided. The hooked joints of adjacent stirrups shall be staggered in the longitudinal direction.

9.6.2 For axially loaded members with spiral or welded loop confinement reinforcement, the arrangement of reinforcement shall conform to the following provisions:

1 The cross-sectional area of longitudinal primary reinforcement shall not be less than 0.5% of the core area confined by stirrups. The core area shall not be less than 2/3 of the gross cross-sectional area of the member.

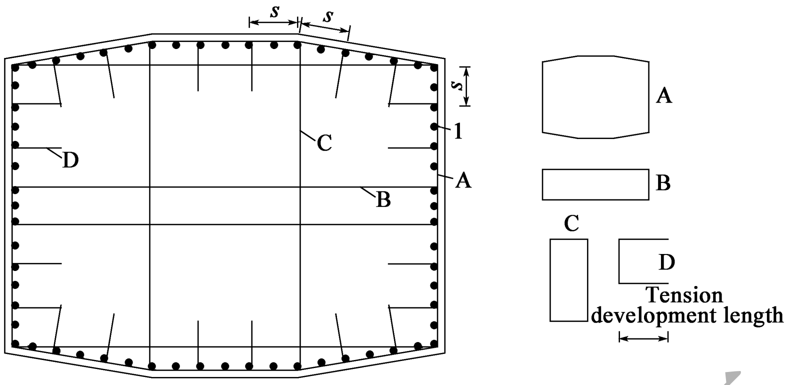


Figure 9.6.1-2 Arrangement of composite stirrups and tie reinforcement inside the column

1-Longitudinal reinforcement; A, B, C-Stirrups; D-Tie reinforcement

2 The pitch or spacing of confinement reinforcement shall not exceed either 1/5 of the core diameter or 80 mm and also shall not be less than 40 mm.

3 Longitudinal primary reinforcement shall be extended into the upper and bottom members connected with the compression member. The extended length shall not be less than the diameter of the compression member and the development length of longitudinal reinforcement.

4 The diameter of the confinement reinforcement shall not be less than 1/4 of that of the longitudinal reinforcement and also shall not be less than 8 mm.

9.6.3 The arrangement of reinforcement in eccentrically loaded compression members shall conform to the provision in Clause 9.6.1 of the *Specifications*. When the sectional depth of the eccentrically loaded compression member is $h \geqslant 600$ mm, longitudinal construction reinforcement with a diameter of 10 ~ 16 mm shall be arranged on the sides of the member, and composite stirrups shall be provided whenever necessary.

9.6.4 For thin-walled piers or spill-through abutments with ribbed stubs, wire fabric should be arranged on the surfaces of piers, the backwalls of the abutments, and the surfaces of the ribs. Their cross-sectional areas in the horizontal and vertical directions shall not be smaller than 250 mm^2/m (including primary reinforcement), and their spacing shall not be larger than 400 mm.

9.6.5 Concrete used for cap beams with a span-to-depth ratio no larger than 5 should have a high strength class, and shall not be lower than C25. Stirrups shall be arranged in cap beam sections. Their diameters shall not be smaller than 8 mm and the spacing should not be larger than 200 mm. Longitudinal reinforcement shall be arranged on two sides of the cap beams, their diameters should not be smaller than 12 mm, and the spacing should not be larger than 200 mm.

When lateral bracings are provided between pier columns, the sectional depth and width of the bracings may be 0.8 ~ 1.0 times and 0.6 ~ 0.8 times the diameter or long side of the columns, respectively. Longitudinal reinforcement with a diameter no less than 16 mm shall be arranged in four corners of the lateral bracings, and stirrups with a diameter no less than 8 mm shall also be arranged, the spacing of the stirrups shall not be larger than either the short sides of the lateral bracings or 400 mm.

9.6.6 When flexible bent piers are used in navigable rivers or rivers with flowing of a large number drifters, protection facilities should be provided for the piers at the upstream of the bridge.

9.6.7 In joints between cap beams and pier columns or between bracings and pier columns, longitudinal reinforcement in cap beams and bracings shall meet the following anchorage requirements:

1 Top joints of pier columns

1) For the upper longitudinal reinforcement in beams extends into joints, the development length shall conform to the requirements in Clause 9.1.4 of the *Specifications*, and shall extend over the center lines of the columns, the extended length should not be less than $5d$, in which d is the nominal diameter of the longitudinal reinforcement. If the cross-sectional dimensions of the columns do not satisfy the development requirements in Clause 9.1.4 of the *Specifications*, the upper longitudinal reinforcement in the beams may be anchored with 90° bend, in which the upper longitudinal reinforcement in the beams shall extend to the inner side of the longitudinal reinforcement that is on the outer sides of the column and bent towards the joints. The horizontal projection length of the horizontal reinforcement entered into the column inclusive of the bend arc shall not be smaller than $0.4l_a$, and the vertical projection length of vertical short reinforcement inclusive of the bend arc shall not be smaller than $15d$.

2) For the bottom longitudinal reinforcement in the beams extending into joints for anchorage, when the tensile strength of the reinforcement is fully developed, the method and length for anchorage of the reinforcement shall conform to the same provisions for the upper longitudinal reinforcement in beams. When the strength of the reinforcement is not used or only the compressive strength of the reinforcement is used, the development length extending into the joints shall conform to the provisions for the development of the bottom longitudinal reinforcement in beams at intermediate joints as specified in Item 2 of this clause.

2 Intermediate joints of pier columns

The upper longitudinal reinforcement in bracings shall pass through joints. The bottom longitudinal reinforcement in bracings should pass through joints. If the reinforcement need to be anchored, the following anchorage requirements shall be conformed to:

1) If the strength of the reinforcement is not considered, the development length embedded into joints shall not be smaller than $12d$ for deformed bars, and $15d$ for plain bars.

2) If the strength of the reinforcement is fully considered, the development length in intermediate joints shall satisfy the requirements in Clause 9.1.4 of the *Specifications*.

3) If the sectional dimension of the column is not large, the anchorage method with 90° bend specified in Item 1 of this clause should be adopted.

9.6.8 In joints between cap beams and pier columns, or between bracings and pier columns, longitudinal reinforcement in columns shall conform to the following anchorage requirements:

1) Longitudinal reinforcement in columns shall pass through the joints between bracings and pier columns, and the splices shall be arranged beyond the joint regions.

2) In joints between cap beams and pier columns, the longitudinal reinforcement in columns shall extend to the top of the cap beams, and the development length measured from the bottom of cap beams shall satisfy the requirements in Clause 9.1.4 of the *Specifications*. If the dimensions of cap beams are not large, the longitudinal reinforcement in columns may also be anchored by 90° bend. In this case, the longitudinal reinforcement in columns shall extend to the inner side of the upper longitudinal reinforcement of beams. The vertical projection length of the vertical short reinforcement entered into the beam inclusive of the bend arc shall not be smaller than $0.4l_a$, and the horizontal projection length of bent horizontal reinforcement inclusive of the bend arc shall not be smaller than $12d$.

9.6.9 Piers for box girder ramp bridges on highways should satisfy the following requirements:

1 Piers should have multiple supports for girders in the transverse direction (multi-column or single-column with double bearings), and the transverse spacing of bearings should be as large as possible. When mechanical behaviors of structures satisfy the design requirements, piers and girders may be rigidly connected.

2 If there are special construction conditions, for example, the pier located in a central divider of a roadway and must adopt single-column with one bearing for the bridge crossing over a roadway, single-column piers with one bearing shall not be used successively for several spans.

3 Reliable restrainers and unseating prevention structures should be provided at transition piers and abutments.

9.6.10 Detailing requirements for pile caps shall conform to not only the relevant provisions in the current *Specifications for Design of Foundation of Highway Bridges and Culverts* (JTG 3362) but also the following requirements:

1 The depth of pile caps should not be less than 1.5 times the pile diameter and also should not be less than 1.5 m.

2 If the center-to-center spacing of piles does not exceed 3 times the pile diameter, primary reinforcement shall be evenly arranged on the whole width of the pile cap; if the center-to-center spacing of piles exceeds 3 times the pile diameter, primary reinforcement shall be evenly arranged within the range 1.5 times the pile diameter distant from the pile center; and construction reinforcement with a reinforcement ratio no less than 0.1% shall be provided beyond this range. Transverse clear spacing and layer spacing of reinforcement shall conform to the provision in Clause 9.3.3 of the *Specifications*, and the minimum concrete cover shall conform to the provision in Clause 9.1.1 of the *Specifications*.

3 If the primary reinforcement is arranged in only one direction of the pile cap, construction reinforcement with a diameter no less than 12 mm and a spacing no larger than 250 mm shall be provided in the direction perpendicular to the primary reinforcement.

4 One layer of wire fabric should be arranged inside the bottom surface of the pile cap. The amount of reinforcement in each direction of the bottom surface should be 1200 ~ 1500 mm^2/m, and the diameter of the reinforcement should be 12 ~ 16 mm.

5 The diameter of vertical connection reinforcement in a pile cap shall not be less than 16 mm.

6 If the center-to-center spacing of piles in a pile cap is larger than or equal to 3 times the diameter of the piles, hanger reinforcement should be provided between two piles in a range with each end one pile diameter distant from each pile center (Figure 9.6.10). The diameter of the reinforcement shall not be smaller than 12 mm and the spacing shall not be larger than 200 mm.

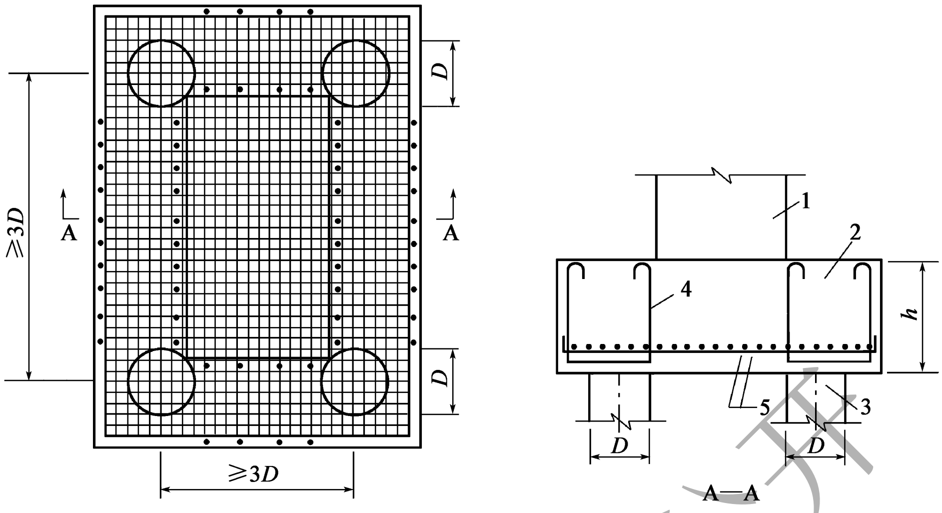


Figure 9.6.10 Arrangement of hanger reinforcement in pile cap

1-Pier; 2-Pile cap; 3-Pile; 4-Hanger reinforcement; 5-Primary reinforcement; D-Diameter of pile

9.7 Bearings and Expansion Joints

9.7.1 Plain laminated elastomeric bearings, polytetrafluorethylene (PTFE) sliding bearings, pot elastomeric bearings or spherical bearings should be selected for highway bridges according to the structural requirements. If there are special requirements, other types of bearings may be selected after verification by specific studies.

9.7.2 Elastomeric bearings shall be selected according to local temperature conditions. Neoprene bearings may be used in zones with a temperature of -25 ~ +60℃, and EPDM (Ethylene-Propylene-Diene Monomer) bearings or natural rubber bearings may be used in zones with a temperature of -40 ~ +60℃.

9.7.3 Arrangement of bearings shall satisfy the following requirements:

1 Double bearings should not be provided in the longitudinal direction for single support of beams.

2 When more than two bearings are used in the transverse direction, the adverse effects due to separation between bearings and the loading surfaces of structures shall be considered in the design.

9.7.4 Leveling measures shall be taken at the bottom of beams and/or the top of pier caps (cap beams) to keep the level of bearings.

9.7.5 Reliable restrainers shall be provided at the locations of movable bearings. Structures to prevent separation between bearings and the loading surface of structures should be provided for compression-only bearings.

9.7.6 Detailing of piers and abutments shall satisfy the requirements for inspection, maintenance and replacement of bearings. Space required for bearing replacement shall be preset between the top of the pier cap and the bottom of the main girder.

9.7.7 Modular joints, finger plate joints and flexible plug joints should be selected for highway bridges according to the structural requirements. If there are special requirements, other types of expansion joints may be selected after verification by specific studies.

9.8 Culverts, Lifting Loops and Hinges

9.8.1 Double-layer reinforcement shall be used in circular culverts with a span of 1 m or above. The minimum concrete cover over the reinforcement shall comply with the provisions in Clause 9.1.1 of the *Specifications*. All types of precast culvert members shall be checked for stresses during transport and installation.

9.8.2 Lifting loops in precast members shall be fabricated with HPB300 reinforcement, and cold-worked reinforcement must not be used. Each lifting loop is calculated as a two-leg section. The tensile stresses in lifting loops under the characteristic self-weight of the member shall not be larger than 65 MPa. When one member is provided with four lifting loops, it is considered in the design that only three lifting loops function simultaneously. The lifting loops shall be embedded into concrete with a length no less than 35 times their diameter. The embedded parts shall have a 180° bend at their ends and shall be welded or tied with the reinforcement in the members. The inner diameter of lifting loops shall not be smaller than 3 times the diameter of the reinforcement and shall not be smaller than 60 mm.

9.8.3 The convex radius of reinforced concrete hinges (Figure 8.6.1), r_1, should be 1.5 ~ 3.0 m. The concrete strength class for hinges shall not be lower than C30.

Appendix A
Practical Refined Analysis Models for Bridge Structures

A.1 General

A.1.1 The spatial grid model, folding surface grillage model, or single beam model with 7 degrees of freedom in this appendix should be adopted in practical refined analysis models for bridge structures.

A.1.2 Concrete box girders whose spacing of the web is not less than 5 m should be analyzed by spatial grid model.

A.1.3 Prefabricated multi-beam bridges or multi-cell concrete box girders should be analyzed by a folding surface grillage model.

A.1.4 Concrete box girder bridges with a curved alignment should be analyzed by a single beam model with 7 degrees of freedom.

A.2 Application Principles

A.2.1 The spatial grid model should satisfy the following requirements:

1 The width of each beam element on the section, b_n, is not larger than 2 m, and the flange width of the I-section beam element, b_f, is not larger than $6h_f$ (Figure A.2.1).

2 A web with prestressing tendons should be meshed into one beam element (Figure A.2.1). When prestressing steel curved in-plane transversely passes through multiple beam elements

of top and bottom slabs, the prestressing effect should be taken into account in those elements with the longest distance that the prestressing steel passes through.

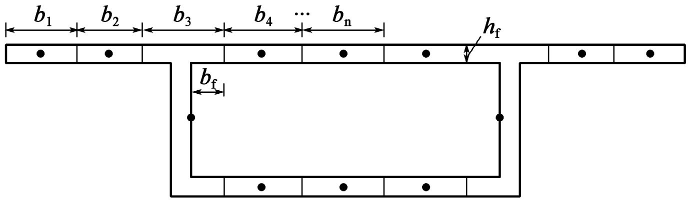


Figure A.2.1 Diagram of spatial grid model

A.2.2 Folding surface grillage model should satisfy the following requirements:

1 The width of each beam element on the section, b_n, should not larger than 3 m, and the flange width of the I-section beam element, b_f, should not larger than $6h_f$ (Figure A.2.2).

2 A web with prestressing tendons should be meshed into one beam element (Figure A.2.2). When prestressing steel curved in-plane transversely passes through multiple beam elements of top and bottom slabs, the prestressing effect should be taken into account in those elements with the longest distance that the prestressing steel passes through.

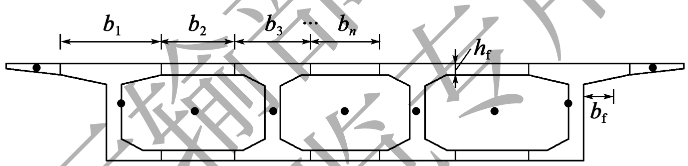


Figure A.2.2 Diagram of folding surface grillage model

A.2.3 Single beam model with 7 degrees of freedom should satisfy the following requirements:

1 For the single beam model with 7 degrees of freedom, the shear lag effect may be considered based on the effective distribution width specified by the provisions in Clause 4.3.4 in the *Specifications*.

2 The amplification factors of normal stress λ_σ and shear stress λ_τ should be calculated by the following Eq (A.2.3-1) and Eq (A.2.3-2) based on the calculated stresses obtained from the analysis using single beam model with 7 degrees of freedom, respectively:

$$\lambda_\sigma = \frac{\sigma_M + \sigma_W}{\sigma_M} \tag{A.2.3-1}$$

$$\lambda_\tau = \frac{\tau_M + \tau_K + \tau_W}{\tau_M} \tag{A.2.3-2}$$

where：

σ_M, τ_M—normal stress and shear stress induced by bending, respectively;

σ_W, τ_W—normal stress and shear stress induced by warping torsion, respectively;

τ_K—shear stress induced by St. Venant torsion.

Appendix B
Analysis Method Using Strut-and-Tie Model

B.1 General

B.1.1 The range of the disturbed regions of concrete bridges should be determined according to the Saint-Venant's principle.

B.1.2 The disturbed regions of concrete bridges may be simplified using the strut-and-tie model in accordance with the following steps:

1 Determine the range of the disturbed regions according to the boundary conditions and stresses of the structures, see Clause B.1.1 of the *Specifications*.

2 Develop the strut-and-tie model in the disturbed regions, see Section B.2 of the *Specifications*.

3 Calculate the design internal forces of each member in the model, according to the design forces on the boundary of the disturbed regions and the stress equilibrium conditions of the strut-and-tie model.

4 Check the strength of ties, struts, and nodes, see Section B.3 of the *Specifications*. Verify if the selected material strength, reinforcement and configuration dimensions for the ties, struts, and nodes can meet the requirements.

5 If the verification fails, the configuration and even structural dimensions of the strut-and-tie model need to be adjusted, and the above steps shall be repeated; if the verification passes, detailed reinforcement design shall be carried out according to the structural

requirements, and the structural distribution of reinforcements on the surface of the member shall meet the provisions of Clause B. 3. 5 of the *Specifications*.

B. 2 Development Method

B. 2. 1 The strut-and-tie model shall satisfy the force equilibrium conditions and correctly reflect the transfer characteristics of the stress flow inside the concrete structures.

B. 2. 2 The strut-and-tie model may be developed using the load path method, stress trace method, stress fluent-line method, criterion of minimum strain energy, criterion of maximum stress, etc.

B. 2. 3 In the strut-and-tie model, the minimum angle between the ties and struts should not be smaller than 25°.

B. 3 Checking Content

B. 3. 1 The resistance of ties, struts, and nodes in the strut-and-tie model shall be checked in accordance with Eq. (B. 3. 1).

$$\gamma_0 S_{\mathrm{STM,d}} \leq R_{\mathrm{STM,d}} \tag{B. 3. 1}$$

where:

$S_{\mathrm{STM,d}}$—design action effect of strut-and-tie model, which is denoted as $S_{\mathrm{T,d}}$, $S_{\mathrm{S,d}}$, and $S_{\mathrm{N,d}}$ for ties, struts and nodes, respectively;

$R_{\mathrm{STM,d}}$—design resistance of strut-and-tie model, which is denoted as $R_{\mathrm{T,d}}$, $R_{\mathrm{S,d}}$, and $R_{\mathrm{N,d}}$ for ties, struts and nodes, respectively.

B. 3. 2 Reinforcing or prestressing steel shall be arranged in the tie, and the design resistance of the tie shall be calculated by Eq. (B. 3. 2):

$$R_{\mathrm{T,d}} = f_{\mathrm{sd}} A_{\mathrm{s}} + f_{\mathrm{pd}} A_{\mathrm{p}} \tag{B. 3. 2}$$

where:

f_{sd}—design tensile strength of reinforcing steel;

f_{pd}—design tensile strength of prestressing steel;

A_{s}—cross-sectional area of reinforcing steel in the tie;

A_{p}—cross-sectional area of prestressing steel in the tie.

B. 3. 3 The design resistance of a concrete strut shall be calculated in accordance with the following provisions:

1 For an unreinforced concrete strut:

$$R_{\mathrm{S,d}} = \mathrm{f}_{ce,d}A_{\mathrm{cs}} \tag{B.3.3-1}$$

$$f_{\mathrm{ce,d}} = \frac{\beta_{\mathrm{c}} f_{\mathrm{cd}}}{0.8 + 170\varepsilon_1} \leqslant 0.85\beta_{\mathrm{c}} f_{\mathrm{cd}} \tag{B.3.3-2}$$

$$\varepsilon_1 = \varepsilon_{\mathrm{s}} + (\varepsilon_{\mathrm{s}} + 0.002)\cot^2\theta_{\mathrm{s}} \tag{B.3.3-3}$$

where:

A_{cs}—effective cross-sectional area of a concrete strut, which is determined in accordance with Figure B.3.3-1, Figure B.3.3-2, or Figure B.3.3-3;

$f_{\mathrm{ce,d}}$—design equivalent compressive strength of concrete strut;

f_{cd}—design axial compressive strength of concrete;

β_{c}—parameter related to the concrete strength class, which is taken as 1.30 for C25 ~ C50 and 1.35 for C55 ~ C80;

ε_1—tensile strain of concrete in the direction perpendicular to the strut;

θ_{s}—minimum angle between strut and adjacent tie, and $\theta_{\mathrm{s}} \geqslant 25°$;

ε_{s}—tensile strain of reinforcing steel in the direction of the tie. If the tie is composed of reinforcing steel, it is calculated using the design internal force under the action combination in the tie. If the tie is composed of prestressing steel, it is taken as $\varepsilon_{\mathrm{s}} = 0.0$ and $\varepsilon_{\mathrm{s}} = (f_{\mathrm{pd}} - f_{\mathrm{pe}})/E_{\mathrm{p}}$ before and after decompression of surrounding concrete, respectively.

2 For a reinforced concrete strut:

$$R_{\mathrm{S,d}} = f_{\mathrm{ce,d}}A_{\mathrm{cs}} + f'_{\mathrm{sd}}A_{\mathrm{ss}} \tag{B.3.3-4}$$

where:

$f_{\mathrm{ce,d}}$—design equivalent compressive strength of concrete strut;

A_{cs}—effective cross-sectional area of concrete strut;

f'_{sd}—design compressive strength of reinforcing steel;

A_{ss}—projected area along the strut's axis for reinforcing steel passing through the strut.

B.3.4 The design resistance of nodes shall be calculated by Eq. (B.3.4):

$$R_{\mathrm{N,d}} = \beta_{\mathrm{n}} f_{\mathrm{cd}} A_{\mathrm{n}} \tag{B.3.4}$$

where:

β_{n}—softening coefficient of concrete strength on the interface of the node, which is taken from Table B.3.4;

A_{n}—cross-sectional area on the interface of the node, which is calculated in accordance with different types of nodes in Figure B.3.3-1, Figure B.3.3-2, or Figure B.3.3-3.

Table B. 3. 4 Softening coefficient of concrete strength on the interface of three types of typical node

Type of node	Meaning	β_n
CCC (compression-compression-compression)	Zone of node surrounded by member and supporting surfaces	$0.85\beta_c$
CCT (compression-compression-tension)	Zone of node forthe anchorage of a one-way tie	$0.75\beta_c$
CTT (compression-tension-tension)	Zone of node forthe anchorage of a two-way tie	$0.65\beta_c$

Notes: When the confined reinforcementis arranged in the node zone, the value in this table may be increased if the confined enhancement effect is verified by analysis or test.

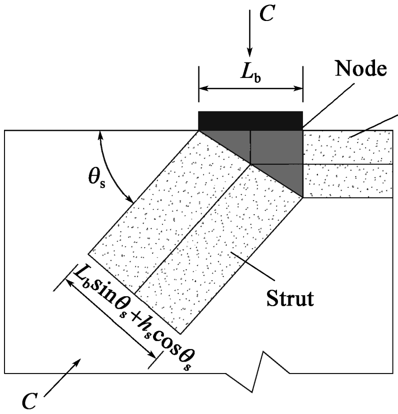


Figure B. 3. 3-1 Strut and zone of CCC node under concentrated load

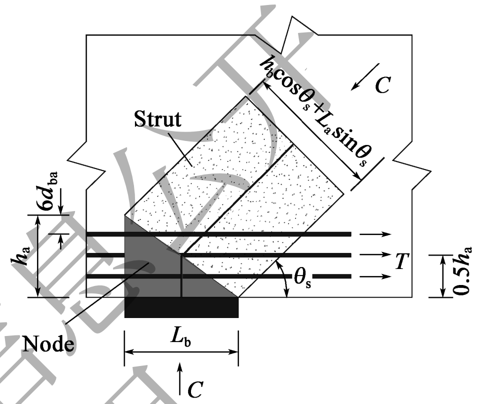
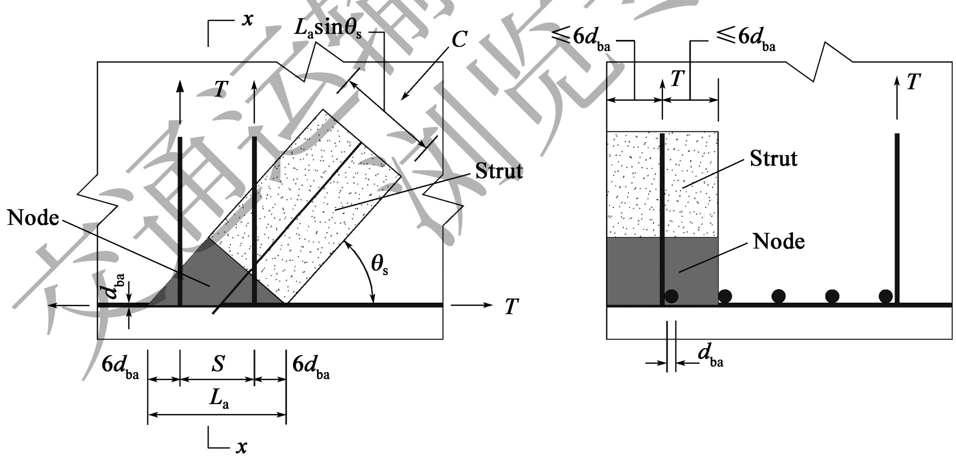


Figure B. 3. 3-2 Strut and zone of CCT node formed by the reinforcement and supporting restriction



a)Dimensions of strut and node zone b)Thickness of strut and node shown in section *x-x*

Figure B. 3. 3-3 Strut and zone of CTT node formed by the reinforcement anchorage

In the above figures:

d—diameter of tension reinforcement in the zone of a node;

S—spacing of reinforcement anchored at the end of the strut;

L_a—effective length of node anchored by reinforcement;

L_b—width of supporting plate;

h_a—height of node confined by reinforcement;

h_s—height of node confined by strut;

θ_s—inclined angle of strut.

B.3.5 In the disturbed region represented by the strut-and-tie model, the orthogonal wire fabric shall be arranged on its surface. The spacing of wire fabric shall not be larger than 300 mm, and the ratio of the reinforcement area to the gross cross-sectional area of concrete in each direction shall not be less than 0.3%.

Appendix C
Concrete Shrinkage Strain and Creep Coefficient Calculation, Ratio of Median to Ultimate Values of Prestress Loss due to Prestressing Steel Relaxation

C.1 Shrinkage Strain

C.1.1 The shrinkage strain of concrete may be calculated by the following equations:

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cs0} \cdot \beta_s(t - t_s) \tag{C.1.1-1}$$

$$\varepsilon_{cs0} = \varepsilon_s(f_{cm}) \cdot \beta_{RH} \tag{C.1.1-2}$$

$$\varepsilon_s(f_{cm}) = [160 + 10\beta_{sc}(9 - f_{cm}/f_{cm0})] \cdot 10^{-6} \tag{C.1.1-3}$$

$$\beta_{RH} = 1.55[1 - (RH/RH_0)^3] \tag{C.1.1-4}$$

$$\beta_s(t - t_s) = \left[\frac{(t - t_s)/t_1}{350(h/h_0)^2 + (t - t_s)/t_1}\right]^{0.5} \tag{C.1.1-5}$$

where:

t—concrete age being considered for calculation (d);

t_s—concrete age at the initial time of shrinkage (d), which may be assumed to be 3 ~7 days;

$\varepsilon_{cs}(t, t_s)$—shrinkage strain from the initial time of shrinkage t_s to the time being considered for calculation t;

ε_{cs0}—notional shrinkage coefficient;

β_s—coefficient to describe the development of shrinkage with time;

f_{cm}—average cylinder compressive strength at the age of 28 days for concrete strength class between C25 and C50 (MPa), $f_{cm} = 0.8f_{cu,k} + 8$ MPa;

$f_{cu,k}$—characteristic compressive strength of concrete cube with 95 percent level of confidence at the age of 28 days (MPa);

β_{RH}—coefficient related to the annual mean relative humidity; Eq. (C.1.1-4) is applicable for $40\% \leqslant RH < 99\%$;

RH—annual average relative humidity of the ambient environment (%);

β_{sc}—coefficient depending on the type of cement, $\beta_{sc} = 5.0$ for ordinary Portland cement or rapid hardening cement;

h—theoretical thickness of member (mm), $h = 2A/u$, in which A is the sectional area of the member, u is the perimeter of the member in contact with the atmosphere.

$RH_0 = 100\%$; $h_0 = 100$ mm; $t_1 = 1$d; $f_{cm0} = 10$ MPa.

C.1.2 The notional shrinkage coefficient ε_{cs0} for the concrete strength class between C25 and C50 may be taken from the value listed in Table C.1.2, which is calculated in accordance with Eq. (C.1.1-2).

Table C.1.2 Notional shrinkage coefficient of concrete ε_{cs0} ($\times 10^{-3}$)

$40\% \leqslant RH < 70\%$	$70\% \leqslant RH < 99\%$
0.529	0.310

Notes: 1. This table is applicable for the concrete where ordinary Portland cement or rapid – hardening cement is used.

2. This table is applicable for the concrete on enviroment with the mean seasonal temperature variation of −20 ~ +40℃.

3. For concrete with a strength class of C50 or above, the value listed in the table shall be multiplied by $\sqrt{\frac{32.4}{f_{ck}}}$, where f_{ck} is the characteristic axial compressive strength of concrete (MPa).

C.1.3 In the bridge design, when the shrinkage effects need to be considered in the analysis or the losses of prestress in corresponding stages are calculated, the shrinkage strain of concrete may be calculated in accordance with the following steps:

1 Calculate by Eq. (C.1.1-5) the coefficients of shrinkage strain development from t_s to t and from t_s to t_0, $\beta_s(t - t_s)$ and $\beta_s(t_0 - t_s)$, respectively. Herein, t is the concrete age (d) being considered for calculation of the shrinkage strain, t_0 is the concrete age (d) at the moment when the bridge structure begins to be affected by shrinkage or at the moment when the prestressing steel is anchored, t_s is the concrete age at the initial time of shrinkage (at the end of the curing period) and may be taken as 3 ~ 7 d in design, $t > t_0 \geqslant t_s$.

2 The shrinkage strain $\varepsilon_{cs}(t, t_0)$ from t_0 to t is calculated by the following equation:

$$\varepsilon_{cs}(t, t_0) = \varepsilon_{cs0}[\beta_s(t - t_s) - \beta_s(t_0 - t_s)] \tag{C.1.3}$$

where the notional shrinkage coefficient ε_{cs0} is taken from Table C.1.2.

C.2 Creep Coefficient

C.2.1 The creep coefficient of concrete may be calculated by the following equations:

$$\phi(t,\ t_0) = \phi_0 \cdot \beta_c(t - t_0) \tag{C.2.1-1}$$

$$\phi_0 = \phi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) \tag{C.2.1-2}$$

$$\phi_{RH} = 1 + \frac{1 - RH/RH_0}{0.46\ (h/h_0)^{\frac{1}{3}}} \tag{C.2.1-3}$$

$$\beta(f_{cm}) = \frac{5.3}{(f_{cm}/f_{cm0})^{0.5}} \tag{C.2.1-4}$$

$$\beta(t_0) = \frac{1}{0.1 + (t_0/t_1)^{0.2}} \tag{C.2.1-5}$$

$$\beta_c(t - t_0) = \left[\frac{(t - t_0)/t_1}{\beta_H + (t - t_0)/t_1}\right]^{0.3} \tag{C.2.1-6}$$

$$\beta_H = 150\left[1 + \left(1.2\frac{RH}{RH_0}\right)^{18}\right]\frac{h}{h_0} + 250 \leqslant 1500 \tag{C.2.1-7}$$

Where:

t_0—concrete age when the load is applied (d);

t—concrete age being considered for calculation (d);

$\phi(t,\ t_0)$—creep coefficient for concrete age from t_0 to t;

ϕ_0—notional creep coefficient;

β_c—coefficient to describe the development of creep with time after loading.

In the equations, the meanings of f_{cm}, f_{cm0}, RH, RH_0, h, h_0, and t_1, as well as their values are the same as those in Clause C.1.1.

C.2.2 The notional creep coefficient ϕ_0 for concrete with a strength class between C25 and C50 may be taken from Table C.2.2, which is calculated in accordance with Eq. (C.2.1-2).

Table C.2.2 Notional creep coefficient of concrete ϕ_0

Loading age t_0 (d)	$40\% \leqslant RH < 70\%$				$70\% \leqslant RH < 99\%$			
	Theoretical thickness h (mm)				Theoretical thickness h (mm)			
	100	200	300	≥600	100	200	300	≥600
3	3.90	3.50	3.31	3.03	2.83	2.65	2.56	2.44
7	3.33	3.00	2.82	2.59	2.41	2.26	2.19	2.08
14	2.92	2.62	2.48	2.27	2.12	1.99	1.92	1.83
28	2.56	2.30	2.17	1.99	1.86	1.74	1.69	1.60
60	2.21	1.99	1.88	1.72	1.61	1.51	1.46	1.39
90	2.05	1.84	1.74	1.59	1.49	1.39	1.35	1.28

Notes: 1. This table is applicable for the concrete where ordinary Portland cement or rapid-hardening cement is used.

2. This table is applicable for the concrete exposed on enviroment with mean seasonal temperature variation of −20 ~ +40℃.

3. For concrete with a strength class of C50 or above, the value listed in the table shall be multiplied by $\sqrt{\frac{32.4}{f_{ck}}}$, where f_{ck} is the characteristic axial compressive strength of concrete (MPa).

4. If the actual theoretical thickness of a member and loading age are between the values listed in the table, the notional creep coefficient is calculated by linear interpolation.

C. 2. 3 In the bridge design, when the creep effects need to be considered in the analysis or the losses of prestress in corresponding stages are calculated, the creep coefficient of concrete may be calculated in accordance with the following steps:

1 Calculate β_H by Eq. (C. 2. 1-7), in which the annual mean relative humidity RH is taken as $RH = 55\%$ when $40\% \leqslant RH < 70\%$, and is taken as $RH = 80\%$ when $70\% \leqslant RH < 99\%$.

2 Calculate the coefficient of creep development $\beta_c(t - t_0)$ by Eq. (C. 2. 1-6) according to the concrete age being considered for calculation t, concrete age at loading t_0, and the calculated β_H.

3 Calculate the creep coefficient $\phi(t - t_0)$ by Eq. (C. 2. 1-1), according to the $\beta_c(t - t_0)$ and the notional creep coefficient listed in Table C. 2. 2 (calculated by linear interpolation if necessary).

Notes: When the actual loading age exceeds 90 d as given in Table C. 2. 2, the notional creep coefficient of concrete may be calculated by $\phi_0' = \phi_0 \beta(t_0') / \beta(t_0)$, in which ϕ_0 is the notional creep coefficient listed in Table C. 2. 2, $\beta(t_0')$ and $\beta(t_0)$ are calculated by Eq. (C. 2. 1-5), where t_0 is the loading age listed in the table, and t_0' is the actual loading age which is longer than 90 d.

C. 2. 4 For concrete that the fly ash is used in the mixture (fly ash concrete), its creep coefficient should be obtained by the creep test to reflect the component characteristics of the concrete material. In the absence of sufficient test data, its creep coefficient may be calculated by Eq. (C. 2. 4):

$$\phi(t, t_0) = \phi(\alpha, t_0) \cdot \phi_0 \cdot \beta_c(t - t_0) \qquad (C. 2. 4)$$

where $\phi(\alpha, t_0)$ is the nominal creep correction coefficient of fly ash concrete, which is taken from Table C. 2. 4 in accordance with the content of fly ash, α, and the loading age t_0; and ϕ_0 and $\beta_c(t - t_0)$ shall be calculated by Clause C. 2. 1.

Table C. 2. 4 Nominal creep correction coefficient of fly ash concrete $\phi(\alpha, t_0)$

Loading age t_0 (d)	Content α (%)		
	10	20	30
7	0.80	0.65	0.53
14	0.70	0.55	0.45
28	0.64	0.50	0.41
60	0.60	0.47	0.38
90	0.58	0.46	0.37

Notes: 1. The content α is the mass percentage.

2. If the content of fly ash and loading age are between the values in the table, the correction coefficient may be taken by linear interpolation.

C.3 Ratio of Median to Ultimate Values of Prestress Loss Due to Prestressing

C.3.1 When the loss of prestress due to prestressing steel relaxation needs to be calculated stage by stage, its median value shall be determined by Table 3.1 in accordance with the time the prestress comes into effect. The ultimate loss of prestress due to prestressing steel relaxation shall be calculated by Clause 6.2.6.

Table C.3.1 Ratio of median to ultimate values of prestress loss due to prestressing steel relaxation

Time (d)	2	10	20	30	40
Ratio	0.50	0.61	0.74	0.87	1.00

Appendix D
Calculation Formulas for Temperature Actions

D.0.1 The temperature gradient-induced stress for a simply – supported beam shall be calculated by the following equations:

$$N_t = \sum A_y t_y \alpha_c E_c \tag{D.0.1-1}$$

$$M_t^0 = -\sum A_y t_y \alpha_c E_c e_y \tag{D.0.1-2}$$

1) The stress induced by positive temperature gradient

$$\sigma_t = -\frac{N_t}{A_0} + \frac{M_t^0}{I_0} y + t_y \alpha_c E_c \tag{D.0.1-3}$$

2) The stress induced by negative temperature gradient shall be calculated by Eq. (D.0.1-3), in which t_y shall be taken as a negative value in Eq. (D.0.1-1) ~ Eq. (D.0.1-3).

In the above formulas:

A_y—area of element inside the section;

t_y—mean value of temperature gradient in element with an area of A_y, all are substituted by positive values;

α_c—coefficient of linear expansion of concrete, which shall be taken from *General Specifications for Design of Highway Bridges and Culverts* (JTG D60—2015);

E_c—modulus of elasticity of concrete;

y—distance from the point of calculated stress to the gravity axis of the transformed section, which is positive above the gravity axis and negative below the gravity axis;

e_y—distance from the centroid of the element's area A_y to the gravity axis of the transformed section, which is positive above the gravity axis and negative below the gravity axis;

A_0, I_0—area and inertia moment of the transformed section, respectively.

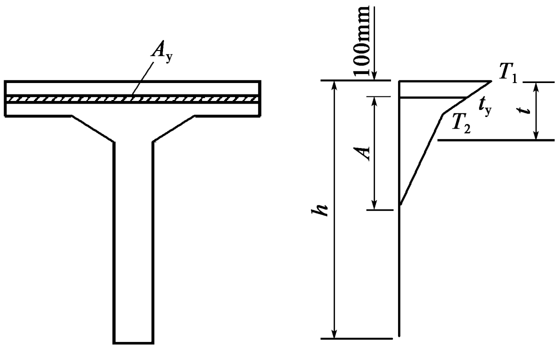


Figure D. 0. 1 Calculation of temperature gradient

D. 0. 2 In the calculation of stress induced by temperature difference for continuous beams, the secondary bending moment M'_t induced by temperature shall also be taken into account. Herein, the moment M_t^0 in the second term on the right – hand side of Eq. (D. 0. 1 – 3) shall be substituted by $M_t = M'_t + M_t^0$.

Appendix E
Simplified Calculation Formula for Effective Length of Compression Member

E.0.1 Effective length of a compression member, l_0, should be calculated by Eq. (E.0.1):

$$l_0 = kl \tag{E.0.1}$$

where:

k—conversion factor for effective length, which may be calculated by experience or in accordance with the equations in Clause E.0.2 or Clause E.0.3 of the *Specifications*. For members with two fixed ends, k may be taken as 0.5. For members with one fixed end and the other end rotation free while translation fixed, k may be taken as 0.7. For members with two ends rotation free while translation fixed, k may be taken as 1.0. For members with a fixed end and a free end, k may be taken as 2.0;

l—length between two supports of a member.

E.0.2 For a member with one fixed end and the other end of both rotational and translational elastic restraints (Figure E.0.2), the conversion factor for effective length k may be determined by the following equations:

$$k = 0.5\exp\left[\frac{0.35}{1+0.6k_\theta} + \frac{0.7}{1+0.01k_F^2} + \frac{0.35}{(1+0.75k_\theta)(1+1.15k_F)}\right] \tag{E.0.2-1}$$

$$k_\theta = K_\theta \frac{l}{EI} \tag{E.0.2-2}$$

$$k_F = K_F \frac{l^3}{EI} \tag{E.0.2-3}$$

where:

k_θ—coefficient of relative rotational restraint stiffness at the end with rotational and translational elastic restraints;

K_θ—rotational restraint stiffness at the end with rotational and translational elastic restraints;

k_F—coefficient of relative translational restraint stiffness at the end with rotational and translational elastic restraints;

K_F—translational restraint stiffness at the end with rotational and translational elastic restraints;
l—length between supports of the member;
EI—sectional flexural stiffness of the member.

E. 0. 3 For a member with one fixed end and the other end of only elastic translational restraint (Figure E. 0. 3), the conversion factor for effective length may be determined by Eq. (E. 0. 3):

$$k = 2 - \frac{1.3k_F^{1.5}}{9.5 + k_F^{1.5}} \tag{E.0.3}$$

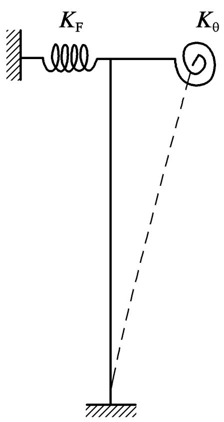


Figure E. 0. 2 Member with one fixed end and the other end of both rotational and translational elastic restraints

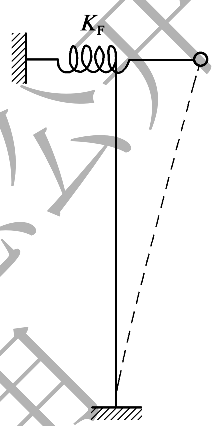


Figure E. 0. 3 Member with one fixed end and the other end of only elastic translational restraint

Appendix F
Calculation for Compressive Resistance of an Eccentrically Loaded Reinforced Concrete Compression Member of Circular Cross-section with Longitudinal Reinforcement Evenly Distributed along the Perimeter

F. 0. 1 For an eccentrically loaded reinforced concrete circular compression member with longitudinal reinforcement evenly distributed along the perimeter, when the concrete with a strength class between C30 and C50 and the longitudinal reinforcement ratio is between 0.5% and 4%, the compressive resistance of the cross – section shall be calculated in accordance with the following requirements:

$$\gamma_0 N_d \leqslant n_u A f_{cd} \tag{F.0.1}$$

where:

γ_0—importance factor of structure;

N_d—design axial force of the member;

n_u—relative compressive resistance of the member, which is determined by Figure F.0.1;

A—cross-sectional area of the member;

f_{cd}—design compressive strength of concrete.

Figure F.0.1 Relative compressive resistance of eccentrically loaded reinforced concrete compression member with circular cross-section, n_u

$\eta \frac{e_0}{r}$	$\rho \frac{f_{sd}}{f_{cd}}$																	
	0.06	0.09	0.12	0.15	0.18	0.21	0.24	0.27	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.10	1.20
0.01	1.0487	1.0783	1.1079	1.1375	1.1671	1.1968	1.2264	1.2561	1.2857	1.3846	1.4835	1.5824	1.6813	1.7802	1.8791	1.9780	2.0769	2.1758
0.05	1.0031	1.0316	1.0601	1.0885	1.1169	1.1454	1.1738	1.2022	1.2306	1.3254	1.4201	1.5148	1.6095	1.7042	1.7989	1.8937	1.9884	2.0831
0.10	0.9438	0.9711	0.9984	1.0257	1.0529	1.0802	1.1074	1.1345	1.1617	1.2521	1.3423	1.4325	1.5226	1.6127	1.7027	1.7927	1.8826	1.9726

续上表

$\eta\frac{e_0}{r}$	$\rho\frac{f_{sd}}{f_{cd}}$																
	0.06	0.09	0.12	0.15	0.18	0.21	0.24	0.27	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.10	1.20
0.15	0.8827	0.9090	0.9352	0.9614	0.9875	1.0136	1.0396	1.0656	1.0916	1.1781	1.2643	1.3503	1.4362	1.5220	1.6077	1.6934	1.7790	1.8646
0.20	0.8206	0.8458	0.8709	0.8960	0.9210	0.9460	0.9709	0.9958	1.0206	1.1033	1.1856	1.2677	1.3496	1.4313	1.5130	1.5945	1.6760	1.7574
0.25	0.7589	0.7829	0.8067	0.8302	0.8540	0.8778	0.9016	0.9254	0.9491	1.0279	1.1063	1.1845	1.2625	1.3404	1.4180	1.4956	1.5731	1.6504
0.30	0.7003	0.7247	0.7486	0.7721	0.7953	0.8181	0.8408	0.8632	0.8855	0.9590	1.0316	1.1036	1.1752	1.2491	1.3228	1.3964	1.4699	1.5433
0.35	0.6432	0.6684	0.6928	0.7165	0.7397	0.7625	0.7849	0.8070	0.8290	0.9008	0.9712	1.0408	1.1097	1.1783	1.2465	1.3145	1.3824	1.4500
0.40	0.5878	0.6142	0.6393	0.6635	0.6869	0.7097	0.7320	0.7540	0.7757	0.8461	0.9147	0.9822	1.0489	1.1150	1.1807	1.2461	1.3113	1.3762
0.45	0.5346	0.5624	0.5884	0.6132	0.6369	0.6599	0.6822	0.7041	0.7255	0.7949	0.8619	0.9275	0.9921	1.0561	1.1195	1.1825	1.2452	1.3077
0.50	0.4839	0.5133	0.5403	0.5657	0.5898	0.6130	0.6354	0.6573	0.6786	0.7470	0.8126	0.8765	0.9393	1.0012	1.0625	1.1233	1.1838	1.2441
0.55	0.4359	0.4670	0.4951	0.5212	0.5458	0.5692	0.5917	0.6135	0.6347	0.7022	0.7666	0.8289	0.8899	0.9500	1.0094	1.0682	1.1266	1.1848
0.60	0.3910	0.4238	0.4530	0.4798	0.5047	0.5283	0.5509	0.5727	0.5938	0.6605	0.7237	0.7846	0.8440	0.9023	0.9598	1.0168	1.0733	1.1295
0.65	0.3495	0.3840	0.4141	0.4414	0.4667	0.4905	0.5131	0.5348	0.5558	0.6217	0.6837	0.7432	0.8011	0.8578	0.9136	0.9689	1.0236	1.0779
0.70	0.3116	0.3475	0.3784	0.4062	0.4317	0.4556	0.4782	0.4998	0.5206	0.5857	0.6466	0.7047	0.7611	0.8163	0.8705	0.9241	0.9771	1.0297
0.75	0.2773	0.3143	0.3459	0.3739	0.3996	0.4235	0.4460	0.4674	0.4881	0.5523	0.6120	0.6689	0.7239	0.7776	0.8303	0.8823	0.9337	0.9847
0.80	0.2468	0.2845	0.3164	0.3446	0.3702	0.3940	0.4164	0.4377	0.4581	0.5214	0.5799	0.6356	0.6892	0.7415	0.7927	0.8432	0.8931	0.9426
0.85	0.2199	0.2579	0.2899	0.3180	0.3436	0.3672	0.3893	0.4104	0.4305	0.4928	0.5502	0.6045	0.6569	0.7078	0.7577	0.8067	0.8552	0.9032
0.90	0.1963	0.2343	0.2661	0.2940	0.3193	0.3427	0.3646	0.3853	0.4051	0.4663	0.5225	0.5757	0.6267	0.6763	0.7249	0.7726	0.8197	0.8663
0.95	0.1759	0.2134	0.2448	0.2724	0.2974	0.3204	0.3420	0.3624	0.3818	0.4419	0.4969	0.5488	0.5986	0.6470	0.6942	0.7406	0.7864	0.8317
1.00	0.1582	0.1950	0.2259	0.2530	0.2775	0.3001	0.3213	0.3413	0.3604	0.4193	0.4731	0.5238	0.5724	0.6195	0.6655	0.7107	0.7553	0.7993
1.10	0.1299	0.1646	0.1939	0.2198	0.2433	0.2649	0.2852	0.3044	0.3227	0.3791	0.4305	0.4789	0.5251	0.5699	0.6136	0.6564	0.6986	0.7402
1.20	0.1087	0.1410	0.1685	0.1929	0.2152	0.2358	0.2551	0.2734	0.2909	0.3446	0.3937	0.4398	0.4838	0.5264	0.5679	0.6086	0.6486	0.6881
1.30	0.0927	0.1224	0.1481	0.1710	0.1920	0.2115	0.2299	0.2472	0.2639	0.3150	0.3618	0.4057	0.4476	0.4882	0.5276	0.5663	0.6043	0.6418
1.40	0.0804	0.1077	0.1316	0.1531	0.1728	0.1912	0.2086	0.2250	0.2408	0.2895	0.3340	0.3759	0.4158	0.4544	0.4920	0.5288	0.5649	0.6006
1.50	0.0708	0.0959	0.1180	0.1381	0.1567	0.1741	0.1905	0.2061	0.2210	0.2673	0.3097	0.3496	0.3877	0.4245	0.4603	0.4954	0.5298	0.5638
1.60	0.0630	0.0862	0.1068	0.1256	0.1431	0.1595	0.1750	0.1897	0.2039	0.2479	0.2884	0.3264	0.3628	0.3979	0.4321	0.4655	0.4984	0.5309
1.70	0.0567	0.0782	0.0974	0.1150	0.1315	0.1469	0.1616	0.1756	0.1891	0.2310	0.2695	0.3058	0.3405	0.3741	0.4068	0.4387	0.4702	0.5012
1.80	0.0515	0.0714	0.0894	0.1060	0.1215	0.1361	0.1500	0.1633	0.1761	0.2160	0.2528	0.2875	0.3207	0.3528	0.3840	0.4146	0.4447	0.4743
1.90	0.0472	0.0657	0.0826	0.0982	0.1128	0.1266	0.1398	0.1525	0.1646	0.2027	0.2378	0.2710	0.3028	0.3335	0.3635	0.3928	0.4216	0.4500
2.00	0.0435	0.0608	0.0767	0.0914	0.1052	0.1183	0.1309	0.1429	0.1545	0.1908	0.2244	0.2562	0.2867	0.3162	0.3449	0.3730	0.4007	0.4279
2.50	0.0311	0.0441	0.0562	0.0676	0.0784	0.0888	0.0987	0.1083	0.1176	0.1470	0.1744	0.2005	0.2255	0.2498	0.2735	0.2968	0.3197	0.3422
3.00	0.0241	0.0345	0.0442	0.0535	0.0623	0.0707	0.0789	0.0869	0.0946	0.1191	0.1421	0.1640	0.1852	0.2057	0.2258	0.2456	0.2650	0.2841
3.50	0.0197	0.0283	0.0364	0.0441	0.0516	0.0587	0.0657	0.0724	0.0790	0.0999	0.1196	0.1385	0.1568	0.1746	0.1919	0.2090	0.2258	0.2425
4.00	0.0166	0.0240	0.0309	0.0376	0.0440	0.0502	0.0562	0.0620	0.0677	0.0859	0.1032	0.1198	0.1358	0.1514	0.1667	0.1818	0.1966	0.2112
4.50	0.0144	0.0208	0.0269	0.0327	0.0383	0.0437	0.0490	0.0542	0.0592	0.0754	0.0907	0.1054	0.1197	0.1336	0.1473	0.1607	0.1740	0.1870
5.00	0.0127	0.0183	0.0237	0.0289	0.0339	0.0388	0.0435	0.0481	0.0526	0.0671	0.0809	0.0941	0.1070	0.1195	0.1319	0.1440	0.1559	0.1677
5.50	0.0113	0.0164	0.0213	0.0259	0.0304	0.0348	0.0391	0.0433	0.0474	0.0605	0.0729	0.0850	0.0967	0.1081	0.1193	0.1304	0.1412	0.1520
6.00	0.0102	0.0149	0.0193	0.0235	0.0276	0.0316	0.0355	0.0393	0.0430	0.0550	0.0664	0.0775	0.0882	0.0987	0.1089	0.1191	0.1291	0.1390

续上表

$\eta \frac{e_0}{r}$	$\rho \frac{f_{sd}}{f_{cd}}$																	
	0.06	0.09	0.12	0.15	0.18	0.21	0.24	0.27	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.10	1.20
6.50	0.0093	0.0136	0.0176	0.0215	0.0252	0.0289	0.0325	0.0360	0.0394	0.0504	0.0610	0.0711	0.0810	0.0907	0.1002	0.1096	0.1188	0.1280
7.00	0.0086	0.0125	0.0162	0.0198	0.0233	0.0266	0.0300	0.0332	0.0364	0.0466	0.0563	0.0658	0.0750	0.0840	0.0928	0.1015	0.1101	0.1186
7.50	0.0080	0.0116	0.0150	0.0183	0.0216	0.0247	0.0278	0.0308	0.0338	0.0433	0.0524	0.0612	0.0697	0.0781	0.0864	0.0945	0.1025	0.1104
8.00	0.0074	0.0108	0.0140	0.0171	0.0201	0.0230	0.0259	0.0287	0.0315	0.0404	0.0489	0.0572	0.0652	0.0730	0.0808	0.0884	0.0959	0.1034
8.50	0.0069	0.0101	0.0131	0.0160	0.0188	0.0216	0.0243	0.0269	0.0295	0.0379	0.0459	0.0536	0.0612	0.0686	0.0759	0.0830	0.0901	0.0971
9.00	0.0065	0.0094	0.0123	0.0150	0.0177	0.0203	0.0228	0.0253	0.0278	0.0356	0.0432	0.0505	0.0577	0.0646	0.0715	0.0783	0.0850	0.0916
9.50	0.0061	0.0089	0.0116	0.0142	0.0167	0.0191	0.0215	0.0239	0.0262	0.0337	0.0408	0.0477	0.0545	0.0611	0.0676	0.0740	0.0804	0.0867
10.00	0.0058	0.0084	0.0110	0.0134	0.0158	0.0181	0.0204	0.0226	0.0248	0.0319	0.0387	0.0453	0.0517	0.0580	0.0641	0.0702	0.0763	0.0822

In the tables:

e_0—eccentricity of axial force with respect to the centroid of the cross-section;

r—radius of the circular section;

η— amplification factor for eccentricity of axial force in eccentrically loaded compression members;

ρ—reinforcement ratio for longitudinal reinforcement which is evenly distributed along the perimeter;

f_{sd}—design tensile strength of longitudinal reinforcement;

f_{cd}—design compressive strength of concrete.

Appendix G
Simplified Calculation for Loss of Prestress in Curved Tendons due to Anchorage Set, Reinforcement Retraction and Joint Compression after Considering Reverse Friction

G.0.1 For post-tensioned concrete flexural members, the loss of prestress due to anchorage set, reinforcement retraction and others after considering reverse friction shall be calculated. The friction coefficient of the duct for reverse friction may be assumed to be the same as that for the forward friction.

G.0.2 The influence length of the reverse friction, l_f, may be calculated by the following equations:

$$l_f = \sqrt{\frac{\sum \Delta l \cdot E_p}{\Delta\sigma_d}} \tag{G.0.2-1}$$

where:

$\Delta\sigma_d$—loss of prestress per unit length due to the duct friction, which is calculated by the following equation:

$$\Delta\sigma_d = \frac{\sigma_{con} - \sigma_l}{l} \tag{G.0.2-2}$$

σ_{con}—tensile stress limit ahead of the anchorage at the jacking end, which is taken in accordance with Clause 6.1.4 of the *Specifications* (MPa);

σ_l—stress of prestressing steel at the anchoring end, after the loss due to friction along the duct is deducted (MPa);

l—distance from jacking end to anchoring end (mm).

When $l_f \leqslant l$, prestress loss of prestressing steel $\Delta\sigma_x(\sigma_{l2})$ at the section away from the jacking end x after considering reverse friction may be calculated by the following equations:

$$\Delta\sigma_x(\sigma_{l2}) = \Delta\sigma\frac{l_f - x}{l_f} \tag{G.0.2-3}$$

$$\Delta\sigma = 2\Delta\sigma_{d} l_{f} \qquad (G.0.2\text{-}4)$$

where $\Delta\sigma$ is the prestress loss of prestressing steel ahead of anchorage within the influence range of l_f when $l_f \leqslant l$, after consideration of reverse friction.

It is denoted that the prestressing steel at x is not affected by the reverse friction if $x \geqslant l_f$.

When $l_f > l$, prestress loss of prestressing steel $\Delta\sigma'_{x}(\sigma'_{l2})$ of the section away from the jacking end x' after considering reverse friction may be calculated by the following equation:

$$\Delta\sigma'_{x}(\sigma'_{l2}) = \Delta\sigma' - 2x'\Delta\sigma_{d} \qquad (G.0.2\text{-}5)$$

where $\Delta\sigma'$ is prestress loss of prestressing steel ahead of anchorage within the influence range of l when $l_f > l$, after consideration of the reverse friction. It may be calculated by the following method: let the area of isosceles trapezoid "*cdbd*" in Figure G.0.2 be $A = \sum \Delta l \cdot E_p$, the cd is obtained by trial computation, then $\Delta\sigma' = cd$.

G.0.3 When both ends of one prestressing steel are jacking (separately or simultaneously) and there is overlap in the influence lengths for the prestress loss due to reverse friction, the stress of prestressing steel of one section within the overlap range after deducting the loss due to the positive friction and the retracted reverse friction may be computed by following steps: jacking and anchoring two ends of the prestressing steel, respectively; calculating the loss of the prestress due to the positive friction and the retracted reverse friction, respectively; subtracting the greater one of the above calculated stresses from the tensile stress limit of the anchorage at the jacking end.

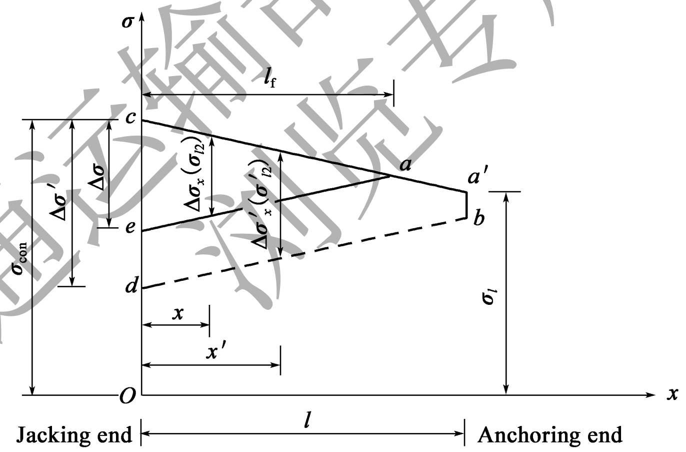


Figure G.0.2 Calculation diagram for loss of prestress after considering reverse friction

In the figure, caa' is the stress distribution line of prestressing steel after the loss due to the positive friction of the duct is deducted.

eaa' is the stress distribution line of prestressing steel after the loss due to the positive friction of the duct and the retraction (after reverse friction is considered) is deducted, when $l_f \leqslant l$.

db is the stress distribution line of prestressing steel after the loss due to the positive friction of the duct and the retraction (after reverse friction is considered) is deducted, when $l_f > l$.

cae is an isosceles triangle; $ca'bd$ is an isosceles trapezoid.

Appendix H
Simplified Calculation for Loss of Prestress Due to Elastic Shortening of Concrete in Post-tensioned Members

H.0.1 When the prestressing steel in the same section are jacked one by one, loss of prestress due to elastic shortening of concrete in a post-tensioned member may be calculated by simplified Eq. (H.0.1):

$$\sigma_{l4}=\frac{m-1}{2}\alpha_{EP}f\Delta\sigma_{pc} \tag{H.0.1}$$

where:

m—number of strands of the prestressing steel;

$\Delta\sigma_{pc}$—normal compressive stress of concrete generated by jacking one prestressing strand, located on the centroid of all prestressing steel in the investigated section (MPa), which is taken as the mean value of all strands.

The above equation in this appendix may also be used to calculate the loss of prestress due to elastic shortening of concrete when prestressing steel is jacking in batches section by section, like a member constructed by segmental cast-in-place cantilever method in the longitudinal direction. In this case, each section of the member is denoted as a batch, and m is the number of prestressing steel through the investigated section in the equation; $\Delta\sigma_{pc}$ is the normal compressive stress of concrete generated by jacking one batch of the prestressing strands located on the centroid of all prestressing steel in the investigated section (MPa), which is taken as the mean value of each batch of strands.

Appendix J
Calculation for Depth of Compression Zone in Type B Prestressed Concrete Flexural Members Permitted to Crack

J.0.1 For T- or I-section prestressed concrete flexural members, the depth of the compression zone, x, may be calculated by the following equations (see Figure 7.1.4):

$$Ax^3 + Bx^2 + Cx + D = 0 \tag{J.0.1-1}$$

$$A = b \tag{J.0.1-2}$$

$$B = 3be_N \tag{J.0.1-3}$$

$$C = 3b_0 h_f'(2e_N + h_f') + 6\alpha_{EP}(A_p g_p + A_p' g_p') + 6\alpha_{ES}(A_s g_s + A_s' g_s') \tag{J.0.1-4}$$

$$D = -b_0 h_f'^2(3e_N + 2h_f') - 6\alpha_{EP}(A_p h_p g_p + A_p' \alpha_p' g_p') - 6\alpha_{ES}(A_s h_s g_s + A_s' \alpha_s' g_s') \tag{J.0.1-5}$$

After A, B, C and D are calculated, x is obtained by substituting them into the Eq. (J.0.1-1).

For prestressed concrete flexural members with rectangular sections, h_f' in Eq. (J.0.1-4) and Eq. (J.0.1-5) equals to zero.

In the above equations:

b—web width of T- or I-section or width of rectangular section;

e_N—distance from the point of application N_{p0} to the edge of the compression zone of the section;

b_0—difference between the width of the compression flange and the web in the T- or I-section, $b_0 = b_f' - b$;

h_f'—depth of the compression flange in T- or I-section;

h_p, h_s—distance from the centroid of prestressing steel and reinforcing steel in the tension zone to the edge of the compression zone of the section, respectively;

g_p, g_s—distance from the centroid of prestressing steel and reinforcing steel in tension zone to the N_{p0}, respectively, $g_p = h_p + e_N$, $g_s = h_s + e_N$;

α_p', α_s'—distance from the centroid of prestressing steel and reinforcing steel in the compression zone to the edge of the compression zone of the section, respectively;

g'_p, g'_s—distance from the centroid of prestressing steel and reinforcing steel in compression zone to the point of application of N_{p0}, respectively, $g'_p = \alpha'_p + e_N$, $g'_s = \alpha'_s + e_N$.

Notes: 1. The stress of reinforcing steel in the compression zone shall conform to the requirement of $\alpha_{ES}\sigma_{cc} \leqslant f'_{sd}$. When $\alpha_{ES}\sigma^{t}_{cc} > f'_{sd}$, A'_s in Eq. (J. 0. 1-4) and Eq. (J. 0. 1-5) shall be substituted by $\frac{f'_{sd}}{\alpha_{ES}\sigma_{cc}}A'_s$, in which f'_{sd} is the design compressive strength of reinforcing steel, σ_{cc} is the compressive stress of concrete at the point of the resultant force of reinforcing steel in the compression zone, which may be calculated by Eq. (7. 1. 4-1). The C in Eq. (7. 1. 4-1) is substituted by the distance from the resultant force in reinforcing steel to the gravity axis of the cracked section.

2. When prestressing steel in the compression zone is subjected to the tensile stress [i. e. ($\alpha_{EP}\sigma_{cc} - \sigma'_{p0}$) is negative], the positive sign in front of the terms with A'_p in Eq. (J. 0. 1-4) and Eq. (J. 0. 1-5) shall be changed to a negative sign. Herein, σ_{cc} is the compressive stress of concrete at the point of the resultant force in prestressing steel in the compression zone.

3. When there is no prestressing steel or reinforcing steel in the compression zone, A'_p or A'_s in Eq. (J. 0. 1-4) and Eq. (J. 0. 1-5) equal to zero.

Wording Explanation for the *Specifications*

1 Words for strictness in implementation of the *Specifications*:

1) "Must" or "must not" is used for a mandatory requirement in any circumstances.

2) "Shall" or "shall not" is used for a mandatory requirement in normal circumstances.

3) "Should" or "should not" is used for an advisory requirement.

4) "May" or "may not" is used for a permissive condition that no requirement is intended.

2 The following expression is used inthe citation of other specifications:

1) To state the relation between the*Specifications* and other specifications in Chapter "General Provisions", it is expressed as "In addition to the *Specifications*, ··· shall also comply with the provisions in the current relevant national and industry standards".

2) When referring to national andindustry standards in clauses of the *Specifications*, it is expressed as "shall comply with the relevant provisions in ×××××××(×××)".

3) When citing other clauses in the *Specifications*, it is expressed as "shall comply with the relevant provisions in Chapter × of the *Specifications*", "shall comply with the relevant provisions in Section ×.× of the *Specifications*", "shall comply with the relevant provisions in Clause ×.×.× of the *Specifications*", or "shall be implemented in accordance with the relevant provisions in Clause ×.×.× of the *Specifications*".

Background to Provisions

1 General Provisions

1.0.1 The *Specifications* is mandatory, which specifies basic requirements for the structural design of highway reinforced concrete and prestressed concrete bridges and culverts.

1.0.2 The *Specifications* is applicable for the design of new reinforced and prestressed concrete structures and members made of normal concrete, which are widely used in practical highway bridges and culverts.

1.0.3 Expressions with partial factors are adopted in the limit state design, in which the design action is expressed by the characteristic action multiplied by a corresponding partial factor. These two values are specified in *General Specifications for Design of Highway Bridges and Culverts* (JTG D60—2015). Design material strengths are directly given in the *Specifications*, and are obtained by dividing the characteristic material strengths by the corresponding partial factors for resistance (materials), which are given in the relevant commentary of the *Specifications*.

2 Terms and Symbols

This chapter only lists the terms that are present and need to be clearly defined in the *Specifications*. General terms related to the bridge profession are familiar to readers and are not included.

Common terms that have been already defined are deleted in this revision, such as terms of reliability, action, combination of actions and others in the former edition of the *Specifications*. Special terms related to concrete structures in the *Specifications* are supplemented, such as disturbed region, strut-and-tie model, development length, thickness of concrete cover, etc.

Explanations for some terms are internationally recognized, for example, limit state. But most of them are general meaning and not internationally or nationally recognized.

Symbols in this chapter are listed for material properties, actions and action effects, geometrical parameters, calculation coefficients, and others. These main symbols are adopted in accordance with the rules in the current standard of China. If no rule is specified in the current standard to follow, they are adopted as customary. Not all the symbols used in the *Specifications* are listed, and only some primary symbols are listed in this chapter.

3 Materials

3.1 Concrete

3.1.1 Characteristic compressive strength refers to the compressive strength (MPa) with 95 percent level of confidence measured by the standard test method, in which the specimens are produced by the standard method and cured 28 d (the test age may be extended appropriately according to the specific situation when a large amount of fly ash and other mineral admixtures are added in cement and concrete). The dimensions of the standard specimen of concrete and the principle for determining the characteristic strength in the *Specifications* are consistent with relevant international standards and the current *Code for Design of Concrete Structures* (GB 50010) (referred to as *Code GB* 50010).

3.1.2 This clause specifies the lower limit of the concrete strength class used for load-bearing members in highway bridges and culverts, the main changes are as follows: all of the lower limits of the concrete strength class of reinforced concrete members are improved by one level: the requirement for concrete strength class of reinforced concrete members is changed from "not lower than C20" to "not lower than C25"; when steel bars of HRB400, HRB500, HRBF400 or RRB400 are used, it is changed from "not lower than C25" to "not lower than C30".

The prestressing steel used in prestressed concrete members is mainly composed of steel strands and steel wires, so that the lowest concrete strength class for the members is taken as C40, which is the same as the former edition of the *Specifications*.

3.1.4 The design axial compressive strength of concrete, f_{cd}, is obtained by dividing the characteristic axial compressive strength of concrete by the partial factor of concrete material, γ_{fc} =

1.45. The partial factor of concrete material is taken to achieve the target reliability index in analyzing a brittle failure member in a structure of safety level II.

The design axial tensile strength of concrete, f_{td}, is equal to the characteristic axial tensile strength of concrete divided by the partial factor of material, which is the same as that for the axial compressive strength of concrete.

3.1.5, 3.1.6 The elastic modulus, shear modulus, and Poisson's ratio of concrete are the same as those in the former edition of the *Specifications*. The elastic modulus of concrete is calculated in accordance with the following formula:

$$E_c = \frac{10^5}{2.2 + \frac{34.74}{f_{cu,k}}} \tag{3-11}$$

3.2 Reinforcement

3.2.1 The types of reinforcement specified in this clause are mainly based on the providsions of the national standards newly issued. Explaination for this clause is as follows:

(1) Reinforcing steel: Hot-rolled deformed bars of 500 MPa is added. High-strength hot-rolled deformed bars of 400 MPa and 500 MPa as the dominant longitudinal steel bars are promoted to be used. Hot-rolled deformed bars of 335 MPa are eliminated in applications. Plain bars of 235 MPa grade are replaced by those of 300 MPa grade. HRBF series of fine grain deformed bars produced by temperature control-rolled process is introduced. In the interim, the design values for plain bars of 235 MPa grade and deformed bars of 335 MPa grade may still be taken in accordance with the former edition of the *Specifications*.

HPB300 is the strength grade of the plain bar, taken from the current *Steel for the Reinforcement of Concrete-Part 1: Hot Rolled Plain Bars* (GB 1499.1), in which the nominal diameter $d = 6 \sim 22$ mm, increasing by an even number of 2 mm. HRB400 and HRB500 are the strength grades of hot-rolled deformed bars, and HRBF400 is the strength grade of fine grain deformed bars, all taken from the current *Steel for the Reinforcement of Concrete-Part 2: Hot Rolled Ribbed Bars* (GB 1499.2), in which the nominal diameter $d = 6 \sim 50$mm, decreasing by an even number of 2 mm if d is smaller than 22 mm, and taking as 25, 28, 32, 36, 40, 50 (mm) if d is larger than 22 mm. RRB400 is the strength grade of quenching and self-tempering ribbed bar, taken from the current *Quenching and Self-tempering Ribbed Bars for the Reinforcement of Concrete* (GB 13014), in which the nominal diameter $d = 6 \sim 50$ mm, and its size grading is the same as HRB. For the convenience in the design and application, the nominal sectional area and weight of the above steel bars are listed in Table 3-2.

Table 3-2 Nominal cross-sectional area and weight of steel bar

Nominal diameter(mm)	Nominal cross-sectional area(mm^2)	Nominal weight(kg/m)
6	28.27	0.222
8	50.27	0.395
10	78.54	0.617
12	113.10	0.888
14	153.90	1.210
16	201.10	1.580
18	254.50	2.000
20	314.20	2.470
22	380.10	2.980
25	490.90	3.850
28	615.80	4.830
32	804.20	6.310
36	1018.00	7.990
40	1257.00	9.870
50	1964.00	15.420

(2) Prestressing steel: steel strands with high strength and large diameters are added, prestressing threaded bars with large diameters are included, and notched steel wires with poor anchoring performance are eliminated.

Steel strands and steel wire are mainly used as prestressing steel in the *Specifications*, prestressing threaded bars are only used for medium and small members or for vertical and horizontal reinforcement. The technique requirements for steel strands are taken from the current specifications *Steel Strand for Prestressed Concrete* (GB/T 5224); prestressing steel wires are the stress-relieving plain and screw-thread ribbed steel wires, and their technique requirements are taken from the current specifications *Steel Wire for Prestressing of Concrete* (GB/T 5223); the technique requirements for prestressing threaded bars are taken from the current specifications *Screw-thread Steel Bars for the Prestressing of Concrete* (GB/T 20065). The nominal cross-section area and weight of steel strands, steel wires, and prestressing threaded bars are listed in Table 3-3.

Table 3-3 Nominal cross-sectional area and weight of prestressing steel

Types of prestressing steel and nominal diameter(mm)			Nominal cross-sectional area(mm^2)	Nominal weight(kg/m)
Steel strand	1×7	9.5	54.8	0.432
		12.7	98.7	0.774
		15.2	139.0	1.101
		17.8	191.0	1.500
		21.6	285.0	2.237

continued

Types of prestressing steel and nominal diameter (mm)		Nominal cross-sectional area (mm^2)	Nominal weight (kg/m)
Steel wire	5	19.63	0.154
	7	38.48	0.302
	9	63.62	0.499
Prestressing threaded bar	18	254.5	2.11
	25	490.9	4.10
	32	804.2	6.65
	40	1256.6	10.34
	50	1963.5	16.28

3.2.2, 3.2.3 The principles for determining the strength index of reinforcement in the *Specifications* are explained as follows:

(1) The characteristic tensile strength of the steel bar is taken as the yield strength of reinforcement specified in the current national standard, with a level of confidence not less than 95 percent. The yield strength of quenched and self-tempered steel bars is stipulated as 440 MPa by the national standard, which is the yield strength on delivery. The strength at the joint of the steel bar after flash butt welding will decrease, so its strength grade is taken as RRB400 and the characteristic tensile strength is taken as 400 MPa in practical engineering applications. The design tensile strength of the steel bar is obtained by dividing the characteristic tensile strength of the steel bar by its material partial factor, $\gamma_{fs} = 1.2$. Applying the design strength, the reliability index obtained from the analysis of the axial tension member exceeds the target reliability index for the ductile failure member specified for the structure with safety level II.

(2) The characteristic tensile strength of steel strand and steel wire is taken from the ultimate tensile strength specified in the current national standard. According to the provisions of the current national standard, the conditional yield strength of steel strand and steel wire is 0.85 times of its tensile strength. In reference to *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTJ 023-85) (hereinafter referred to as *Specifications JTJ 023-85*), the safety factor of the design strength for steel strand and steel wire is 1.25. Therefore, the design tensile strength of steel strand and steel wire in *Specifications* is taken as $f_{fd} = f_{fk} \times 0.85/1.25 = f_{fk}/1.47$, so it is obtained from the characteristic tensile strength divided by the material partial factor, $\gamma_{fp} = 1.47$.

The characteristic tensile strength of prestressing threaded bar is taken from the yield strength of the reinforcing steel in the current national standard, and its partial factor of material is the same as that of reinforcing steel, $\gamma_{fp} = 1.2$.

(3) The design compressive strength of the steel bar, f'_{sd} or f'_{pd}, is determined according to the following two conditions:

1) Compressive strain of steel bar, ε'_s (or ε'_p) $= 0.002$.

2) The design compressive strength of steel bar, f'_{sd} (or f'_{pd}) $= E_s\varepsilon'_s$ (or $E_p\varepsilon'_p$), must not be higher than the design tensile strength of steel bar, f_{sd} (or f_{pd}).

For example, $f'_{sd} = 0.002 \times 2.0 \times 10^5 = 400$ MPa for HRB400-class steel bar is larger than the design tensile strength of steel bar, $f_{sd} = 330$ MPa, then $f'_{sd} = 330$ MPa is taken as the design compressive strength; $f'_{sd} = 0.002 \times 2.0 \times 10^5 = 400$ MPa for HRB500 class steel bar is smaller than the design tensile strength of steel bar $f_{sd} = 415$MPa, then $f'_{sd} = 400$ MPa is taken as the design compressive strength; For steel strand with the characteristic tensile strength, its design value $f'_{pd} = 0.002 \times 1.95 \times 10^5 = 390$ MPa is smaller than the design tensile strength $f_{pd} = 1260$ MPa, then $f'_{pd} = 390$ MPa is taken as the design compressive strength.

The steel strands with a strength of 1960 MPa and a diameter of 21.6 mm are supplemented in this revision. When they are used as prestressed reinforcement, attention shall be paid to match their anchor fixture. Only after the reliability of the anchor fixture and process are checked and confirmed, it may be used in the project.

According to relevant experimental research results, the design tensile strength of stirrups to resist shear, torsional moment and punching is limited, but it is not limited when the stirrup is confinement reinforcement (such as continuous spiral stirrups or closed welded stirrups) used as a hoop to confined the concrete, thus the advantage of high-strength reinforcement may be fully exerted.

4 Basis of Structural Design

4.1 General

4.1.1 The ultimate limit state includes strength failure of members and connectors, instability of structures or members, and overturning of structures. The serviceability limit state includes cracking and deformation affecting the normal use of the structures and members.

4.1.2 In order to effectively ensure the safety and durability of the bridge and culvert structures, the basic requirements for structural design has been supplemented in this revision. The content of structural design, including structural scheme, mechanical analysis, section design, joint construction, durability, as well as special performance design of the project, shall consider two levels: the whole structural system and a single member.

4.1.3 In accordance with provisions in *General Specifications for Design of Highway Bridges and Culverts* (JTG D60—2015), the standardized spans are taken as 0.75 m, 1.0 m, 1.25 m, 1.5 m, 2.0 m, 2.5 m, 3.0 m, 4.0 m, 5.0 m, 6.0 m, 8.0 m, 10 m, 13 m, 16 m, 20 m, 25 m, 30 m, 35 m, 40 m, 45 m and 50 m.

4.1.4, 4.1.5 Concrete beam bridges are divided into two categories: prefabricated structure and cast-in-place structure. The span limits for various structures are refined appropriately according to the standardized spans based on the former edition of the *Specifications*.

4.1.7 For concrete bridges that have system conversion in the construction process, the incremental superposition method is generally adopted to calculate the action effects, *i. e.*, to calculatethe increment of deformation, internal force, and stress of the structure in each stage, and then to accumulate them stage by stage). In analyzing the effects of shrinkage and creep of concrete, the internal force redistribution in the statically indeterminate structures and the stress redistribution due to the concrete constrained by steel bars need to be taken into account. The effects can be calculated according to the functions of concrete shrinkage and creep in Appendix C, considering the corresponding internal forces and concrete age of the concrete structures at the construction stages.

The main spatial effects of concrete bridges are the shear lag effect, transverse distribution effect of vehicular load, and thin-wall effect of box girder. The analysis methods of spatial effects mainly include the analysis method with framed model + simplified parameter, practical refined analysis model in Appendix A, and solid element model.

Among them, the analysis method with framed model + simplified parameter is the most widely used. The shear lag effect is considered by the effective width of the flange (see Section 4.3 for details), and the transverse distribution of vehicular load and the thin-wall effect of box girder are considered by multiplying a coefficient due to transverse positioning of vehicular loads. Generally, this coefficient for a box girder is taken as 1.15, while for prefabricated girders it is approximately obtained from the analysis for the transverse distribution coefficient of vehicular load. The boundary conditions of concrete bridges with complex shapes, such as curved, wide, skewed, with various width or divergent, are inconsistent with the algorithm assumptions for the transverse distribution coefficient of load, and thus, the simplified parameters shall not be applied blindly in the structural analysis, and the refined finite element model shall be adopted.

Compared with the solid element model, the application of Appendix A *Practical Refined Analysis Models for Bridge Structures* eliminates the process of obtaining the internal force from the stress according to the integral rule, and the internal forces of the structure can be obtained directly, and then the strength can be checked.

4.1.8 Since 2007, accidents of transverse overturning instability and collapse in ramp bridges with box sections have happened successively in Baotou of Inner Mongolia Province, Tianjin, Shangyu of Zhejiang Province, Harbin of Heilongjiang Province, and Heyuan of Guangdong Province. The basic characteristics of the bridge in an accident (as shown in Figure 4-1) are as follows: the superstructure adopts a monolithic box girder; the structural system is continuous beam and the superstructure is supported on compression-only bearings. There are double or three bearings on the abutment or transition piers, while only one bearing is on all or part of the interior piers.

The sequence of failure of the bridge in an accident is as follows: the compression-only bearings deviate from the normal compression state first; the supporting system of the superstructure no longer provides effective constraints; the torsional deformation of the superstructure tends to diverge until its collapse due to transverse instability; the bearing and the substructure are damaged finally, as shown in Figure 4-2. According to the current standard of *Unified Standard for Reliability Design of Engineering Structures* (GB 50153), this kind failure belongs to the category of the ultimate limit state of load-carrying capacity.

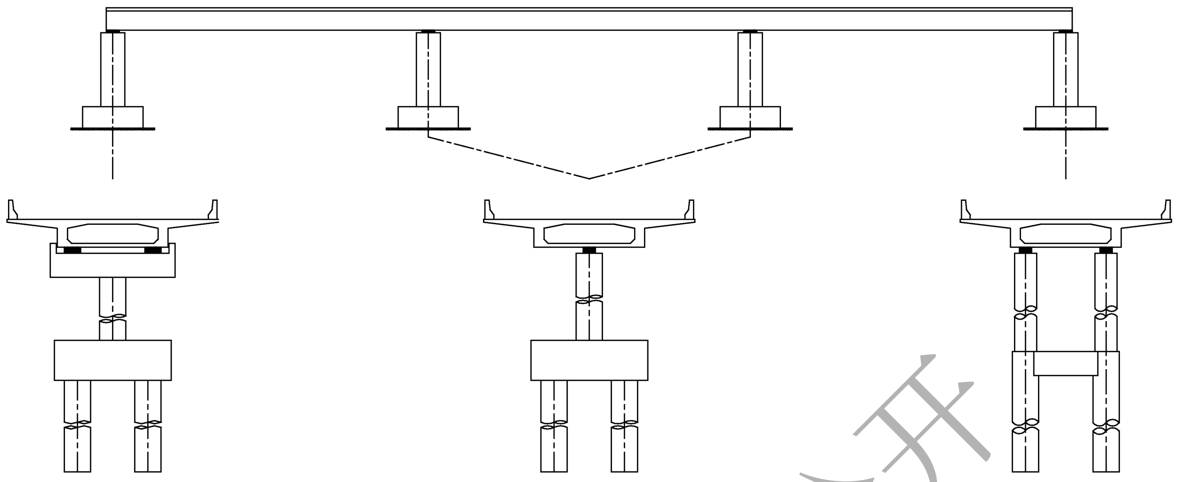


Figure 4-1　Typical types of bridges failed due to overturning

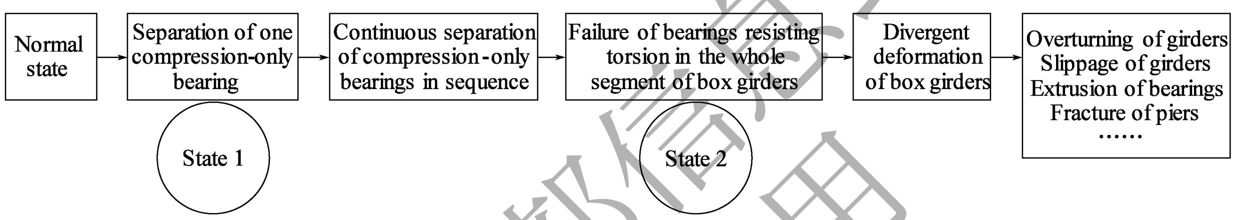


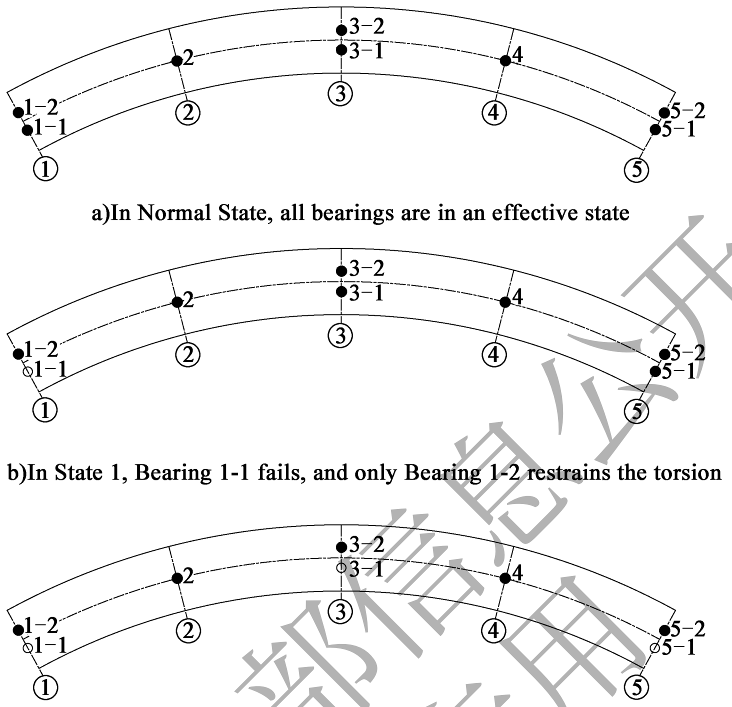
Figure 4-2　Typicalsequence of failure

The overturning failure has two distinct states. In State 1, the compression-only bearing of the box girder loses the contact with the support due to uplift; in State 2, bearings of the box girder fail to resist torsion. Anti-overturning is checked in these two states referring to relevant specifications in the world:

(1) For State 1, the compression-only bearings of the box girder bridge are in the compression state under the fundamental combination of actions.

(2) A pair of double bearings on the same pier of the box girder bridge forms a bearing support against torsion, which can restrain the torsion and torsional deformations. When the vertical force of one bearing becomes zero and fails to support the girder, the other effective bearing only restricts the torsion moment, while can not provide constraint to the torsional deformation. When all torsional bearings of the box girder fail, the box girder is in the ultimate limit state of force balance or failure state induced by torsional deformation, which is named as State 2, as shown in Figure 4-3c. For State 2, the expression of "stability due to action effect ≥ stability coefficient × instability due to action effect" is adopted referring to the checking against transverse overturning in the retaining wall and rigid foundation.

When the box girder bridge is in the State 2, there is an effective bearing on each pier, as shown in Figure 4-4. The stability effect and instability effect shall be calculated according to the moment of the failed bearings to the effective bearings:



a)In Normal State, all bearings are in an effective state

b)In State 1, Bearing 1-1 fails, and only Bearing 1-2 restrains the torsion

c)In State 2, Bearings 1-1, 3-1 and 5-1 all fail, and torsional deformation of the box girder cannot be restrained by the effective bearings

Figure 4-3　Sequences of box girder bridge reaches to the State 2

Action effect for stability　　$\sum S_{bk,\ i} = \sum R_{Gki} l_i$　　(4-1)

Action effect for instability　　$\sum S_{sk,\ i} = \sum R_{Qki} l_i$　　(4-2)

where:

l_i——distance between the centers of the failed bearing and effective bearing on the ith pier;

R_{Gki}——reaction force of the failed bearing at the ith pier under the permanent action, which is determined according to the supporting system with effective bearings, and is taken by the combination of their characteristic values;

R_{Qki}——reaction force of the failed bearing at the ith pier under the variable action, which is determined according to the supporting system with effective bearings, and is taken by the combination of their characteristic values. The effect of vehicular load (including impact force) is determined according to the most unfavorable layout corresponding to each failed bearing.

Taking into account the deviation coefficient of the simplified analysis method and the amplification factor of vehicular load effect under the actual dense arrangement of vehicles on the deck, the factor of safety against overturning in the transverse direction is determined to be 2.50.

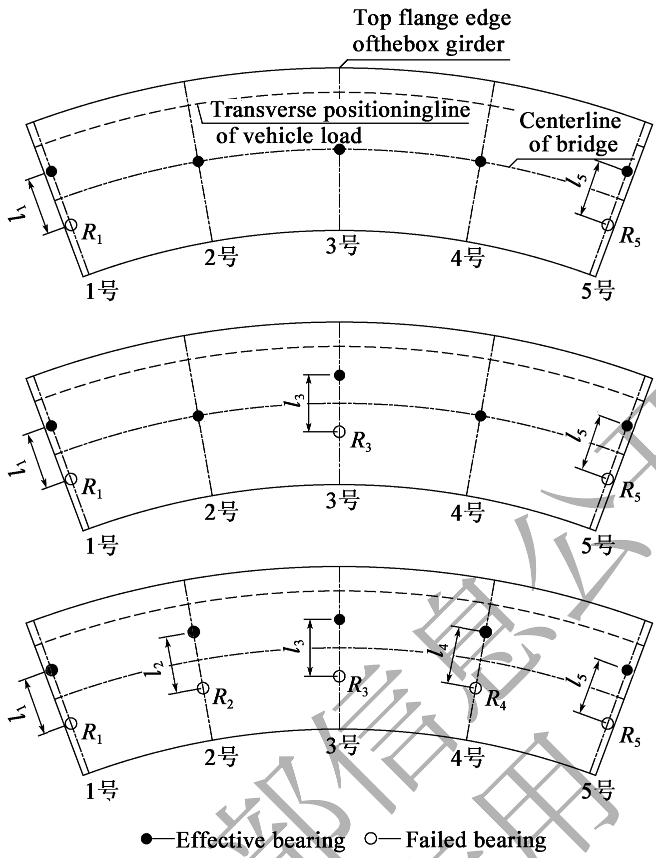


Figure 4-4　Diagram of effective bearings in State 2

4.1.9　Since 1980s, the international engineering community has advocated dividing the concrete structure into B region and D region: B region refers to the area where the sectional strain distribution matches the assumption that plane sections remain plane and is calculated according to the "beam system"; D region, namely the stress disturbance zone, refers to the area where the sectional strain distribution does not match to the assumption that plane sections remain plane, generally located near the point of application of concentrated force or where the geometric size changes. The typical stress disturbance zone of a concrete beam bridge is shown in Figure 4-5.

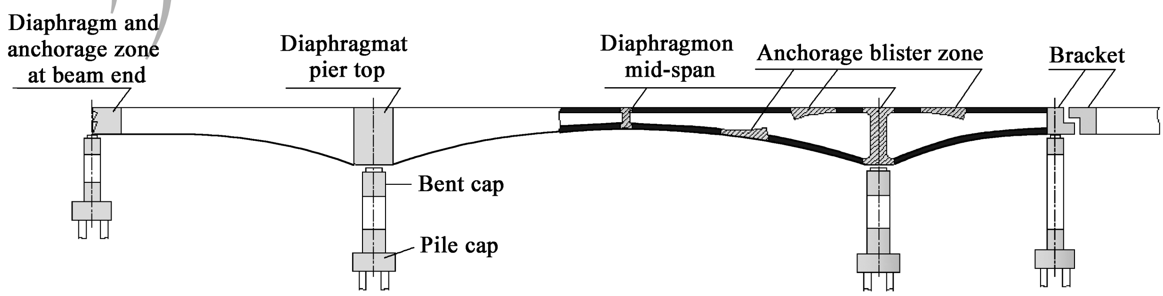


Figure 4-5　Typical stress disturbance zone of a concrete beam bridge

Common design methods for stress disturbance zones include the strut-and-tie model method, solid finite element model method, or simplified formula method for special stress situations:

(1) The strut-and-tie model is a simplified mechanical analysis method based on the internal force transmission path of the continuum. In the former edition of the *Specifications*, the method of strut-and-tie model was introduced in the calculation of pile caps. In this revision, the method is further extended to the anchorage zone of post-tensioned prestressed members, the diaphragm at bearing, as well as the bent cap. The strut-and-tie model is based on the theorem of the lower-bound plasticity. It can be found from the test and theoretical studies of the stress disturbance zone in China and other countries that the mechanical mechanism of the stress disturbance zone can be well revealed by the strut-and-tie model, and the calculation of the bearing capacity in the stress disturbance zone by such method is relatively safe.

(2) In the solid finite element model method, the stress distribution in the stress disturbance zone is analyzed by using elastic and elastic-plastic solid finite element model, and the reinforcement is designed according to the analysis results.

(3) For special stress situations analyzed by the simplified formula method, the reinforcement is designed by using the analytical formula of internal force derived by means of elastic mechanics or force streamline model. In Chapter 8 of the *Specifications*, the simplified formula of tension in the typical stress disturbance zone is proposed. The calculated tension by this simplified method equals to the internal force of the tie calculated by the strut-and-tie model.

4.1.10 The durability of the concrete structure largely depends on the construction quality and the proper maintenance and routine inspection during the service life, therefore, the relative requirements are specified specially in this clause. Detail requirements on quality control and quality assurance in the construction stage have been specified in *Technical Specification for Construction of Highway Bridge and Culvert* (JTG/T F50—2011), thus it is not repeated hereby.

4.2 Analysis of Slabs

4.2.1 For slabs supported on four sides under uniformly distributed load q, the deformation of the mid-span at the long side is $\Delta_1 = k\frac{q_1 l_1^4}{EI}$, and the deformation of the mid-span at the short side is $\Delta_2 = k\frac{q_2 l_2^4}{EI}$, in which the coefficient k is determined by the support conditions of the slab, q_1 and q_2 are the uniform load distributed on the long side and the short side of the slab, respectively, l_1 and l_2 are the calculating spans of the long and short sides, respectively, and EI is the flexural stiffness of the slab. According to the conditions of $\Delta_1 = \Delta_2$ and $q = q_1 + q_2$, we can obtain $q_1 =$

$\frac{l_2^4}{l_1^4 + l_2^4}q$ and $q_2 = \frac{l_1^4}{l_1^4 + l_2^4}q$. When $\frac{l_1}{l_2} \geqslant 2$, $q_1 \leqslant \frac{1}{16}q_2$, it can be found that the load distributed by the long end span is much less than that of the short end span. If the bending moment is compared, the ratio of the bending moment of the long span to the short span is:

$$(M_1/M_2) = \frac{k'q_1 l_1^2}{k'q_2 l_2^2} = \frac{[l_2^4/(l_1^4 + l_2^4)]q}{[l_1^4/(l_1^4 + l_2^4)]q} \times \frac{l_1^2}{l_2^2} = \left(\frac{l_2}{l_1}\right)^2$$

When $\frac{l_1}{l_2} \geqslant 2$, $M_1 \leqslant \frac{1}{4}M_2$, in which the coefficient k' is determined by the support conditions of the slab. Therefore, it can be calculated as a one-way slab, taking the long side as the support and the short side as the span. If $\frac{l_1}{l_2} < 2$, the load shall be distributed according to the elastic mechanics method. In Clause 3.24.6 of *AASHTO Standard Specifications for Highway Bridges* (14th Edition, 1989) (hereinafter referred to as *U.S. AASHTO Specifications 14 th Edition*), the simplified method for load distribution is: uniform load is $q_2 = \frac{l_1^4}{l_1^4 + l_2^4}q$, and concentrated load is $P_2 = \frac{l_1^3}{l_1^3 + l_2^3}P$ (q_2 and P_2 are the uniform load and concentrated load distributed on the short span, respectively; q is the uniform load on the slab, and P is the concentrated load on the slab). In addition, when $\frac{l_1}{l_2} \geqslant 1.5$, it is also specified that the short span is calculated as a one-way slab, in which the threshold of $\frac{l_1}{l_2}$ is smaller than that of the *Specifications*.

4.2.2 The effective span for bending moment is taken as the sum of clear span and slab thickness, but not larger than the distance of the supporting points because the slab is not completely fixed without rotation at the supports.

For deck slabs integrated with the beam ribs, the support section is considered conservatively as a fixed support, and its bending moment is:

$$M = -\frac{1}{12}ql^2 = -\frac{2}{3} \times \frac{1}{8}ql^2 = -0.67M_0 \approx -0.7M_0 \tag{4-3}$$

where:

q——uniformly distributed load per unit length of the slab;

l——effective span of the slab;

M_0——bending moment at the midspan of the simply supported slab.

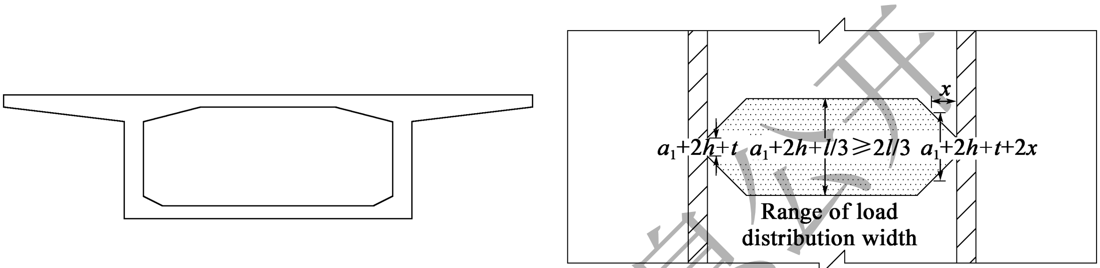
The bending moment of the section at the mid-span is considered safe by taking the support of the slab as elastic semi-fixed, which is calculated by:

$$M = +\frac{1}{16}ql^2 = +\frac{1}{2} \times \frac{1}{8}ql^2 = +0.5M_0 \tag{4-4}$$

When the ratio of the slab thickness to the beam depth is not smaller than 1/4, the constraint from the support member to the slab is decreased, and the moment at the mid-span is taken as $+0.7M_0$.

4.2.3 When the wheel load is located on the mid-span of the slab, its distribution width perpendicular to the span direction of the slab is generally expressed in the form of the spreading width of the wheel patch through the deck overlay, adding a fractional value of the effective span. The distribution width is stipulated in this clause as the spreading width plus the 1/3 effective span. According to the elastic theory, the distribution width of wheel load on the mid-span is generally 0.6 ~ 0.7 times the effective span, so it is stipulated no less than 2/3 of the effective span.

Generally, the wheel load distribution width on the top slab of a box girder is calculated according to the diagram shown in Figure 4-6.



a) b)

Figure 4-6 Calculation of wheel load distribution width

4.2.4 According to the test data presented by Olsen, the conditions for skewed slab bridges calculated as right slab bridges are listed in Table 4-2. A Type 2 or Type 3 monolithic skewed slab bridge with a skew angle $\phi < 15°$ can be calculated as a right bridge with an effective span specified in this provision. The slabs in prefabricated skewed slab bridges are jointed by shear keys in the transverse direction, where only shear force transferred between the adjacent slabs is considered, each slab is narrow and has a large ratio of span-to-width, therefore these bridges are categorized as Type 1. Therefore, the slabs in prefabricated skewed slab bridge with skew angle $\phi \leqslant 40°$ may be calculated as right slabs with effective skew span. In addition, according to the description in *Design Manual for Highway Bridge* published by China Communication Press in 1978 (in Chinese), skewed slab bridges with $l > 1.25b'$ (b' is the supporting width of the skewed slab) and $\phi < 40°$ may be calculated as right bridges with spans equal to their skew spans.

Table 4-2 Conditions for skewed slab bridges calculated as right slab bridges

Type	Skew span / slab width l/b	Skew angle ϕ	Effective span	Primary reinforcement arrangement
1	$\geqslant 1.3$	$\leqslant 40°$	Skew span	Parallel to the skew span
2	1.3 ~ 0.7	$< 15°$	Right span	The central part is perpendicular to the length direction of the pier/abutment, and the edge part parallel to the skew span
		$15° < \phi < 40°$	1/2(skew span + right span)	
3	< 0.7	$< 40°$	Right span	Perpendicular to the length direction of the pier/abutment

Notes: The skew span refers to the span along the bridge axis, the right span refers to the distance perpendicular to the distance between two piers (abutments), and the slab width is the width b perpendicular to the bridge longitudinal axis.

4.2.5 The distribution of the wheel load in this clause applies in the situation when l_c is less than 2.5 m. When l_c is larger than 2.5 m in the cantilever slab, the negative bending moment per unit width at the fixed end, m_x, can be calculated by Sanko-Bakht formula:

$$m_x = f(o, y) = -\frac{P}{\pi}A' \frac{1}{\mathrm{ch}\left(\frac{A'y}{a_0} / \frac{l_c}{a_0}\right)} \tag{4-5}$$

where:

P——concentrated load;

l_c——length along the x-axis from the load position to the support;

a_0——span of the cantilever slab;

x, y——plane coordinate system (Figure 4-7);

A'——correlation parameter, which is taken from Table 4-3.

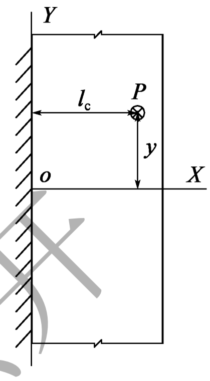


Figure 4-7 Calculation diagram of m_x

Table 4-3 Value of parameter A'

Position of load (l_c/a_0)	0.25	0.50	0.75	1.00
A'	1.07	1.17	1.30	1.53

Notes: If the position of the load is different from the values in the Table, A' is calculated by linear interpolation.

The calculated result of Eq. (4-5) is generally 1.15 ~ 1.30 times the calculated result of Eq. (4.2.5). In addition, there will be a positive bending moment under the point of application of the wheel load, and thus the reinforcement for positive bending moment should be considered.

4.2.6 Experiments indicated that the stress flow of a slab strengthened by haunch with $\tan\alpha > 1/3$ was concentrated within the range of $\tan\alpha = 1/3$, and part of the haunch did not carry the load, Therefore, in the *Specifications*, the effective depth of the slab is considered by $\tan\alpha = 1/3$ when $\tan\alpha > 1/3$.

4.3 Analysis of Girders

4.3.1 The action effect of statically indeterminate structure is related to the flexural stiffness of members, E_cI, in which E_c will decrease after the structure is subjected to repeated loading, and I will also reduce due to crack for members permitted to crack. In the previous analysis of the action effects on the statically indeterminate structures, according to Clause 4.4 in *Code for Design of Highway Bridge* issued in 1975, the value of E_cI for the reinforced concrete statically indeterminate structures is multiplied by 1/1.5 because no reinforcement has been designed in the preliminarily determined sections when the action effect is calculated. In the *Specifications*, it is multiplied by 0.8, referring to the provisions on calculation of structural deformation of railway reinforced concrete statically indeterminate structures. As for the prestressed concrete members expected not to crack, $E_c I$ was not reduced in the former design, and this method is still adopted in the *Specifications*, herein, I is the inertia moment of the concrete gross section because there is no prestressing steel in the preliminarily determined sections when the action effect is calculated. This clause is applicable only for the analysis of action effects, rather than the deflection calculation in the serviceability limit state.

4.3.2 The effective flange width is used for the two limit states in the *Specifications*, the reason is as follows: when the reinforcement in the beam reaches the yield strength, the concrete within the effective width of the beam will first be damaged due to the shear lag effect, at this moment the beam can be safety considered to be in the ultimate limit state. For the stress calculation of the prestressed concrete members during jacking operation, the stress generated by the prestressed force as axial force can be calculated by the physical full width of the flange referring to Clause 5.1.3.2 of the *Design Specifications for German Concrete Bridges* DIN1075 (hereinafter referred to as *German Standard DIN 1075*), because the stress induced by the axial force shall be calculated by the full width, while the effective width is only applicable to the calculation for flexural members.

4.3.3 For T- and I-beams subjected to bending moments, the normal stress of the flange close to the web is the same as that of the web, but decreases with the increases of the distance from the web (Figure 4.8) due to the shear lag effect in the transverse direction. This normal stress varied along the flange width in the same fiber layer needs to be analyzed by advanced material mechanics, which can be referred to Chapter 6 of "*Theory and Calculation of Beam Bridges*" edited by Xiangyun Cheng published by the China Communication Press in Chinese (hereinafter referred to as *Beam Bridges*).

In order to apply the elementary material mechanics in the calculation, the effective flange width is used, which is determined by assuming that the resultant of normal stress in the range of effective flange width is equal to that in the range of the original full width of the flange, and the normal stress of any fiber layer within the effective width is same as the stress in the web of the same fiber layer. As shown in Figure 4-8, the effective flange width is set as b'_f, on any fiber layer, the area of the equivalent normal stress shown in the dashes area including the effective width on both sides of the web shall be equal to the area of the actual normal stress shown in the solid line area. The calculation formula of λ_x (refer to the *Advanced Mechanics of Materials* edited by Timoshenko) (Figure 4-8) is:

$$2\lambda_x = \frac{4l}{\pi(1+\upsilon)(3-\upsilon)} \tag{4-6}$$

where:

υ——Poisson's ratio of material of the beam;

l——design span of the beam.

According to the above equation, $2\lambda_x = 0.379l$. For safety, the effective width b'_f of a simply supported beam is taken as $l/3$, which includes the web. For a continuous beam, it is taken as 1/3 times of the spacing of contraflexure points. The spacing of contraflexure points in the positive bending moment region of interior span for a continuous beam is taken as $0.6l$, the effective width is $0.6 \times l/3 = 0.2l$; the spacing of contraflexure points in the positive bending moment region of end span of continuous beam is taken as $0.8l$, the effective width is $0.8 \times l/3 = 0.27l$; the spacing of contraflexure points in the negative bending moment region of interior supports of continuous beam is taken as 0.2 times of the sum of the span lengths of the two adjacent span at the supports $(l_i + l_{i+1})$, the effective width is $0.2 \times 1/3 \times (l_i + l_{i+1}) = 0.07(l_i + l_{i+1})$.

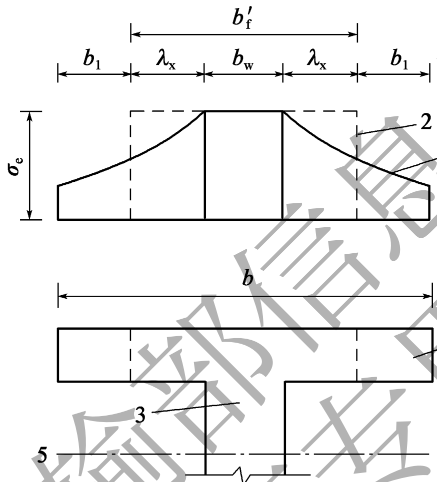


Figure 4-8 Stress diagram of T-beam

1-actual normal stress; 2-equivalent normal stress; 3-web; 4-flange; 5-neutral axis; σ_e-equivalent normal stress

Another control condition of effective flange width is that both sides of the web should be counted into 5 ~ 8 times of the cantilever plate depth as the effective flange depth, which is related to the shear strength of the flange.

As shown in Figure 4-8, the volume of normal stress in two flanges and the web is the same within the effective width b'_f (dash line) and the original full-width b (solid line), and thus whether the effective width and equivalent normal stress, or full width and actual normal stress are used, both shall have the same neutral axis in the serviceability limit state (elastic state). Therefore, when the equivalent normal stress is calculated by the section with effective width, the neutral axis shall take the neutral axis of the original full-width section, which is detailed in Chapter 6 in "*Beam Bridges*". In the design calculation of highway reinforced concrete T-beam, the full width is mostly adopted. For box girders (see Clause 4.3.4 of the *Specification*), the difference between the positions of the two neutral axes of the full-width section and the effective section is small. Accordingly, there is no hard-and-fast provision for whether the neutral axis of the full-width section or effective section should be used in the *Specifications*.

4.3.5 The distribution of negative bending moment at the interior support of the continuous beam is tapered form in theory, however, in fact, the bearing at support has a certain width, and there is a diaphragm at the support in addition, the reaction force of support will be spread and distributed in the girder, thus the actual bending moment diagram shows a smooth curve. Assume that the reaction force of support is spread to the gravity axis of the beam at a rigidity angle of 45°, the distribution length at the gravity axis is a, and the unit load intensity is $q(=R/a)$, thus a bending moment is $M'=qa^2/8$. The bending moment M_e after reduction is obtained by subtracting M' from the theoretical bending moment M. It is specified that M' shall not be larger than 10% of M to avoid too much reduction for a beam with a large depth.

4.3.6 The action effect of a continuous beam with variable sections is related to the change of the inertia moment, however, if the change is small, including the haunch depth and/or the bottom slope is not large, it can also be calculated as a girder with constant cross-section. Such a calculation in which the action effect induced by the change of inertia moment is not considered is better to be limited in the case the ratio of inertia moment at support to that of mid-span is not larger than 2.

4.3.7 The cross-section of the continuous beam has a sharp change in the support due to diaphragms, which complicates the calculation of the action effect. For practical convenience, the influence of diaphragms can be neglected in the calculation.

4.3.8 The effects induced by actions of temperature gradient and nonuniform settlement of foundation shall be considered for continuous beams, while the effects induced by uniform temperature difference and concrete shrinkage action shall also be considered for other statically indeterminate structures.

For the prestressed concrete continuous beams or other statically indeterminate structures, the secondary effect due to the constraint of elastic deformation induced by prestressing force shall also be considered: the eccentric bending moment on the beam induced by prestressing force, which will act on the beam as an external force and will produce a secondary reaction force at its support, this reaction force further generates secondary shear force and secondary bending moment on the beam. All these effects are collectively known as the secondary effect. In Clause 3.4.7 of *Specifications JTJ 023-85*, it was specified that the secondary effect caused by prestressing force might be excluded in the plastic stage (equivalent to the ultimate limit state). In the *Specifications*, considering that the plastic hinge has not been completely formed in the ultimate limit state, it is specified that the secondary effect caused by prestressing force in this state shall be considered. As for the influence of concrete creep on the above actions, it generally plays a role in unloading qualitatively and can be taken into account if there is a clear provision in specifications or if there is a reliable calculation method (such as loss of prestress and system conversion).

For the prestressed concrete continuous beams without system conversion in the construction process, after the completion of the instantaneous losses of prestress, other prestress losses will affect the secondary effect, including the prestress losses due to relaxation of reinforcement, concrete shrinkage and creep, all these will continuously happen until they are completed, and their influences on the secondary effect are complicated to be calculated. The simplified methods to reflect the influence of the various factors on the secondary effects are summarized as follows: the total secondary effect (including elastic deformation and creep) induced by prestressing force can be obtained from the secondary effect due to elastic deformation caused by pre-applied stress multiplying by the average effective coefficient C of the prestressing force. The average effective coefficient is calculated as follows:

$$C = P_{\mathrm{e}}/P_{\mathrm{i}} \tag{4-7}$$

where:

P_e——average prestressing force of the prestressing steel after all the prestress loss is completed;

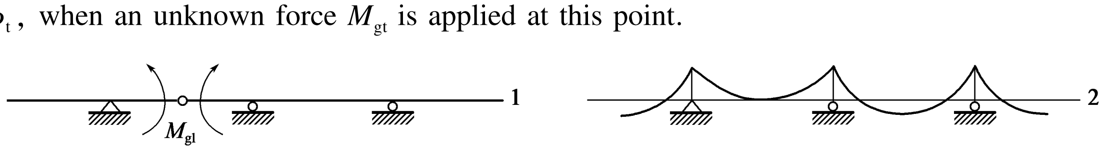
P_i——average prestressing force of the prestressing steel after the instantaneous prestress loss (first batch) is completed.

The secondary effect induced by concrete creep must be considered for a prestressed concrete continuous beam, which is constructed by structural system conversion and the structure after conversion is a statically indeterminate system. The relative calculation is complicated, the simplified method under certain conditions is as follows:

(1) Assume the concreting, prefabrication, and erection of the simply supported or cantilever beams and the conversion of the continuous beam are conducted and completed at the same time τ. From time τ, the concrete creep will be restricted by redundant constraints, which will lead to changes in redundant constraining force and action effect in the structure.

(2) Set the bending moment of the prephase structure induced by self-weight as $M_{1\mathrm{g}}$. If the self-weight of the prephase structure is applied to the anaphase structure, its bending moment is $M_{2\mathrm{g}}$. When the prephase structure is converted to the anaphase structure at τ time, $M_{1\mathrm{g}}$ gradually approaches $M_{2\mathrm{g}}$, and reaches M_{gt} at t time.

Taking the anaphase structure with a hinge at one point as the basic structure (Figure 4-10), the creep increment at the hinged location of the basic structure in dt time induced by self-weight is $\Delta_{\mathrm{g}} \mathrm{d}\phi_{\mathrm{t}}$, when an unknown force M_{gt} is applied at this point.



a)Basic structure b)Diagram of bending moment

Figure 4-10 Redistribution of action (load) effect during system conversion

In the basic structure, the increment of the elastic deformation induced by the increment of the constraint effect $\mathrm{d}M_{\mathrm{gt}}$ in dt time is $\mathrm{d}M_{\mathrm{gt}} \cdot \delta$.

In the basic structure, the increment of the creep deformation induced by M_{gt} in dt time is $M_{gt} \cdot \delta \cdot \mathrm{d}\phi_t$.

The deformation compatibility condition at the hinge of the basic structure in dt time is that the sum of the deformation increments shall be zero, that is:

$$\mathrm{d}M_{gt} \cdot \delta + M_{gt} \cdot \delta \cdot \mathrm{d}\varphi_t + \Delta_g \cdot \mathrm{d}\varphi_t = 0 \tag{4-8}$$

$$\frac{\mathrm{d}M_{gt}}{\mathrm{d}\varphi_t} + M_{gt} = -\frac{\Delta_g}{\delta} = M_{2g} \tag{4-9}$$

where:

Δ_g——elastic deformation (angle) at the hinged location of the basic structure induced by the self-weight;

δ——elastic deformation (angle) at the hinged location of the basic structure induced by the unit bending moment acting on the hinged location.

Eq. (4-9) can be solved as:

$$M_{gt} = e^{-\varphi_t}(M_{2g} \cdot e^{\varphi_t} + c) \tag{4-10}$$

Using the initial conditions, when $t=\tau$, substitute $M_{gt}=M_{1g}$ into Eq. (4-10), we can obtain c as:

$$c = -(M_{2g} - M_{1g})e^{\varphi_t}$$

Substitute the value of c into Eq. (4-10), we can obtain:

$$M_{gt} = e^{-\varphi_t}[M_{2g} \cdot e^{\varphi_t} - (M_{2g} - M_{1g}) \cdot e^{\varphi\tau}] = M_{2g} - (M_{2g} - M_{1g}) \cdot e^{-\varphi_{(t,\tau)}} \tag{4-11}$$

$$\text{or } M_{gt} = M_{2g} + M_{1g} - M_{1g} - (M_{2g} - M_{1g}) \cdot e^{-\varphi_{(t,\tau)}}$$

$$= M_{1g} + (M_{2g} - M_{1g}) \cdot (1 - e^{-\varphi_{(t,\tau)}}) \tag{4-12}$$

(3) Eq. (4-11) and Eq. (4-12) are also applicable for calculating the bending moment redistribution induced by prestressing force, in which substituting the bending moment induced by effective prestressing force of prestressing steel at time t for the bending moment induced by self-weight, that is, substituting M_{1pt} for M_{1g}, and M_{2pt} for M_{2g}:

$$M_{pt} = M_{1pt} + (M_{2pt} - M_{1pt}) \cdot [1 - e^{-\varphi_{(t,\tau)}}] \tag{4-13}$$

$$M_{2pt} = M^0_{2pt} + M'_{2pt} \tag{4-14}$$

$$M_{1pt} = M^0_{1pt} + M'_{1pt} \tag{4-15}$$

$$M^0_{2pt} = M^0_{1pt} \tag{4-16}$$

where:

M_{2pt}——bending moment on the anaphase structure subjected to the prestressing force on the prephase structure (at time t);

M_{1pt}——bending moment calculated on the prephase structure subjected to the prestressing force on the prephase structure (at time t);

$M_{1pt}^0 = M_{2pt}^0$——initial bending moment induced by the prestressing force, which is equal in the prophase and the anaphase structure;

M'_{2pt}——elastic secondary bending moment on the anaphase structure subjected to the prestressing force on the prephase structure;

M'_{1pt}——elastic secondary bending moment on the prephase structure subjected to the prestressing force on the prephase structure, which is taken as zero when the prephase structure is a statically determinate one.

By substituting Eq. (4-14) ~ Eq. (4-16) into Eq. (4-13), we can obtain:

$$M_{pt} = M_{1pt} + (M'_{2pt} - M'_{1pt}) \cdot [1 - e^{-\phi_{(t,\tau)}}] \tag{4-17}$$

When the prephase structure is statically determinate, $M'_{1pt} = 0$, thus Eq. (4-17) can be simplified as

$$M_{pt} = M_{1pt}^0 + M'_{2pt}[1 - e^{-\phi_{(t,\tau)}}] \tag{4-18}$$

In the above formulas, the creep coefficient is considered for the period of time from the loading age τ (also known as system conversion age) of the simply supported beam to the time t the structure being analyzed.

(4) Eq. (4-12) is obtained by assuming that the prefabrication, erection of the simply supported beam, and the conversion of the continuous beam are completed at the same time. If there is a long gap between the loading age of the simply supported beams and the time when theys are converted to a continuous beam, then the creep effect on concrete in this period of the prephase structure shall be considered. If the loading age of the simply supported beams is τ_0 and the time when they are converted to continuous beam is τ, then Eq. (4-12) can be changed to:

$$M_{gt} = M_{1g} + (M_{2g} - M_{1g})(1 - e^{-[\phi_{(t,\tau_0)} - \phi_{(\tau,\tau_0)}]}) \tag{4-19}$$

Similarly, for the calculation of the bending moment redistribution caused by the prestressing force, Eq. (4-17) can be changed as:

$$M_{pt} = M_{1pt} + (M'_{2pt} - M'_{1pt})(1 - e^{-[\phi_{(t,\tau_0)} - \phi_{(\tau,\tau_0)}]}) \tag{4-20}$$

4.3.9 If the concrete compressive stress is less than 0.3 ~ 0.6 times the cubic strength of concrete, it can be considered that the relationship between concrete stress and creep is linear. In general, the compressive stress of concrete does not exceed half of the cubic strength, so it may be considered that creep maintains a linear relationship with the stress.

4.3.10 The temperature gradient due to the nonuniform temperature rise in the cross-section induced by the solar radiation and heat absorption of the beam, is known as the positive temperature gradient. The temperature gradient due to the nonuniform temperature drop in the cross-section induced by the heat reflection and dissipation of the beam after sunset, is known as the negative temperature gradient. No matter whether it is a positive or negative temperature gradient, both of them will result in thermal stresses in the beam cross-section.

4.4 Analysis of Arches

4.4.1 Nowadays, in general only the arch is considered as the main bearing structure in the mechanical analysis of an arch bridge. In terms of structural details, the arch and the spandrel structure shall have a good combination on the premise of not causing excessive constraint effects on the spandrel structures.

According to the technical data of built arch bridges, positive moments induced by the lane loads should be corrected by introducing the reduction coefficient.

4.4.2 At present, the arch axis is selected through a trial and error process: i) assuming the geometrical parameters of the arch; ii) calculating all the coordinate points of the thrust line induced by the dead load along the arch; and iii) checking the deviation of the arch axis from the thrust line and adjust the arch axis to coincide with the thrust line if necessary. The above process should be repeated until the deviation is as small as desired. The thrust lines in the process are obtained by numerical method without considering the elastic shorten. For the arch axis selected by such a method, besides the coordinate points of the arch axis at the sections of the arch crown and arch springing can coincide with the thrust line, the coordinate points in other sections can approximately coincide with the thrust line, therefore, it is an appropriate arch axis.

The load effects caused by the deviation of the arch axis from the thrust line of the dead load should be considered for long-span arch bridges.

4.4.3 This clause is formulated by reference to Clause 5.1.3 in *Code for Design of Highway Masonry Bridges and Culverts* (JTG D61—2005) (hereinafter referred to as *Masonry Code JTG D61*).

4.4.8 The requirement that the lateral (out-of-plane) stability for the arch ring with a width less than 1/20 the arch span shall be checked is taken from Clause 5.1.4 in *Masonry Code JTG D61*). This provision is widely applied on highway and railway bridges. At present, among the arch bridges built in the world, the KrK I Bridge and Šibenik Bridge in Croatia have a relatively small width-to-span ratio of 1/30 and 1/32.8, respectively; there are also several arch bridges in China whose width-span ratio is less than 1/20, for example, the Danhe Bridge with a width-span ratio of 1/26.67. In other words, for an arch ring with a width-span ratio not smaller than 1/20, its lateral stability may not be checked, such a provision has been proved by practices to be feasible and safe for structures.

The lateral stability of the hingeless arch ring should be checked by structural analysis programs, which may also be compared with the following simplified method. In the simplified method, the arch ring may be approximately regarded as a pin-ended axial compression member with a length of $l_0 = r\pi\sqrt{\frac{1}{k}}$, the stability coefficient of axial compression member ϕ is taken from Table 5.3.1 in the *Specifications*, and then the axial compression strength is check by Eq. (5.3.1), in which r is the calculated radius of the circular arch. An arch with other curves can be converted as a circular arch with a radius of r by putting the rise-to-span ratio $\beta = f/l$ into $r = \frac{l}{2}\left(\frac{1}{4\beta} + \beta\right)$, where k is a factor related to the central angle (in radians) of the circular arch α, see Table 4-7.

Table 4-7 Coefficient *k*

α/π	0.25	0.50	1.00
k	60.1	12.6	1.85

In the above method, $l_0 = r\pi\sqrt{\frac{1}{k}}$ is obtained by setting $N_{cr} = kEI_y/r^2$ equal to $\pi^2 EI_y/l_0^2$, in which the former is the critical force of circular hingeless arch under uniform distributed load in radial direction (see Eq. (9-12) in *Manual for Design of Highway: Arch Bridges* (Vol. 1, 1978)) published by China Communication Press (in Chinese), while the latter is the critical force of pin-ended straight bar. The effective length of lateral stability of the hingeless arch with different rise-to-span ratios, l_0, is shown in Table 4-8. For the intermediate value of α/π listed in Table 4-7, it is determined by linear interpolation.

Table 4-8 Effective length for lateral stability of hingeless arch l_0

Rise-to-span ratio f/l	1/3	1/4	1/5	1/6	1/7	1/8	1/9	1/10	Factor
Effective length l_0	1.1665	0.9622	0.7967	0.5759	0.4590	0.4519	0.4248	0.4061	r

For the lateral stability of the arch, it was suggested to calculate by regarding the arch approximately as a straight bar whose length is equal to the length of the arch axis in Clause 2-317 of the second part of the *Technical Specification for Railway Engineering* (1975). This method was also used in the design of highway arch bridges in past time. The effective length l_0 in Table 4-8 is close to the arc length of the arch axis multiplied by a factor of 0.5 for a column with two fixed ends.

Calculation of the lateral stability of braced rib arch is complicated, and thus structural analysis programs should be used for super long and long span arch bridges. The calculated result may be compared with that from the following simplified method. The simplified method at present in designing highway or railway bridges is to approximately regard an arch as a laced straight column whose length is equal to the arc length of the arch axis. The method is introduced as follows:

The arch rib connected with bracings will be treated as a plane truss (Figure 4-11), whose length is equal to the arc length of the arch axis.

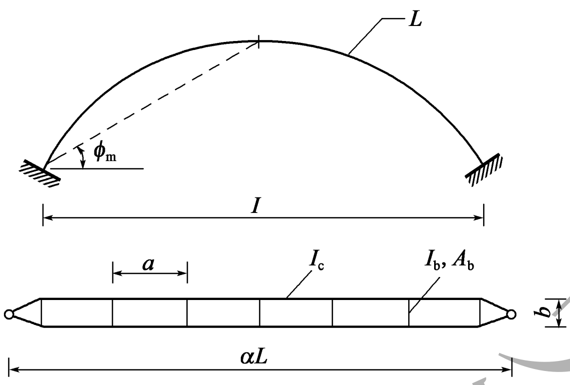


Figure 4-11　Effective length for lateral stability of arch rib

According to the study of Timoshenko, the critical force in the lateral direction (out of plane) of the arch rib connected by dense bracings is:

$$N_{cr} = \alpha_0 \frac{\pi^2 EI}{(\alpha L)^2} \tag{4-22}$$

Let it be equal to the critical force $N'_{cr} = \frac{\pi^2 EI}{l_0^2}$ of a pin-ended straight bar, we can obtain:

$$l_0 = \alpha L / \sqrt{\alpha_0} \tag{4-23}$$

$$\alpha_0 = \frac{1}{1 + \frac{EI\pi^2}{(\alpha L)^2}\left(\frac{ab}{12EI_b} + \frac{a^2}{24EI_c} \times \frac{1}{1-\beta} + \frac{na}{bA_b G}\right)} \tag{4-24}$$

$$\beta = \frac{N_{cr} a^2}{2\pi^2 EI_c} \tag{4-25}$$

where:

l_0——effective length for lateral stability of arch;

α——factor for supporting conditions of arch springing. $\alpha = 1$ for two-hinged arch and $\alpha = 0.5$ for hingeless arch;

L——arc length of arch axis;

EI——product of compressive elastic modulus E and inertia moment I of the arch rib; I is the lateral inertia moment of the two arch ribs about the longitudinal axis of the bridge;

α_0——influence factor of shear force;

a——spacing of bracings (measured along the arch axis);

b——spacing of arch rib axis;

I_b——sectional inertia moment of a bracing about its vertical axis;

I_c——sectional inertia moment of an arch rib about its vertical axis;

A_b——cross-sectional area of the bracing;

n——factor related to the shape of the bracing, which is taken as 1.20 for a rectangular section, and 1.11 for a circular section;

G——shear modulus of the bracing.

In the calculation, the value of β is assumed first and substituted into Eq. (4-24) to solve α_0, which is substituted into Eq. (4-23) to obtain l_0. Then, N_{cr} is calculated by Eq. (4-22) and substituted into Eq. (4-25) to obtained β. If the calculated β is significantly different from the assumed β, it needs to try again until the calculated β is close to the assumed β. Thus, α_0 and l_0 can be calculated by Eq. (4-23) and Eq. (4-24), respectively.

After the effective length l_0 for lateral stability is obtained, the stability factor ϕ can be taken from Table 5.3.1 in the *Specifications*, in which the minimum radius of gyration r shall be that of the section of two arch ribs about the longitudinal axis of the bridge. Then, the strength of the axial compression section can be checked by Eq. (5.3.1).

The critical load obtained may be too large because the nonlinear property of the material is ignored in the calculation above mentioned. Therefore, a certain safety margin is required in the calculation.

4.4.9 This clause is specified with reference to Clause 5.1.6 in *Masonry Code JTG D 61*. In calculating the transverse wind force on the bridge, the total wind force F_{wh} of the whole bridge should be calculated at first; then the uniformly distributed load $q_{1w} = F_{wh}/l$ (l is the effective span) is acted on the hypothesized horizontal straight beam with fixed ends, and the bending moment of $M_{1w} = q_{1w}l^2/12$ at the fixed end is obtained; after that, the uniformly distributed load $q_{2w} = F_{wh}/2f$ (f is the effective rise) is acted on the vertical cantilever beam, and the bending moment of $M_{2w} = q_{2w}f^2/2$ at the fixed end is obtained. In calculating the centrifugal force, the centrifugal force P of vehicles on the whole bridge should be calculated first; for the horizontal straight beam with fixed ends, the uniformly distributed load is $q_{1c} = P/l$ and the bending moment at the fixed end is $M_{1c} = q_{1c}l^2/12$; for the vertical cantilever beam, the concentrated load acted on its free end is $P/2$ and the bending moment at the fixed end is $M_{2c} = Pf/2$. Totally, $M_1 = M_{1w} + M_{1c}$, $M_2 = M_{2w} + M_{2c}$. Substituting them into Eq. (4.4.9), the bending moment M of the arch springing section perpendicular to the curve plane can be obtained.

4.4.12 Truss arch is a two-hinged arch system, a structure with one degree of indeterminacy externally. The action effects on members of the truss arch with hinge joints are close to the experimental results and the calculated results for the arch structure with rigid joints. The sectional strength of the bottom chord shall have a margin of no less than 20% when the effects are calculated by truss arch with hinge joints because the secondary internal forces are not considered.

4.4.13 Rigid frame arch is applicable for lightweight arch bridges with a span of 80 m or less, and even a span of 90 m in individual examples (such as the Yangshan Huaxi Bridge in Guangdong). Its arch rib is composed of arch legs (equivalent to the bottom chord) and a solid member at the mid-span. Based on the arch rib, top chords are set on both sides of the solid member above the arch legs. Secondary inclined legs may be arranged between the middle part of the top chords and the arch springings in order to shorten the flexible span and compression length of the top chord to decrease the bending moments. The two ends of the top chord are supported by movable bearings. The rigid frame arch shall have a strong connection system in the transverse direction.

4.4.14 This clause is specified with reference to Clause 9.2.9 in *Code TBJ 2-85*. The bending moment distribution on the tie and the arch is related to the ratio of their flexural stiffness. When $E_aI_a/E_bI_b < \frac{1}{100}$ (E_aI_a and E_bI_b are the flexural stiffness of the arch and the tie, respectively), the bending moment may be resisted only by the tie. When $E_aI_a/E_bI_b > 100$, the bending moment may be resisted only by the arch. The connection between the arch and the tie may be regarded as a hinged joint due to the great difference in flexural stiffness between them in the two type structures mentioned above.

4.5 Requirements for Durability Design

4.5.2 The environment in which the structure is located is an external factor affecting its durability. In this revision, the environmental category affecting the durability of concrete structures is classified in detail. In structural design, the suitable category of environment may be determined according to the actual situation.

In the current industry standard "*Guidelines for Durability Design of Concrete Structures in Highway Engineering*", according to the corrosion mechanism and the traditional experience of the transportation industry, the environment for concrete structures in highway engineering in China is divided into 7 categories: general environment, freeze-thaw environment, coastal or marine chloride environment, deicing salt and other chloride environments, salt crystallization environment, chemical corrosive environment, and abrasion environment. At the same time, according to the influence degree of different environmental categories on the deterioration and corrosion of concrete structures, the environmental effects are divided into six levels. The *Specifications* presents provisions for environmental categories, considering the characteristics of concrete bridges and the environmental impact on the structure, and referring to the provisions in "*Guidelines for Durability Design of Concrete Structures in Highway Engineering*".

4.5.4 The prestressing steel has disadvantages, such as stress corrosion and hydrogen embrittleness, which are not conducive to durability. The prestressing steel with a small diameter is sensitive to corrosion and has serious damage consequences. Therefore, effective protection measures should be taken for the parts prone to corrosion, such as prestressing steel, connector, anchor holder, and anchorage device.

The improvement of concrete impermeability and frost resistance is beneficial to the durability of concrete structures under harsh environments. The grade division, mix proportion design and test method of concrete frost resistance and impermeability can be executed by referring to the current industry standard "*Guidelines for Durability Design of Concrete Structures in Highway Engineering*".

Moisture is an essential condition for corrosion. Investigations have shown that the concrete corrosion situation of the structure in good ventilation condition is quite different compared to a structure with poor ventilation since moisture and vapor tend to condense on the surface.

5 Ultimate Limit State in Persistent Situations

5.1 General

5.1.1 In this section, the calculation of the ultimate limit state deals with the structures in persistent situation. Ultimate limit state design in persistent situation shall include checking for members on the flexural, compression, tension, shear and torsion strengths, etc., and the stability of members in compression; resistance against overturning and sliding of the structure shall be checked out if necessary. These checkings are the most important part of the structural design.

5.1.2 Eq. (5.1.2-1) in this clause is the general expression for checking on the ultimate limit state of members, which is basically the same as that in the former edition of the *Specifications*.

(1) In the expression, the action effect item is multiplied by the importance factor of the structure, γ_0, which expresses the adjustment for reliability of bridge structures with different safety levels.

(2) In the *Specifications*, the action effect and the material strength index in the expressions are the design values, both include their corresponding partial factors.

(3) The secondary effect caused by prestress should be taken into account in the calculation for the ultimate limit state of statically indeterminate structures such as continuous beams of prestressed concrete. Because it has been experimentally shown that the secondary effect partly or wholly exists at the time when the structures fail. For continuous beam prestressed by mild steel, if the reinforcement ratio is low and the balanced depth of the compression

zone is small, a plastic hinge prone to rotation may be formed at the time when the structure fail, part of the prestressing steel reaches the stage of ductility limit when the structure are failure, secondary effect still exists although majority of it has been disappeared. When hard steel is used as prestressing steel, or mild steel is used but the balanced depth of the compression zone is large, no mature plastic hinge for rotation is formed in the section, and a secondary effect always exists in the structure even after failure.

5.1.3 The basic assumptions about the calculation for resistance on the vertical section in this clause basically follow those provisions in the former edition of the *Specifications*.

(1) The assumption that plane sections remain plane is still used. It has been experimentally shown that the average strain of the cross-section basically conforms to the assumption before and after the longitudinal tensile stress reaches the yield strength if the plastic rotation is limited in some certain range. The employment of the assumption that plane sections remain plane can chains through the calculations for the resistances on cross-sections with various types (including cross-section reinforced around the perimeter) under one-way or two-way loading, which improves the logicality and orderliness of the calculation methods, and makes the calculation equations with clear physical concepts. The assumption that plane sections remain plane is adopted commonly in the major specifications in the world.

(2) The bonding performance between internal steel bars and concrete is good, and their deformation is compatible.

5.1.4 In this clause, the basic assumption about the calculation of the concrete compressive stress on cross-sections in compression zones of flexural and eccentrically loaded members follows the provisions of the former edition of the *Specifications*.

(1) The concrete stress distribution in the compression zone is equivalent to a rectangular stress block. The value of β is the ratio of the depth of the rectangular stress block, x, to the physical depth of the compression zone, x_0. For concrete of C50 or below, $\beta = 0.8$; for C80 or above, $\beta = 0.74$; for concrete between C50 and C80, β may be obtained by linear interpolation.

(2) The equivalent stress of the rectangular stress block equals to the design axial compressive strength of concrete.

5.1.5 Eqs. (5.1.5-1) and (5.1.5-2) in this clause are used to calculate the stress of the internal steel bar. Based on the assumption that the plane sections remain plane and the deformation compatibility condition of reinforcement and concrete, the following relationships are obtained:

Relative compression depth

$$\xi = \frac{x}{h_0} = \frac{\beta \varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_s} \tag{5-1}$$

Strain of reinforcement is

$$\varepsilon_s = \varepsilon_{cu}\left(\frac{\beta}{\xi} - 1\right) \tag{5-2}$$

Therefore, stress of reinforcement is

$$\sigma_s = \varepsilon_s E_s = \varepsilon_{cu} E_s\left(\frac{\beta h_0}{x} - 1\right) \tag{5-3}$$

The stress of prestressing steel is obtained by replacing E_S with E_P in the above equation, plus the existing stress of prestressing steel at the decompression state, σ_{p0}.

The tension and compression stress of reinforcement computed by the above equations are positive and negative, respectively. The applicable conditions are as follows:

Stress of reinforcing steel

$$-f'_{sd} \leqslant \sigma_{si} \leqslant f_{sd}$$

Stress of prestressing steel

$$-(f'_{pd} - \sigma_{p0i}) \leqslant \sigma_{pi} \leqslant f_{pd}$$

5.1.6 In the calculation of flexural resistances of the cross-section and inclined section at the end anchorage zone of pre-tensioned concrete members, the design tensile strength of prestressing steel is taken as zero at the starting point of anchorage and f_{pd} at the end point of the anchorage and is interpolated linearly between the two points. The development length of prestressing steel in Table 5.1.6 of this clause, l_a (mm), is obtained by the calculation result according to the following equation, which is not less than the minimum development length of tension reinforcement.

$$l_a = \alpha \frac{f_{pd}}{f_{td}} d \tag{5-4}$$

where:

f_{pd}——design tensile strength of anchorage reinforcement;

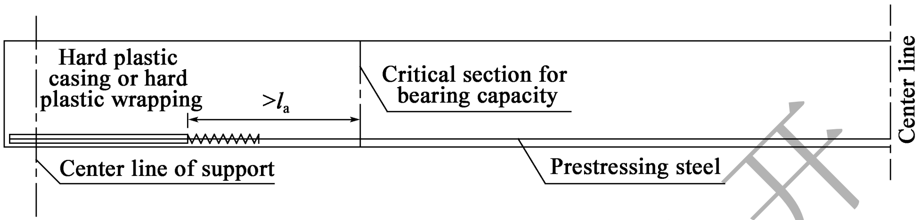
f_{td}——design tensile strength of concrete;

α ——shape factor of anchorage reinforcement. For steel strands of 7 bundles, $\alpha = 0.17$; for spiral ribbed steel wires, $\alpha = 0.13$;

d ——nominal diameter of anchorage reinforcement. An equivalent diameter of $\sqrt{n}d$ is adopted for steel strand bundles, in which n represents the number of single steel strand and d represents the diameter of a single steel strand.

The development lengths of prestressing steel in Table 5.1.6 of this *Specification* are calculated according to a certain design tensile strength of each kind of reinforcement. When the design tensile strength of the reinforcement is changed, the development length shall be increased or decreased in proportion to the strength in the Table. In this revision, the types of prestressing steel are adjusted, see Table 3.2.2-2, and changes have also been made in Table 5.1.6 accordingly.

Generally, hard plastic casing or hard plastic wrapping is provided at the end of prestressing steel for pretensioned members, as shown in Figure 5-1. Here, the ineffective length of prestressing steel shall not be too long. The development length of prestressing steel calculated from the critical section for flexural resistance or shear resistance shall meet the requirements of Table 5.1.6.



re 5-1　Requirement for development length of prestressing steel for pretensioning

5.2　Flexural Members

5.2.1　For a flexural member, if the longitudinal reinforcement in tension and concrete in the compression zone reach their design strength simultaneously, the relative balanced compression depth in the cross-section of the member, ξ_b, may be obtained based on the following equation established by the assumption that plane sections remain plane after deformation:

(1) If the reinforcement is hot rolled reinforcing steel

$$\xi_b = \frac{\beta}{1 + \dfrac{f_{sd}}{\varepsilon_{cu} E_s}} \tag{5-5}$$

(2) If the reinforcement is steel strands or steel wires

$$\xi_b = \frac{\beta}{1 + \dfrac{0.002}{\varepsilon_{cu}} + \dfrac{f_{pd} - \sigma_{p0}}{\varepsilon_{cu} E_p}} \tag{5-6}$$

where:

β ——$\beta = x/x_0$, where x is the depth of rectangular compressive stress block and x_0 is the physical depth of compression zone in flexural members. β is obtained from Table 5.1.4 of the *Specifications*;

f_{sd}, f_{pd}——design tensile strength of reinforcing steel and prestressing steel, respectively;

ε_{cu}——ultimate compressive strain of extreme compression fiber of concrete in flexural member, which is obtained according to the provisions specified in Clause 5.1.5 of the *Specifications*;

σ_{p0}——stress of prestressing steel when normal stress of concrete at the point of the resultant force in longitudinal prestressing steel in the tension zone is equal to zero.

Since all the calculation parameters in Eq. (5-5) are known, ξ_b can be calculated directly and has been listed in the table of the *Specifications*. Since only σ_{p0} is unknown in Eq. (5-6), a certain range of ($f_{pd} - \sigma_{p0}$) may be assumed according to the earlier design experiences to obtain the maximum and minimum values of ξ_b. The concrete strength class has only a slight effect on ξ_b, so the calculated ξ_b can be combined properly. Finally, in determining the values in Table 5.2.1 of the *Specifications*, the calculated minimum values are selected to make the members as ductile as possible. For flexural prestressed concrete members with prestressing threaded bars, according to the earlier design experience, ξ_b is adopted as the same as that in the member prestressed with steel strands or steel wires.

5.2.2 ~ 5.2.6 The formulas for calculating the flexural resistance of flexural members are established according to the following basic assumptions:

(1) In the calculation of the ultimate limit state, the design tensile strength f_{sd} or f_{pd} is taken as the stress of internal reinforcement in the tension zone;

(2) In the calculation of the ultimate limit state, the design compressive strength f'_{sd} or f'_{pd} is taken as the stress of internal reinforcement in the compression zone;

(3) The effective stress after subtracting prestress loss in the service stage, $\sigma_{pe,ex}$, is taken for the stress of the external tendon. Compared with the internal tendon, the external tendon is located outside the concrete section of the box girder, only constrained by the box girder at the anchorage and deviators, its deformation is inconsistent with the cross-section deformation. Thus, there exists the secondary effect induced by the external tendon, that is, the additional prestressing effect caused by the inconsistency between the displacement of the external tendon and the deformation of the box girder, as shown in Figure 5-2.

In the "*Design of External Prestressing in Bridges*" (XU Dong, 2008) published by China Communication Press (in Chinese), the results that the influence of secondary effect on concrete stress under vehicular loading is not more than 3% were presented according to the analyses on the sample bridges, i.e, 30 m and 50 m simply supported beams, 3 × 70 m continuous beams, and (100 + 180 + 100) m continuous spans with fixed superstructure-pier connection. The secondary effect may be reduced by providing deviators, such as $e_1 < e_2$ shown in Figure 5-2. Among them, a vertical displacement restraint device arranged at the maximum deflection point of the box girder is the most effective way to reduce the secondary effect of the external tendons. Therefore, the secondary effect of external tendons can be ignored in the overall analysis of bridges with external tendons. Due to the incompatible deformation between external tendons and concrete sections, the external tendons do not reach the yield strength when the concrete structural members reach the ultimate limit state. In calculating the ultimate flexural resistance of a member, the sum of effective prestress σ_{pe} and stress increment $\Delta\sigma$ is generally taken as the ultimate stress of the external tendon, σ_{pu}. The stress increment is related to the span-to-depth ratio, reinforcement ratio, arrangement of prestressing steel and other factors, and there are some different provisions among specifications in various countries, as shown in Table 5-1. For conservative consideration, it is suggested to refer to the European specification of CEB-FIP 90 to calculate the load-carrying capacity, and the effective stress after deducting prestress loss in the service stage is taken as the stress of the external tendons.

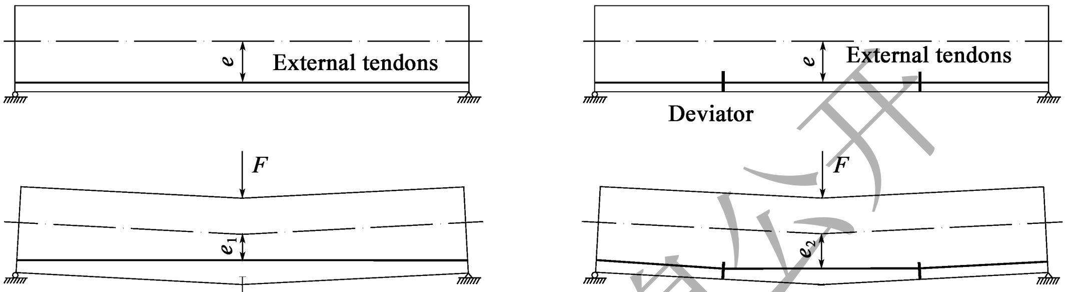


Figure 5-2　Diagram of the secondary effect of external tendons($e_1 < e_2 < e$)

Table 5-1　Provisions of ultimate stress in external tendons at ultimate limit state in some specifications

Specification	U. S. AASHTO LRFD Specifications	Technical Specification for Concrete Structures Prestressed with Unbonded Tendons (JGJ 92—2004)	European CEB-FIB Bulletin 90: Externally applied FRP reinforcement for concrete structures
Ultimate stress	$\sigma_{pu} = \sigma_{pe} + 103\text{MPa}$	$\sigma_{pu} = \sigma_{pe} + \Delta\sigma_p$ $\Delta\sigma_p = (240 - 335\xi_0)(0.45 + 5.5h/l_0)$	$\sigma_{pu} = \sigma_{pe}$

Note: ξ_0-overall index of reinforcement; l_0- effective span of flexural members; h -sectional depth of flexural members.

The calculation equation for the stress of the internal longitudinal reinforcement at any position of the section is provided in Clause 5. 1. 5 of the *Specifications*. When the position of internal longitudinal reinforcement is close to the neutral axis of the section, it is prone to have large errors when the load-carrying capacity is calculated using the above assumptions. Therefore, the stress of internal longitudinal reinforcement at any position shall be calculated according to the actual situation, followed by the calculation of the flexural resistance on cross-section.

To prevent over-reinforced design for a flexural member, the boundary condition of depth of compression zone, $x < \xi_b h_0$, is stipulated in the *Specifications*, in which the relative balanced compression depth ξ_b is listed in Table 5. 2. 1 of the *Specifications*. When reinforcement type and concrete strength class are given, the corresponding tension reinforcement ratio, ρ_b, may be obtained according to ξ_b, which is the limit (maximum) reinforcement ratio for the flexural member. Therefore, the boundary condition of the depth of the compression zone in the cross-section is used to limit the reinforcement ratio for the flexural member. Beyond the boundary condition, the flexural member may be over-reinforced and may cause brittle failure. Generally speaking, if the designed depth of the compression zone cannot satisfy the above requirements, the longitudinal reinforcement in the tension zone is too much or the member depth is not large enough, and adjustment for them is needed; when various types of reinforcements are arranged in tension zone of the member, the smallest ξ_b among all of them corresponding to various types of reinforcements shall be selected for calculation, in order to have certain ductility of the member. However, the boundary condition can only guarantee the ductility of the member theoretically. For a flexural member in which the x is close or equal to $\xi_b h_0$, a balanced failure with obvious brittle failure characteristics may still occur. Therefore, the situation that x and $\xi_b h_0$ are close or equal to each other shall be avoided in actual projects. To avoid brittle failure mode, the reinforcement ratio is limited to a very low level in some specifications in other countries, for example, $\rho < 0.75\rho_b$ is specified in the U. S. specifications.

In the calculation for flexural resistance on the cross-section of flexural members, in order to make the internal longitudinal reinforcement in the compression zone reach the design compressive strength, the *Specifications* stipulates that the depth of sectional compression zone, x, must satisfy the requirements of Eq. (5.2.2-4) or (5.2.2-5) in Clause 5.2.2; if not, it may be calculated approximately according to the equation provided in Clause 5.2.4 or 5.2.6. Eqs. (5.2.4-1), (5.2.4-2), (5.2.6-1) and (5.2.6-2) are established by assuming that the point of application of the compressive force in concrete is consistent with that of the resultant force in longitudinal compression reinforcement, and this point of application is taken as the center of the moment.

5.2.7 Longitudinal tension reinforcements are generally designed according to the requirements from the calculation of the ultimate limit state and serviceability limit state, and detailing requirements. In calculating the depth of the cross-sectional compression zone of concrete, x, if the cross-sectional area of longitudinal tension reinforcements according to the calculation requirements of serviceability limit state and detailing requirements is greater than that required by the calculation of ultimate limit state, we may only consider the longitudinal tension reinforcements required by the calculation of ultimate limit state.

5.2.9 This clause is on checking for shear resistance on the inclined section of flexural members. Compared to the former edition of the *Specifications*, it has the following changes:

(1) The value of $\rho_{sv} f_{sv}$ in calculating V_{cs} of flexural members arranged with vertical prestressing steel is modified. According to the former edition of the *Specifications*, when vertical prestressing steel is adopted, ρ_{sv} and f_{sv} should be replaced by ρ_{pv} and f_{pv} in the calculation equation of V_{cs}, where ρ_{sv} and f_{sv} are the reinforcement ratio and the design tensile strength of vertical prestressing steel, respectively. It remains to be discussed for the requirement in calculating V_{cs} of flexural members with stirrups and vertical prestressing steel provided in the former edition of the *Specifications.* In this revision, it is assumed

that the stress of vertical prestressing steel is the design tensile strength f_{pv} in the ultimate limit state, and the effect of vertical prestressing steel is taken into account, then the reduction coefficient 0.6 is introduced, $\rho_{sv}f_{sv}$ in the calculation equation of V_{cs} is replaczed by($\rho_{sv}f_{sv}+0.6\rho_{pv}f_{pv}$).

(2) For the beam portion with variable depth, the additional shear force caused by the change of position of the cross-sectional centroid is taken into account in the general calculation of the finite element method, so it is not necessary to be considered again. In the revision of the *Specifications*, the provision of "for continuous beam and cantilever beam of reinforced concrete with variable depth (haunch), the effect of additional shear stress shall be taken into account in the beam portion with variable depth" in the note of the former edition of the *Specifications* has been deleted.

(3) When the effective depth h_0 is calculated, the bent bar may not be counted in the longitudinal tension reinforcement.

Some calculation parameters in the calculation formulas of shear resistance are described in detail as follows:

(1) The binomial product is adopted for the contribution of concrete and stirrup to shear resistance in the *Specifications*. The form has been employed in previous *Specifications JTJ 023-85*, it is specified that the calculation formulas of shear resistance of reinforced concrete members are $V_{cs}=0.0349bh_0\sqrt{(2+p)\sqrt{R}u_kR_{gk}}$ (kN), and the unit of cross-sectional dimension is expressed in centimeters. In comparison with other specifications and data, the contribution of longitudinal reinforcement percentage to shear resistance in the formula increases too fast with the increase of reinforcement percentage, then the (2 + p) in the formula is changed to (2 + 0.6p). Considering the change in the standard specimen of concrete and design tensile strength of the stirrup, as well as the change of reinforcement percentage and measurement unit of longitudinal reinforcement, after conversion, the formula for shear resistance of the *Specification*s is changed to $V_{cs}=0.45\times10^{-3}bh_0\sqrt{(2+0.6p)\sqrt{f_{cu,k}}\rho_{sv}f_{sv}}$. The shear resistance of the formula is basically equal to that of previous *Specifications JTJ* 023-85. Compared with general flexural members of *Code for Design of Concrete Structures* (GBJ 10-89) (hereinafter referred to as *Code GBJ 10-89*) and other data, this shear resistance is rather low. When the member is designed by this formula, its inclined crack width in service can be controlled within 0.2mm generally.

(2) Box-girders with overhang flanges or T-beams are commonly used in highway bridges. Shear tests on reinforced concrete beam showed that flanges in compression can improve

the shear resistance of the beam. The ratio of the average actual resistance obtained from the tests on 50 specimens to the resistance calculated according to the equation in the former edition of the *Specifications* are 1.67 and 2.18 for the rectangular beam and T-beam, respectively, which indicates that the shear resistance of the T-beam is 30% higher than that of the rectangular beam due to the increase of shear resistance by the flanges.

According to the specifications in the former Soviet, in the shear calculation of T-beam, the increased coefficient of compression flange was adopted as $\beta_f = 1 + 0.75(b'_f - b)b'_f / bh_0$, but $\beta_f \leqslant 1.2$, and the width of compression flange b'_f was taken not higher than $b + 2h'_f$ or $2b$. According to this provision, the β_f of highway bridges in China is generally greater than 1.2. When the flange width is twice the web thickness, the shear resistance of the T-beam is increased by about 20% compared with that of the rectangular beam, and the influence will be small if the flange width continues to be increased. Therefore, it is suitable to adapt β_f not larger than 1.2. In the *Specifications*, the influence coefficient of compression flange, α_3, is multiplied directly to the shear resistance of concrete and stirrup, while the favorable effect of the compression flange has only little relationship with the stirrup, so that α_3 should be taken as a value smaller than the β_f, thus $\alpha_3 = 1.1$ is adopted in the *Specifications*. Under the conditions of concrete strength class and longitudinal reinforcement ratio commonly used in highway reinforced concrete bridges, the shear resistance on inclined sections of the T-beam calculated by this provision is generally lower than that of general flexural members calculated by *Code GBJ 10-89* and other data.

(3) For shear resistance on inclined sections of a continuous beam, the test results in China and other countries, including the researches by the former Soviet Academy of Sciences and by Tongji University, China, showed that the shear properties of concrete and stirrup of the continuous beam portion close to ending support were the same as those of simply supported beam, and the shear resistance on inclined sections might be calculated according to the provisions of the simply supported beam; the shear resistance of the continuous beam portion close to interior support was affected by opposite-sign bending moment and the value had some decrease. The test results showed that when the generalized shear span-to-depth ratio was large [$m = M/(Vh_0) = 2.67$], two main inclined cracks appeared on both sides of the contraflexure point when the beam failedt, and each of them did not cross the contraflexure point, the stress properties (tension and compression) of longitudinal reinforcement along the top and bottom of the beam was completely consistent with the positive and negative signs of the diagram of bending moment [see Figure 5-3a)]; if the shear span-to-depth ratio was small [$m = M/(Vh_0) = 1.0$], the main inclined crack crossed the contraflexure point and past through the positive and negative bending moment areas [see Figure 5-3b)] when the beam failed. Therefore, stress redistribution occured in the longitudinal reinforcement intersecting with the main inclined crack, the stress was changed from original compression into tension, and the

bond strength along the longitudinal reinforcement failed, resulting in tear cracks, which reduced the bolt effect for shear resistance; compression on concrete in compression zone also increased, which reduced the shear resistance of concrete. These reasons lead to the decrease of shear resistance of continuous beams subjected to opposite-sign bending moments.

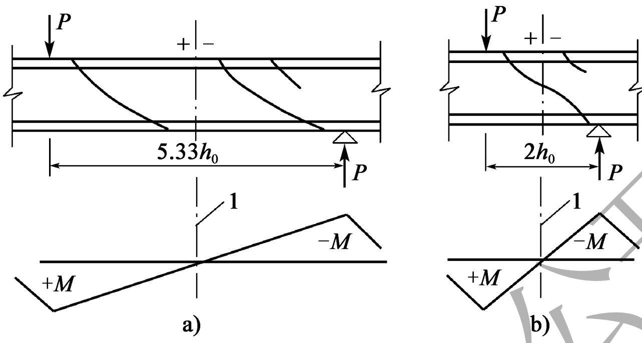


Figure 5-3　Typical shear failure of reinforced concrete beams subjected to opposite-sign bending moments
1-Contraflexure point

Comparing concrete resistance V_c^s with V_c^j , it is found that V_c^s is 12% lower than V_c^j on average, where V_c^s is the test results from 160 beams of constant depth without shear reinforcement ($f_{cu,k}$ = 19.0 ~ 55.9MPa, m = 0.34 ~ 6.0, ρ = 10% ~ 4.76%) in China and other countries, and V_c^j is the concrete resistance of the specimens with the same concrete strength class considering them as simply supported beams calculated according to the previous *Specifications JTJ* 023-85. For another 151 tested beams with shear reinforcement ($f_{cu,k}$ = 19.0 ~ 45.0 MPa, ρ = 0.47% ~ 4.76%, $\rho_{sv}f_{sv}$ = 0.39 ~ 7.51MPa), comparison of test results and analysis results indicates that V_{cs}^j is 7% lower than V_{cs}^j on average.

In order to verify the shear resistance of continuous beams calculated by using α_1 = 0.9, four large-size specimens of two-span continuous beams (5 m + 5 m) were specially tested. The ratio of tested and calculated shear resistance were 2.05 and 1.98 for two continuous beams with constant depth, respectively; and 1.95 and 1.81 for two continuous beams with variable depth, respectively. Here, the tested shear resistance was the maximum shear force measured at the interior support of the continuous beam when the specimen failed; the calculated shear resistance was the sum of the shear resistance of concrete and stirrup calculated according to the previous *Specifications JTJ 023-85* multiplied by the influence coefficient of opposite-sign bending moment, 0.9, and the shear resistance of the bent bar. It is obvious that after considering the influence coefficient of the opposite-sign bending moment, there are still enough safety margins for shear resistance in continuous beams, even if the influence coefficient of the compression flange is considered.

(4) Studies in China and other countries have shown that prestressing can improve the shear resistance of a beam. The main reason is that axial force can prevent or delay inclined crack and its propagation, increase the depth of the shear-compression zone of concrete,

and thus improve the shear resistance of concrete; the length of the inclined crack of prestressed concrete beam is longer than that of the reinforced concrete beam, and shear resistance of stirrup in the inclined crack is also improved.

The increase coefficient of prestressing specified in *Building Specifications* (СНИП2.03.01-84) in the former Soviet Union is as follows:

$$\alpha_2 = 1 + \phi_n \leqslant 1.5 - \phi_f \tag{5-7}$$

$$\phi_n = 0.1\frac{\sigma_{a0}}{f_{tk}} \tag{5-8}$$

where:

ϕ_f——increase coefficient of compression flange, $\phi_f = 0.1$, which indicates that the maximum increase coefficient of pre-stressing is 1.4;

f_{tk}——characteristic tensile strength of concrete;

σ_{a0}——pre-applied axial compressive stress of concrete, for safety, σ_{a0} at the centroid of the rectangular section ($h/2$) is adopted (σ_{a0} at the centroid of T-section and box section is always larger than σ_{a0} of a rectangular section due to the centroid axis moving upward).

According to the provisions of the *Specifications*, the maximum compressive concrete stress at the extreme fiber of the section in service stage is equal to $0.5f_{ck}$, then $\sigma_{a0} = 0.5f_{ck}/2 = 0.25f_{ck}$, and thus $\alpha_2 = 1 + \phi_n = 1 + 0.1\sigma_{a0}/f_{tk} = 1 + 0.025f_{ck}/f_{tk}$. In the applicable range of prestressed concrete beam, the average f_{ck}/f_{tk} of concrete is about 12.3, then $\alpha_2 = 1 + \phi_n = 1 + 0.025 \times 12.3 = 1.30$, and $\alpha_2 = 1.25$ is adopted in the *Specifications*.

Comparing concrete resistance V_c^s with V_c^j, it is found that the average V_c^s / V_c^j is 3.38, where the V_c^s is the shear test result of 52 simply-supported prestressed concrete beams of rectangular section, T-section and I-section without web reinforcement (excluding the influence of stirrup on shear resistance) from China and other countries ($f_{cu,k} = 22.6 \sim 70.0$MPa, $\rho = 0.9\% \sim 3.29\%$, $m = 1.03 \sim 6.7$), and V_c^j is concrete resistance calculated according to the previous *Specifications JTJ* 023-85. For other 30 simply supported beams of prestressed concrete with web reinforcement ($f_{cu,k} = 29.4 \sim 62.5$ MPa, $\rho = 1.58\% \sim 2.63\%$, $\rho_{sv}f_{sv} = 1.036 \sim 3.451$ MPa, $m = 2 \sim 4$), comparison of V_{cs}^s and V_{cs}^j indicates that V_{cs}^s/V_{cs}^j is 2.27, where V_{cs}^s and V_{cs}^j is test and calculated shear resistance contributed by both concrete and stirrups, respectively. It shows that even if the influence coefficient of compression flanges $\alpha_3 = 1.1$ is taken into account, V_c^s / V_c^j and V_{cs}^s / V_{cs}^j are still larger than 1.25, indicating that $\alpha_2 = 1.25$ is safe and desirable for the calculation. However, for prestressed concrete beams permitted to crack, the prestress may disappear when the member reaches the load-carrying capacity; or when the direction of bending moment caused by the resultant force in reinforcement on the section is the same as the external bending moment, the prestressing steel cannot fully contribute to the axial compression effect. In both cases, the beneficial effect of prestress is not considered and $\alpha_2 = 1.0$ is adopted.

(5) Eqs. (5.2.9-3) and (5.2.9-5) in this clause are equations for the shear resistance of bent

bars intersecting with inclined sections. Test results showed that most of the internal bent bars in prestressed concrete continuous beams could reach the yield strength in the failure stage; therefore, the calculation formula for shear resistance of internal bent bars in simply-supported prestressed concrete beams can still be used for continuous beams. The computed results from Eqs. (5.2.9-3) and (5.2.9-4) are close to those from the formulas in the previous *Specifications JTJ* 023-85, therefore, only the design tensile strength of reinforcement and measurement unit are converted in the revision.

Shear performance of externally prestressed concrete beams was tested in the thesis *Calculating Method for Design of Externally Prestressed Concrete Bridges* (in Chinese) by LI Guoping of Tongji University. The experimental results showed that the stress of internal prestressing steel could reach the yield strength when the shear failure occurred in the externally prestressed beams, and the stress increase of the external prestressing steel was small. Therefore, in calculating $V_{pb,ex}$, the effective stress of external prestressing steel at the service stage after deducting the prestress loss, $\sigma_{pb,ex}$, is adopted as the stress of external prestressing steel in the *Specifications*.

5.2.10 The calculation formula (5.2.10) for horizontal projection length C on inclined sections in this clause is the same as that in the former edition of the *Specifications*. The test results showed that the inclined angle of shear failure of the reinforced concrete continuous beam with constant or variable depth was approximately equal to that of the simply supported beam. Thus, the horizontal projection length of inclined sections in reinforced concrete flexural members may be taken as a unified value. For prestressed concrete continuous beams, test results showed that the lengths of main inclined cracks were 1.3 ~ 1.5 times of those in reinforced concrete beam. However, its horizontal projection length of inclined sections is still calculated by the formula for that of the reinforced concrete beam in the *Specifications*. Such a calculation is conservative since the web reinforcement in the region with increased crack length is not considered.

5.2.11 The "upper limit of shear resistance" of Eq. (5.2.11) in this clause is used to prevent too wide diagonal cracks or diagonal compression failure occurring in the reinforced concrete beams. If the requirements of the formula cannot be satisfied in the calculation, the cross-sectional dimensions of the beam or the concrete strength class should be increased.

5.2.12 The "lower limit of shear resistance" of Eq. (5.2.12) in this clause is used to determine the threshold between beams with and without web reinforcement. When the beam or a beam portion satisfies the requirements of the equation, the stirrup may be designed only according to the detailing requirements. The beneficial effect of prestressing on prestressed concrete members is considered in Eq. (5.2.12). However, in the calculation for prestressed concrete flexural members permitted to crack or structures in which the direction of the bending moment caused by the resultant force in reinforcement is the same as that of the external bending moment, the beneficial effect shall not be considered.

5.2.13 The clause specifies the design method of shear reinforcement for simply-supported reinforced concrete beams, continuous and cantilever beams with constant or variable depth (haunched). In the method, the stirrup spacing, bent bar and bending-up starting points are determined according to the envelope diagram of the design shear force. In the *Specifications*, more than 60% of the maximum design shear force should be resisted by concrete and stirrups, while no more than 40% is resisted by bent bars.

Generally, bent bars are not arranged in flexural prestressed concrete members, and the only design work for shear reinforcement is to determine stirrup spacing. As long as the design shear force caused by curved tendons is subtracted from the maximum design shear force caused by the loads, it can be calculated as the same as that of flexural reinforced concrete members. For example, for simply supported or continuous prestressed concrete beams, the stirrup spacing s_v (mm) may be calculated according to the following formula:

$$s_v = \frac{0.2 \times 10^{-6} \alpha_1^2 \alpha_2^2 \alpha_3^2 (2 + 0.6p) \sqrt{f_{cu,k}} A_{sv} f_{sv} b h_0^2}{(\gamma_0 V_d - V_{pb})^2} \tag{5-9}$$

Where:

V_d——design shear force caused by action and used for reinforcement design (kN), which is taken in accordance with the provisions in this clause;

V_{pb}——design shear resistance of internal curved tendons intersecting with the inclined sections (kN), which is calculated by Eq. (5.2.9-4) of the *Specifications*.

5.3 Compression Members

5.3.2 This clause follows the provisions in the former edition of the *Specifications*. Test results from other countries showed that the strength improvement value induced by lateral pressure for 80 ~ 100 MPa high-strength concrete was about 25% lower than that in conventional strength concrete. Tests from China on high-strength concrete-filled steel tubular columns also showed that the hooping coefficient for concrete core with 80 MPa strength was 1.8, while that for conventional strength concrete was 2.0 ~ 2.1. It can be considered that the hooping coefficient decreases with the increase of the concrete strength class. In the *Specifications*, for concrete strength class equal to or lower than C50, $k = 2.0$; for concrete strength class between C50 and C80, $k = 2.0 \sim 1.7$.

5.3.3 The relative balanced depth of the compression zone, ξ_b, is given in this clause to judge whether the eccentricity is large or small for eccentrically loaded compression members. Its calculation equation is derived from the assumption that the plane sections remain plane after deformation, the same as that of ξ_b for a flexural member to judge whether it is over-reinforced. For eccentrically loaded reinforced compression concrete members, the parameters of concrete and reinforcement in the calculation equation are known; therefore, the value of ξ_b from the calculation for flexural members may be adopted. For eccentrically loaded compression prestressed concrete members, σ_{p0} is unknown in the calculation equation, which can only be obtained after the prestressing steel is designed and other conditions are assumed. If the calculation method for flexural members is adopted, i. e., σ_{p0} is assumed in advance, and ξ_b is calculated and listed in the *Specifications*, then the following situation may occur in the calculation for specific members: large eccentricity is assumed for a compression member, but small eccentricity is obtained from the calculation, vice versa, small eccentricity is assumed, but large eccentricity is calculated. Therefore, the calculation equation of ξ_b for eccentrically-compressed prestressed concrete members is provided in the *Specifications*, so that designers can calculate it according to the specific conditions of the members.

5.3.4 The fundamental formulas for calculating the compressive resistance of eccentrically loaded compression members and the principles for identification of eccentrically loaded compression members with small or large eccentricity are the same as those in the former edition of the *Specifications*.

This clause provides the approximate calculation formula for the cross-sectional area of reinforcement in compression members of rectangular sections with small eccentricity and reinforced symmetrically, aims to calculate directly the required cross-sectional area of reinforcement in its reinforcement design. In highway bridges, there are many eccentrically-compressed reinforced concrete members (eccentrically-compressed prestressed concrete members are rarely used), among them many have rectangular sections reinforced symmetrically. This formula for calculating A_s (or A'_s) derived from the fundamental equation of eccentrically loaded compression members provides a convenient approach for the design of such members.

In checking for eccentrically loaded compression members, a formula for calculating the position of the neutral axis (the depth of the compression zone) was provided in *Specifications JTJ 023-85*. The formula is obtained by taking the moment of the cross-sectional internal force with respect to the point of application of the axial force. There are also other methods to obtain the position of the neutral axis, for example, to obtain the depth of compression zone x according to Eqs. (5.3.4-1) and (5.3.4-2) in the *Specifications*, herein, equal signs are adopted in the two formulas. Therefore, such formulas are not provided in the *Specifications*, and it is not necessary to specify the formula to be used, and the calculation is to be carried out by the consideration of the designer.

For most of the eccentrically loaded reinforced concrete compression members in highway bridges, it may be more convenient to design reinforcement than to check the compressive resistance after reinforcement is designed. In reinforcement design, when $\eta e_0 \leq 0.3h_0$, it can be calculated as compression members with small eccentricity. When $\eta e_0 > 0.3h_0$, it can be firstly calculated as a compression member with large eccentricity, and the cross-sectional area of tension reinforcement must be greater than the minimum reinforcement ratio specified in Clause 9.1.12 of the *Specifications*, otherwise, the cross-sectional area of reinforcement is to be calculated as a compression member with small eccentricity. The criterion is not necessarily applicable for eccentrically loaded and symmetrically reinforced compression members. When $A_s = A'_s$, it can be directly identified by the fact that the axial force $\gamma_0 N_0$ is equal to the compressive force of concrete in the compression zone. For a rectangular section, if $\gamma_0 N_0 \leqslant f_{cd}\xi_b h_0$, it is a compression member with large eccentricity; and if $\gamma_0 N_0 > f_{cd}\xi_b h_0$, it is a compression member with small eccentricity.

5.3.7 This clause is developed with reference to *Code GBJ 10-89*. The resistance on cross-section of eccentrically loaded compression members with longitudinal reinforcement evenly arranged in the web consists of two parts: one is the resistance contributed by concrete and the upper and lower longitudinal reinforcement A_s and A'_s; the other is the resistance contributed by the longitudinal reinforcement A_{sw} evenly arranged in the web.

The former is calculated in the same way as the general eccentrically loaded reinforced concrete compression members. By using Eqs. (5.3.5-1) and (5.3.5-2) of the *Specifications*, the following results can be obtained through simple conversion:

Axial force $N_{cs} = f_{cd}[\xi b h_0 + (b'_f - b) b'_f] + f'_{sd}A'_s - \sigma_s A_s$

Bending moment $N_d e = f_{cd}[\xi(1 - 0.5\xi) b h_0^2 + (b'_f - b) h'_f (h_0 - \frac{h'_f}{2})] + f'_{sd}A'_s(h_0 - a'_s)$

The latter may be calculated according to the fundamental assumption, equilibrium equation and deformation compatibility condition, but the calculation process is tedious and inconvenient for design and application. In general, the simplified method is adopted. The longitudinal reinforcements in the web are required to be arranged with equal diameter and spacing. No less than 4 rebars shall be arranged in one row. In calculation, the rebars are converted into a steel strip with a cross-sectional area A_{sw} and a depth S_{w0}. According to the fundamental assumptions of Clauses 5.1.3 to 5.1.5, the calculation diagram of such members can be drawn up, as shown in Figure 5-4.

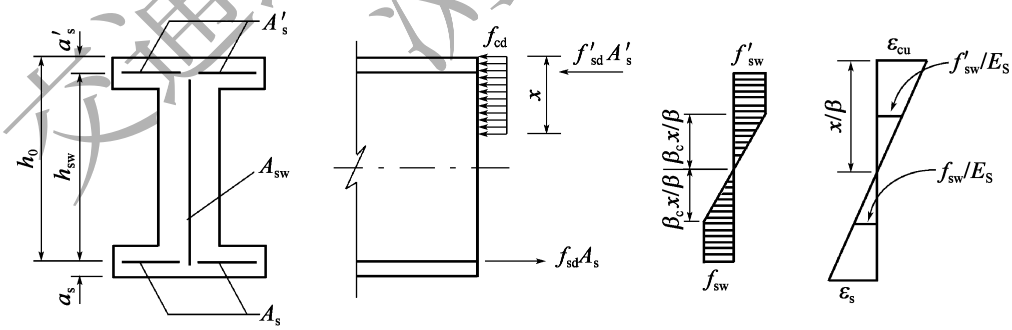


Figure 5-4 Calculation for the resistance of eccentrically loaded compression member with reinforcement evenly arranged along the depth of the cross-section

Let the distance between the fiber and the neutral axis be $\beta_c x/\beta$ when the strain of the evenly arranged reinforcing steel (steel strip) reaches yield strength, it can be obtained from Figure 5-4 that:

$$\frac{f_{sw}/E_s}{\varepsilon_{cu}} = \frac{\beta_c x/\beta}{x/\beta} = \beta_c \tag{5-10}$$

$$\beta_c = \frac{f_{sw}/E_s}{\varepsilon_{cu}} \tag{5-11}$$

The value of β_c is related to the type of reinforcing steel. After the selection of the type of evenly arranged reinforcing steel, β_c is a constant value. For common reinforcing steel, β_c may be approximately adopted as 0.4, which has only little effect on the resistance of the members. For reinforced concrete members, the concrete strength class is generally not greater than C50, therefore, β may be adopted as 0.8.

If $\xi \leqslant \xi_b$, it can be obtained as a compression member with large eccentricity as follows:

Axial force

$$N_{sw} = (1 + \frac{\xi - \beta}{0.5\beta\omega}) f_{sw} A_{sw}$$

Bending moment

$$M_{sw} = [0.5 - \frac{(\xi - \beta)^2 + \frac{1}{3}(\beta_c \xi)^2}{(\beta\omega)^2}] f_{sw} A_{sw} h_{sw}$$

If $\xi > \xi_b$, it can be obtained as a compression member with small eccentricity:

Axial force

$$N_{sw} = \left\{1 - \frac{[\beta - (1 - \beta_c \xi]^2}{1.6\omega\beta_c \xi}\right\} f_{sw} A_{sw}$$

Bending moment

$$M_{sw} = \left\{0.5 - \frac{[\beta - (1 - \beta_c)\xi]^3}{3.85\omega^2 \beta_c \xi}\right\} f_{sw} A_{sw} h_{sw}$$

The above formulas for N_{sw} and M_{sw} can approximately be fitted as a straight line and a quadratic curve, respectively. By subtituting $\beta_c = 0.4$ into the fitted line and curve, we have:

$$N_{sw} = (1 + \frac{\xi - \beta}{0.5\beta\omega}) f_{sw} A_{sw}$$

$$M_{sw} = [0.5 - (\frac{\xi - \beta}{\beta\omega})^2] f_{sw} A_{sw} h_{sw}$$

Finally, the resistance of the two parts are added together, then:

$$\Sigma N = N_{cs} + N_{sw}$$

$$\Sigma M = N_d e + M_{sw}$$

Negative and positive values of N_{sw} represent tension and compression, respectively. Negative M_{sw} has the same direction as N_de, and positive M_{sw} has the opposite direction to N_de.

5.3.8 To simplify the calculation method for resistance of eccentrically loaded compression members with circular section, the calculation formulas in the former edition of the *Specifications* are modified referring to the current *Code GB 50010*. It can be found by comparison that the fundamental principles of the two formulas are consistent. Different simplified methods lead to different expressions, while the calculation results of the two formulas are similar. When the formula in this clause is employed, the resistance may be determined by calculation with trial-and-error and iteration methods for α.

5.3.9 For eccentrically loaded compression members with a large slenderness ratio, there is no simple method to calculate the second-order bending moment caused by member deflection under vertical load. Therefore, most specifications in China and other countries adopt the simplified calculation method, in which the influence of second-order bending moment on cross-sectional resistance is considered by introducing together the amplification factor for eccentricity, η, and effective length of member, l_0. The basic idea of the method is: firstly, based on the standard compression pin-ended member with equal eccentricity, the expression of amplification factor for eccentricity at the mid-length section of the standard member, η, is given according to the experimental study; then, the effective length l_0 is used to reflect the length of the standard member corresponding to each type of eccentrically loaded compression member with various end conditions. In this way, the second-order bending moment calculated by the standard member with a length of l_0 can be close to the actual one in the critical section of the member. This method is simple in calculation but approximate for the results, in which l_0 can only be determined according to practical experience and reference from some theoretical analysis results.

The amplification factor for eccentricity in a standard pin-ended compression member under the action of vertical load and with equal eccentricity e_0 at the two ends may be expressed as follows:

$$\eta = \frac{e_0 + f_{\max}}{e_0} = 1 + \frac{f_{\max}}{e_0} \tag{5-12}$$

The expression η provided in this clause is based on the theory of limit curvature. The maximum deflection of the mid-length section of the member in Eq. (5-12), $f_{\max}$, may be obtained by the integral method:

$$f_{\max} = \frac{l_0^2}{\beta r_c} \tag{5-13}$$

$$\eta = 1 + \frac{1}{e_0}\left(\frac{l_0^2}{\beta r_c}\right) \tag{5-14}$$

where β is the coefficient related to the curvature distribution of the member. When the curvature distribution conforms to the sine curve, $\beta = \pi^2 \approx 10$; $1/r_c$ is the ultimate curvature of the critical section, which depends on the strain of the tension reinforcement and the concrete at the extreme compression fiber of the critical section.

For compression members with large eccentricity, the test results show that the ultimate curvature of critical compression situation may be approximately adopted when the member reaches the ultimate limit state; when the effect of long-term load is taken into account, according to the assumption that plan sections remain plane after deformation, it may be expressed as:

$$\frac{1}{r_c} = \frac{\phi\varepsilon_{cu} + \varepsilon_y}{h_0} \tag{5-15}$$

where:

ε_{cu}——ultimate compressive strain of extreme compression fiber of concrete, $\varepsilon_{cu} = 0.0033$;

ε_y —— strain of tension reinforcement corresponding to its yield strength, the strain corresponding to the characteristic tensile strength of HRB400 reinforcement is adopted, i. e. , $\varepsilon_y = 0.0020$;

ϕ ——strain increase coefficient caused by the creep of concrete under long-term loading, $\phi = 1.25$.

Under the critical condition, the influence of load eccentricity and slenderness ratio on curvature (see below) is expressed by ξ_1 and ξ_2, respectively, then

$$\eta = 1 + \frac{1}{e_0}\left(\frac{\phi\varepsilon_{cu} + \varepsilon_y}{h_0} \cdot \frac{l_0^2}{\beta}\right)\zeta_1\zeta_2 \tag{5-16}$$

By substituting the above values and taking $h \approx 1.1h_0$, the equation for calculating η in the eccentrically loaded compression member can be obtained as follows:

$$\eta = 1 + \frac{1}{1300e_0/h_0}\left(\frac{l_0}{h}\right)^2\zeta_1\zeta_2 \tag{5-17}$$

ξ_1 in Eq. (5-17) is the correction coefficient for the section curvature, which mainly depends on the relative eccentricity e_0/h_0, therefore, it is also called the influence coefficient of load eccentricity on section curvature. As mentioned above, the calculation equation of η is established based on the critical curvature of the critical section, and the compression member with large eccentricity conforms to this premise. However, for members with non-critical conditions, such as compression members with small eccentricity, the premise is not satisfied. In the ultimate limit state, the stress of tension reinforcement in compression members with small eccentricity cannot reach the yield strength, and the ultimate compressive strain of the extreme compression fiber of concrete will also decrease with the increase of the depth of compression zone, then, the cross-sectional curvature will decrease with the increase of the axial compressive force. Therefore, ξ_1 is needed to be introduced for modification. The calculation equation of ξ_1 is derived from *Specifications* GBJ10-89, which is established on the condition that the equation almost needs no modification when $e_0/h_0 = 0.3$.

The ξ_2 in Eq. (5-17) is the influence coefficient of slenderness ratio on section curvature. Test results show that with the increase of the slenderness ratio, the curvature of the critical section will decrease when the member reaches the ultimate limit state. Therefore, $\xi_2 = 1.15 \sim 0.01 l_0/h$ is introduced for modification. The application range of the equation is $15 \leqslant l_0/h \leqslant 30$. When $15 < l_0/h$, the effect is not significant, it is not necessary to modify, taking $\xi_2 = 1$; when $l_0/h > 30$, the failure mode of the members will change from material failure to instability failure, which is out of the scope for consideration. When $l_0/h = 30$, $\xi_2 = 0.85$, it is the minimum value.

It was stipulated in the previous *Specifications* JTJ 023-85 that the effect of the second-order bending moment was only to be taken into account when the slenderness ratio of the member was greater than 28 ($l_0/i > 28$ or $l_0/h > 8$). Referring to the specifications of industrial and civil structures in China and the relevant specifications in other countries, the critical condition of $l_0/i \leqslant 17.5$ (or $l_0/h \leqslant 5.0$) in the former edition of the *Specifications* continues to be used in the *Specifications*, under which the second-order bending moment is not taken into account.

5.4 Tension Members

5.4.3 ~5.4.4 These two clauses refer to the provisions of current *Code GB 50010*.

For eccentrically loaded tension circular members with longitudinal reinforcement evenly distributed along a perimeter, the resistance basically conforms to the variation law of $\frac{N_d}{N_{ud}} + \frac{M_d}{M_{ud}} = 1$, which will give somewhat conservative results and is rewritten as Eq. (5.4.4). Test results showed that the rewritten formula was also applicable to the biaxial eccentrically-tensioned reinforced concrete members of rectangular sections with symmetrical reinforcement. By substituting $\frac{e_0}{M_{ud}} = \sqrt{\left(\frac{e_{0x}}{M_{ux}}\right)^2 + \left(\frac{e_{0y}}{M_{uy}}\right)^2}$ into Eq. (5.4.4), Eq. (5.4.3) is obtained.

5.5 Torsion Members

5.5.1 Nowadays, there are two calculation theories and models for calculating the ultimate torsion of pure torsion members with rectangular sections: the variable angle space truss model and the skew-bending model. When the ratio of wall thickness to the corresponding wall depth (or wall width) reaches a certain value, the box section may also be calculated as a rectangular section. The same ultimate torsion moment can be obtained using the above two models.

$$T_u = 2\sqrt{\zeta}\frac{f_{sv}A_{sv1}A_{cor}}{s_v} \tag{5-25}$$

where ζ is the ratio of the strength of longitudinal reinforcement to that of the stirrup in pure torsion member, i. e. :

$$\zeta = \frac{f_{sd}A_{st}s_{vr}}{f_{sv}A_{sv1}U_{cor}} \tag{5-26}$$

The ζ geometrically represents the inclination angle α between the failure crack and the longitudinal axis in the torsion member. In the previous *Specifications JTJ 023-85*, α is generally assumed to be about 45°, while in the variable angle space truss model, α is taken as the inclination angle of the diagonal compression strut. It is not a constant of 45° and is calculated according to the following equation:

$$\tan\alpha = \sqrt{\frac{1}{\zeta}} = \sqrt{\frac{f_{sv}A_{sv1}U_{cor}}{f_{sd}A_{st}s_v}} \tag{5-27}$$

When $\alpha = 45°$ and $\zeta = 1$, according to Eq. (5-26):

$$\frac{f_{sv}A_{sv1}}{s_v} = \frac{f_{sd}A_{st}}{U_{cor}}$$

Substituting the above relation into Eq. (5-25), the following two forms can be obtained:

$$T_u = 2\frac{f_{sv}A_{sv1}A_{cor}}{s_v} \tag{5-28}$$

$$T_u = 2\frac{f_{sd}A_{st}A_{cor}}{U_{cor}} \tag{5-29}$$

The symbols in the above forms are replaced by symbols in the previous *Specifications JTJ 023-85*, then Eqs. (5-28) and (5-29) are changed into Eqs. (4.1.23-1) and (4.1.23-2) in Clause 4.1.23 of the previous *Specifications JTJ 023-85*. In other words, the equations for calculating the ultimate torsion moment of pure torsion members with the rectangular section in the previous *Specifications JTJ 023-85* were derived based on the assumption of $\alpha = 45°$ and $\zeta = 1$. However, the torsional resistance of concrete is not taken into account in the formulas of the previous *Specifications JTJ 023-85* and Eq. (5-25) of the *Specifications*.

Test results show that the calculation equation in the previous *Specifications JTJ 023-85* is conservative when the reinforcement ratio is low, because the torsional resistance of concrete is not considered in the equation; while the calculated value is high when the reinforcement ratio is high, because the longitudinal reinforcement and stirrup generally cannot yield at the same time in the member as assumed in the equation. Therefore, the calculation equation in the previous *Specifications* is necessary to be modified. In addition to the correction of the inclination angle α of spiral failure crack, many scholars suggested that the influence of concrete strength observed in tests should be reflected in the calculation equation for ultimate torsion, and the following calculation mode was recommended to be adopted:

$$T_u = T_c + \alpha_1\frac{f_{sv}A_{sv1}A_{cor}}{s_v} \tag{5-30}$$

On the right-hand side of the Eq. (5-30), the first item T_c is the torsional resistance of concrete, and the second item is the torsional resistance of reinforcement. The two items of Eq. (5.5.1-1) provided in this clause were derived from *Code GBJ 10-89*. Torsional moment T_{cr} (50% of $0.7f_{td}W_t$) at cracking was adopted for the first item to reflect the torsion resistance of concrete and the coefficient α_t of the second item was adopted as $1.2\sqrt{\zeta}$ to reflect the torsional resistance of reinforcement. The lower value of the test data was adopted as the total torsional resistance. The test result showed that if ζ in the equation is in the range of 0.5 ~ 2.0, the longitudinal reinforcement and stirrup can basically yield at the same time when the reinforced concrete member fails. For the sake of safety, the limiting condition is adopted as $0.6 \leqslant \zeta \leqslant 1.7$, and it is most possible for reinforcement in the member to reach yield when ζ is about 1.2. For members with asymmetrically arranged longitudinal reinforcement, only the cross-sectional area of the symmetrically arranged longitudinal reinforcement is taken into account in the calculation in order to balance the internal force of the cross-section.

The resistance calculation for the reinforced concrete member with a box section under pure torsion, the *Specifications* refers to specifications of other countries. Here, the first item of torsional resistance of the concrete is reduced by multiplying by β_a (the smaller value between $4\frac{t_2}{b}$ and $4\frac{t_1}{h}$ is adopted, and it shall not exceed 1).

Test results of prestressed concrete members under pure torsional moment showed that the precondition of improvement for torsional resistance by prestressing was that the longitudinal reinforcement should not be yielded. When the normal stress of concrete caused by prestress does not exceed the specified limit value, the torsional resistance of pure torsion member may increase by $0.08\dfrac{N_{p0}}{A_0}W_t$. Considering the actual uneven distribution of stress and other adverse effects, only the increased value of $0.05\dfrac{N_{p0}}{A_0}W_t$ is adopted in the *Specifications*, and it is only applicable to the members with eccentricity $e_{p0} \leqslant h/6$. In calculating ζ, the effect of prestressing steel is not taken into account.

Test results also show that the beneficial effect of prestressing on resistance shall not be considered too much in the calculation, therefore, when $N_{p0} > 0.3 f_{cd}A_0$, $N_{p0} = 0.3 f_{cd}A_0$ shall be adopted.

For a reinforced concrete structure or member, if the torsion is directly caused by load and can be obtained by static equilibrium conditions, it is generally called balanced torsion. If the torsion is caused by the restraint against the rotation between structures or adjacent members and the torsion can be determined by the continuous conditions of rotational deformation, it is generally called coordinated torsion or additional torsion. The continuous deformation of the latter may cause the redistribution of internal forces and reduce the designed torsion. The coordinated torsion and additional torsion are not taken into account in the calculation of torsion members in this section, in other words, the calculation of torsion members of the *Specifications* is only applicable to members subjected to balanced torsion.

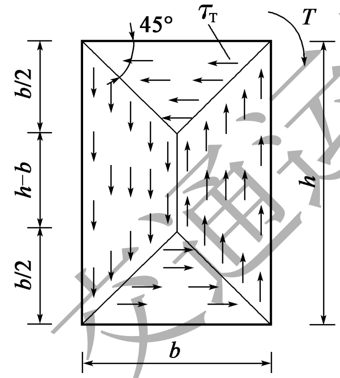


Figure 5-5 Diagram of shear flow distribution

5.5.2 Eq. (5.5.2-1) in this clause is obtained by taking the moment of the shear flow τ_t against the torsion center of the section under the assumption that when a rectangular section of reinforced concrete member enters a fully plastic state, a boundary distribution area of shear stress of 45° to each side of the section appears. From the equilibrium conditions, the following equation may be obtained (Figure 5-5):

$$T = \left\{2\frac{b}{2}(h-b)\frac{b}{4} + 4\frac{b}{2}\frac{b}{2}\frac{1}{2}\frac{b}{3} + 2\frac{b}{2}\frac{b}{2}\left[\frac{2}{3}\frac{b}{2} + \frac{1}{2}(h-b)\right]\right\}\tau_t = \frac{b^2}{6}(3h-b)\tau_t$$

Hence,

$$W_t = \frac{b^2}{6}(3h-b) \tag{5-31}$$

The plastic section modulus of the box section to resist torsion is calculated from the difference between those of two solid rectangular sections, one has the same profile as the box section and the other has the hollow area of the box. The plastic section modulus of both solid rectangular sections to resist torsion is calculated by the above equation.

5.5.3 Test results showed that the concrete would be crushed while the steel bars did not yield if too much torsional reinforcement was arranged. Therefore, the minimum dimensions of the member cross-section must be specified to make the shear stress of concrete not exceed a certain limit value, which is similar to the upper limit value in the calculation for shear resistance on an inclined section. For members subjected to combined flexure, shear and torsion, due to the complexity of the mechanical behaviors, only it can be done nowadays to add the shear stress caused by torsion to the shear stresses caused by bending-shear, and limit the sum of shear stress not exceed the specified limit of concrete strength. The limit value specified in this clause is close to that in the previous *Specifications JTJ 023-85*. In design, when shear stresses caused by shear-torsion exceed the limit value specified in Eq. (5.5.3-1) of the *Specifications*, the cross-sectional dimensions shall be enlarged or the higher concrete strength class shall be used in members.

Eq. (5.5.3-2) in this clause is similar to the lower limit value in the calculation for shear resistance on the inclined section of a member. If the result calculated by the equation meets the requirement of the limit value, torsional reinforcement may not be designed in the member. However, in order to prevent brittle fracture and to ensure the ductility of members during failure, reinforcement shall still be arranged according to the detailing requirements of Clause 9.3.13 of the *Specifications*. The limit value specified in Eq. (5.5.3-2) is close to that specified in the previous *Specifications JTJ 023-85*.

5.5.4 Nowadays, in calculating the resistance of reinforced concrete shear-torsion members, the general approach is to calculate the torsion resistance and shear resistance of the member separately, and then to superimpose them together. However, for members subjected to combined shear and torsion, shear and torsion have a certain effect on both concrete and stirrup in the member. If a simple superposition method is adopted, it will be unsafe for the stirrups and concrete, especially the concrete. Test results showed that under the combined shear and torsion, both shear and torsion stresses would exist in certain compression zones, which would reduce the shear and torsional resistance of concrete in the member. Due to the complex mechanical behavior of torsional members, the resistance of stirrups is simply superimposed at present, a reduction coefficient β_t is introduced for the concrete resistance in the calculation equations for resistances of the shear and torsion members. Eq. (5.5.4-3) for calculating β_t in the *Specifications* was taken from *Code GBJ 10-89*. It was obtained and simplified from the tested interaction curve of shear-torsion resistance based on the precondition that the calculated curve be close to the tested one, which is explained as follows:

An experimental study on shear-torsion members with or without web reinforcement indicated that the tested concrete shear-torsion interaction curve with the dimensionless coordinate (T_c/T_{co}, V_c/V_{co}) has a shape similar to 1/4 circle. Accordingly, the interaction curve of concrete strength in shear-torsion members may be plotted, as shown in Figure 5-6. The T_c and V_c mentioned above are the torsional and shear resistance which can be resisted by concrete in shear-torsion members with web reinforcement, respectively; T_{co} and V_{co} are torsional strength and shear strength which can be resisted by concrete in pure torsion member and a flexural member both have web reinforcement, respectively. In order to simplify the calculation, the 1/4 circular curve *EF* in Figure 5-6 is approximately replaced by trilinear lines of *EG*, *GH* and *HF*, and *GH* is extended to intersect coordinate axes *C* and *D*, and $\angle OCD = 45°$ is adopted, then $CE = DF = b$.

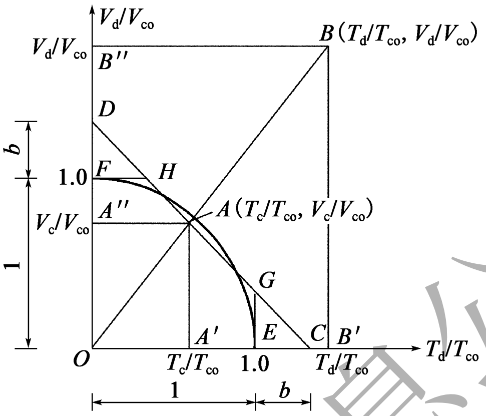


Figure 5-6 The interaction curve of concrete strength in shear-torsion member

The following equation can be obtained from $\triangle AA'C$:

$$\frac{\dfrac{V_c}{V_{co}}}{1 + b - \dfrac{T_c}{T_{co}}} = 1$$

Then,

$$V_c = 1 + b - \frac{T_c}{T_{co}} \tag{5-32}$$

Because $\triangle 0AA'' \approx \triangle 0BB''$,

$$\frac{\dfrac{V_c}{V_{co}}}{\dfrac{V_d}{V_{co}}} = \frac{\dfrac{T_c}{T_{co}}}{\dfrac{T_d}{T_{co}}}$$

Therefore,

$$\frac{V_c}{V_d} = \frac{T_c}{T_d} \tag{5-33}$$

Substituting Eq. (5-32) into Eq. (5-33), then

$$\frac{T_c}{T_d} = \left[1 + b - \frac{T_c}{T_{co}}\right] \frac{V_{co}}{V_d}$$

It can be rewritten as:

$$T_c = \frac{(1 + b) V_{co}}{\dfrac{V_d}{T_d} + \dfrac{V_{co}}{T_{co}}} \tag{5-34}$$

Substituting Eq. (5-34) into Eq. (5-33), then

$$V_c = \left[(1+b) - \frac{(1+b)}{\frac{V_d}{T_d} + \frac{V_{co}}{T_{co}}} \frac{V_{co}}{T_{co}}\right] V_{co} = \left[(1+b) - \frac{(1+b)}{1 + \frac{V_d}{T_d}\frac{T_{co}}{V_{co}}}\right] V_{co}$$

Let

$$\beta_t = \frac{1+b}{1 + \frac{V_d}{T_d}\frac{T_{co}}{V_{co}}} \tag{5-35}$$

Thus,

$$V_c = (1 + b - \beta_t) V_{co} \tag{5-36}$$

It can be obtained from Eq. (5-34):

$$T_c = \frac{(1+b)}{1 + \frac{V_d}{T_d}\frac{T_{co}}{V_{co}}} T_{co} = \beta_t T_{co} \tag{5-37}$$

For b in Eqs. (5-36) and (5-37), the computed interaction curve of $\frac{V_d}{V_0} \sim \frac{T_d}{T_0}$ can fit best with the tested one when $b = 0.5$ is taken among all b values, where V_d and T_d are the design shear force and torsion of shear-torsion members with web reinforcement, respectively. V_0 and T_0 are the shear resistance of flexural members with web reinforcement and the torsional resistance of pure torsion members with web reinforcement, respectively.

According to *Code GBJ 10-89*, the shear resistance of concrete in flexural members with web reinforcement was taken as $V_{co} = 0.07 f_c b h_0$ (f_c is equivalent to f_{cd} in the *Specifications*). The torsional resistance of concrete in pure torsion member with web reinforcement was taken as $T_{co} = 0.35 f_t W_t$ (f_t is equivalent to f_{td} in the *Specifications*), and $f_t \approx 0.1 f_c$, $b = 0.5$ were adopted. All of them are substituted into Eq. (5-35) to obtain

$$\beta_t = \frac{1.5}{1 + 0.5 \frac{V_d}{T_d}\frac{W_t}{b h_0}} \tag{5-38}$$

In prestressed concrete members, the prestress influence on the reduction coefficient β_t for the torsional resistance of concrete may be ignored.

Substituting β_t into Eqs. (5-36) and (5-37), respectively, then

$$V_c = 0.07(1.5 - \beta_t) f_c b h_0 \tag{5-39}$$

$$T_c = 0.35 \beta_t W_t \tag{5-40}$$

These are the calculation equations for shear and torsional resistances of concrete in shear-torsion members with web reinforcement in *Code GBJ 10-89* (shear and torsional resistance of stirrup are taken as the same as those of flexural member and pure torsion member, respectively). Specific study on the resistance of shear-torsion members was not carried out for the *Specifications*, so the calculation methods for torsional resistance are taken from *Code GBJ 10-89*. However, for the shear resistance in shear-torsion members which is computed based on that in flexural members, the equation with product of two terms (shear resistance of concrete and stirrup is considered together, not calculated separately) has been adopted for a long time in the *Specifications*.

Therefore, the equation with a reduction only of the concrete item in *Code GBJ 10-89* cannot be directly introduced and proper adjustment is necessary. Through calculation and comparison of various members, the reduction coefficient of shear resistance in the shear-torsion member is changed from $(1.5-\beta_t)$ to $(10-2\beta_t)/20$, to make the percentage of the reduced value of total shear resistance in the *Specifications* close to the percentage of the reduced value of shear resistance of concrete in *Code GBJ 10-89*.

5.5.5 To calculate the torsion resistance, reinforced concrete torsion members with T-sections, I-sections or box sections with flanges may be divided into several rectangular sections. The principle of division is that a web or rectangular box with the total depth of the section is firstly divided out, and then the compression flanges and tension flanges. The test results of pure torsion members with T-sections or I-sections show that the first inclined crack appear at the middle part of the side surface of the web. When the web width is greater than the flange depth, it can be seen that the cracks on the side of the web and the top of the flange are basically joined if the overhanging flange is removed, thus interconnected spiral inclined cracks are developed off and on. In other words, the web cracks develop independent, affected little by the flange. Therefore, calculations on the torsional resistance of the web and flange may be carried out separately. The divided web or rectangular box (without overhanging flange) is calculated as a shear-torsion member; while compression and tension flange are calculated as a pure torsion member without considering shear. At the same time, the test result also shows that the torsional resistance of the flange with a closed stirrup will be improved with the increase of the overhanging length of the flange. However, if the overhanging part is too large, the overall stiffness reduces when the flange and web are connected, and the flange is prone to fracture after flexural deformation. Therefore, if the overhanging part is too large, the torsion resistance of the flange will decrease significantly. In the *Specifications*, it is stipulated that the overhanging length of the flange shall not exceed 3 times its depth. The design torsion moment resisted by each rectangular basic element is shared from the total torsion of the member according to the ratio of the plastic section modulus of its section to the that of the whole section.

The equation for calculating the plastic section modulus of compression flanges to resist torsion, $W'_{tf} = \frac{h_f^{'2}}{2}(b'_f - b)$, is explained as follows: according to Eq. (5.5.2-1) in the *Specifications*, the plastic section modulus of rectangular section to resist torsion is $W_t = \frac{b^2}{6}(3h - b)$, which can be written as $\alpha b^2 h$, $\alpha = \frac{1}{6}\left(3 - \frac{b}{h}\right)$, α is the coefficient related to the ratio of short side b to long side h of the section.

If the flange section is narrow and long, for full plastic materials, let $b/h = 0$, then $\alpha = \frac{1}{2}$, $W_t = \frac{1}{2}b^2 h$; for flange, $W'_{tf} = \frac{h_f^{'2}}{2}(b'_f - b)$, h'_f is the thickness of the flange and $(b'_f - b)$ is the length of the flange.

5.5.6 In actual bridge engineering, most of the members are subjected to combined flexure, shear and torsion, only a few are pure torsion members or shear-torsion members. According to the provisions of Clause 5.5.5, the reinforcement of these members subjected to combined flexure, shear and torsion may be analyzed and arranged by dividing the member section into several rectangular sections. For example, the required cross-sectional area of flexural longitudinal reinforcement shall be calculated according to the flexural resistance of the flexural member and shall be arranged at the edge of the tension zone. For rectangular sections, the webs in the T-sections and I-sections as well as the rectangular sections of the box with flanges, they shall be analyzed as shear-torsion members; the cross-sectional area of the longitudinal reinforcement shall be obtained according to the required torsional resistance, the reinforcement shall be evenly and symmetrically arranged along the perimeter of the web or perimeter of the rectangular box; stirrup shall be arranged on inclined section, in which the cross-sectional area of the stirrup is sum up from the area calculated according to the required shear resistance and torsional resistance, respectively. For the compression or tension flange of T-section, I-section and box section with flanges, the cross-sectional area of longitudinal reinforcement and stirrup shall be calculated according to the required torsional resistance of pure torsion member, and the longitudinal reinforcement shall be evenly and symmetrically arranged along the perimeter of the flange.

5.6 Members for Punching Shear

5.6.1 This clause is about the calculation provisions of resistance to punching shear of a reinforced concrete slab without punching shear reinforcement. An empirical coefficient of 0.7 is included in Eq. (5.6.1). The β_h is the dimensional effect coefficient of section depth considering the decrease of resistance to punching shear of the slab with the increase of slab thickness. For slabs with prestressing steel in the web, the prestressing can prevent the occurrence and development of inclined cracks, and can increase the depth of the shear-compression zone of concrete, which are beneficial for slab against the punching shears, so this favorable factor is added as the second item in the right-hand side of the equation.

5.6.2 In practical engineering, when the punching shear resistance only provided by concrete cannot meet the requirements or it is difficult to increase the depth of the slab, only increasing the concrete strength class is not a reasonable solution to improve punching shear resistance, arranging punching shear reinforcement is a necessary in design. Test data showed that punching shear reinforcement could increase significantly the punching shear resistance of the slab. The reinforcement shall be arranged near the action plane of the concentrated load; otherwise, the improvement effect is not obvious. In addition, the anchorage of punching shear reinforcement is also very important, poor anchorage will affect the full play of its strength.

Experimental study showed that the failure mode and mechanical behaviors of concrete slabs with punching shear reinforcement were similar to those of flexural members with web reinforcement. When the punching shear reinforcement reaches a certain number (cross-sectional area), the punching shear resistance of the slab almost does not increase with the reinforcement. Therefore, it is necessary to restrict the punching shear reinforcement by specifying the minimum dimensions of the punching shear section of the slab, similar to the requirement for the minimum dimensions of the sections resisting shear in flexural members. Specifications of China and other countries have some provisions about this, that is, the maximum punching shear resistance of the slab with punching shear reinforcement should not be more than 1.5 times that of the slab without punching shear reinforcement. In this way, the function of punching shear reinforcement can generally be full play, and too wide inclined cracks in the service stage can be avoided.

According to research results from other countries, stirrups and bent reinforcement have better effects than other types of punching shear reinforcement. Stirrups or bent reinforcement shall be arranged where inclined cracks may occur. In concrete slabs with punching shear reinforcement, the punching shear resistance of concrete is reduced due to the occurrence and development of inclined cracks. The load corresponding to the occurrence of inclined cracks is about half of the ultimate punching shear load of a concrete slab without punching shear reinforcement.

It is also needed to check the punching shear in other sections without punching shear reinforcement in the concrete slab, *i. e.* outside the section reinforced with punching shear reinforcement, to prevent premature punching shear failure in this zone. Here, the perimeter at $0.5h_0$ outside the punching failure cone is taken as the most unfavorable perimeter in the calculation.

5.7 Members for Local Compression

5.7.1 This clause mainly follows the expression form of the former edition of the *Specifications*. Some parameters are explained as follows:

(1) In calculating the increase coefficient β for compressive strength of concrete under local compression, both A_l and A_b include the hole area. At the same time, for anchor plate with trumpet pipe commonly used in practical engineering, its area A_{ln} is subtracted by the hole area, this consideration comes from engineering design and practical experience.

(2) For high-strength concrete members under local compression, the increased coefficient of strength is lower than that of normal concrete at both the ultimate loading stage and cracking stage. In the *Specifications*, the adjustment coefficient η_s is used to consider this decreasing influence with the increase of concrete strength class (C50 ~ C80).

(3) Design values are taken for both load and material strength. The partial factor for prestressing force in the local compression zone of the anchorage device in post-tensioned members is taken as 1.2.

5.7.2 This clause mainly follows the expression form in the former edition of the *Specifications*. Some parameters are explained as follows:

(1) In calculating the increase coefficient β_{cor} under local compression, A_{cor} and A_l include the area of the hole.

(2) The influence coefficient k introduced in the second item on the right-hand side of Eq. (5.7.2-1) to express that the improve effect of the confinement reinforcement on the local compression strength decreases with the increase of the concrete strength.

6 Serviceability Limit State in Persistent Situations

6.1 General

6.1.1 In the design calculation for the serviceability limit state, the action effect shall be computed under the frequent combination and the quasi-permanent combination of actions. The frequent combination of actions is the combination of the permanent actions with characteristic values, the dominant variable actions with frequent values and the accompanying variable actions with quasi-permanent values, which is similar to the short-term effect combination in the former edition of the *Specifications*. The quasi-permanent combination of actions is the combination of the permanent actions with characteristic values and the dominant variable actions with quasi-permanent values, which is similar to the long-term effect combination in the former edition of the *Specifications*.

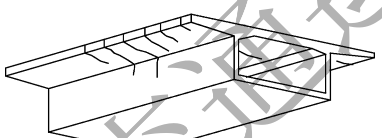
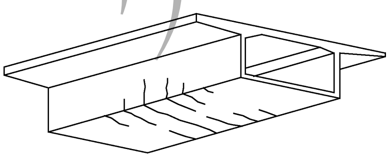
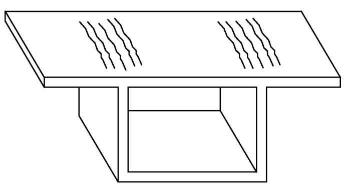
It is generally considered that vehicular load is the dominant live load for highway bridges. Therefore, it is specified clearly in this clause that the impact effect may not be considered in the design calculation for the serviceability limit state.

6.1.2 For fully prestressed concrete members, tensile stress is not permitted to occur at the extreme tension fiber on any cross-section of the member under the frequent combination of actions. Therefore, a high level of prestressing is required on the members. For partially prestressed concrete members, the tensile stress or crack will occur at the extreme tension fiber on the critical section under the frequent combination of actions. Compared with fully prestressed concrete members, the level of prestressing is somewhat decreased in the partially prestressed concrete members, which means that less amount of prestressing steel can be used, this is one of their design purposes. For Type A partially prestressed concrete members, tensile stress at the extreme tension fiber on the critical section is restricted; while for the members whose tensile stress exceeds the limit value and even crack appears, they belong to Type B partially prestressed concrete members.

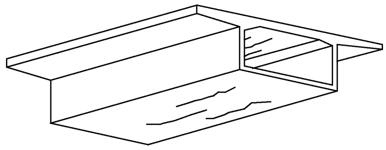
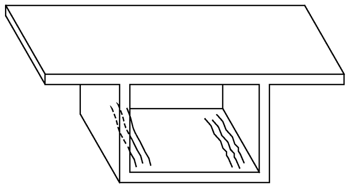
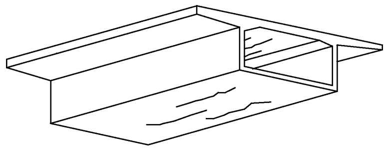
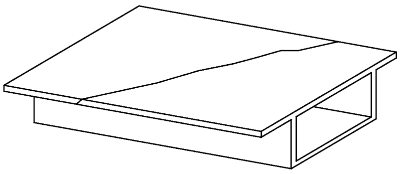
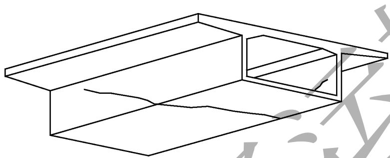
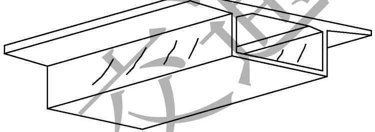
Partial prestress can not only improve the mechanical behavior in the preloading zone of the member, save prestressing steel and even reduce the member depth, but also avoid large inverted camber of the beam, especially for bridges with small spans and subjected to large live load, which can receive obvious benefits by such a design. Even for the partially prestressed members of Type B permitted to crack, the cracks are closed at most times during the service life of the bridge. Only in a short time when the load reaches the maximum design load, the member may crack. In accordance with the provisions of the *Specifications*, mixed reinforcement must be arranged in partially prestressed concrete members. Generally, the reinforcing steel is arranged close to the surfaces of the member while the prestressing steel is arranged farther from the surfaces of the member. As long as the design is reasonable, the prestressing steel will not corrode due to cracks.

6.1.3 Typical cracks in box girder bridges were summarized in the *Guide for Design and Construction of Long Span Prestressed Concrete Highway Bridge* (Zhang Xigang, 2012) published by China Communication Press (in Chinese). The relationship between concrete cracking and stress index were analyzed in the same book, as shown in Table 6-1. If a box girder structure is designed in accordance with Table 6.1.3, the completeness in checking cracking resistance can be ensured in the case that uncontrollable factors, such as material, construction temperature and construction technology are not considered.

Table 6-1 Typical cracks of box girder and the corresponding stress index

Sketch of cracks		Stress index
	Transverse cracks on the top slab at piers, vertical cracks at the upper portion of the web	Longitudinal tensile stress due to negative moment of box girder
	Transvers cracks on the bottom slab at mid-span, vertical cracks at the lower portion of the web	Longitudinal tensile stress due to positive moment of box girder
	Longitudinal cracks on top slab on web	Transverse tensile stress due to local negative moment of slab

continued

<table>
<tr><th colspan="2">Sketch of cracks</th><th>Stress index</th></tr>
<tr><td></td><td>Longitudinal cracks on the top slab at mid-span sections</td><td>Transverse tensile stress due to local positive moment of slab</td></tr>
<tr><td></td><td>Longitudinal cracks on the bottom slab close to the web</td><td>Transverse tensile stress due to excursion force outside of prestressing tendon</td></tr>
<tr><td></td><td>Longitudinal cracks on the bottom slab at mid-span sections</td><td>Transverse tensile stress due to bursting force outside of prestressing tendon</td></tr>
<tr><td></td><td rowspan="2">Spiral cracks through the top slab, web and bottom slab at the girder portion between L/4 and 3L/4 sections</td><td>Principal tensile stress of top slab</td></tr>
<tr><td></td><td>Principal tensile stress of bottom slab</td></tr>
<tr><td></td><td>Diagonal cracks at the girder portion from L/4 section to the end of the girder</td><td>Principal tensile stress of web</td></tr>
</table>

6.1.4 Compared to the former edition of the *Specifications*, the changes of provision on the maximum jacking stress in prestressing steel in this clause are as follows:

(1) In many years of practice, no problems occurred in the design and construction in regard to internal prestressing steel wire and strand by using the maximum jacking stress in the former edition of the *Specifications*. Therefore, the provisions on maximum jacking stress of internal prestressed steel wire and steel strand in the former edition of the *Specifications* remain in the *Specifications*.

(2) The maximum jacking stress in the external prestressing strand is closely related to prestress loss, stress increment due to live load and tensile stress limit in the service stage. Taking 30 m, 50 m, and 75 m beam bridges as samples, taking $0.60 f_{pk}$ as the maximum stress in external prestressing steel in the service stage, the maximum jacking stress in the external prestressing steel is calculated to be $0.68 f_{pk} \sim 0.71 f_{pk}$. In the *Specifications*, $0.70 f_{pk}$ is adopted as the maximum jacking stress in the external prestressing strand.

(3) Considering that reinforcement fracture may occur when a prestressing threaded bar is tensioned, and referring to the current *Code for Design of Concrete Structures* (GB 50010), the maximum jacking stress for the prestressing threaded bar specified in the *Specifications* is appropriately decreased.

It needs to be pointed out that the maximum jacking stress specified in the *Specifications* refers to the stress in reinforcement ahead of anchorages in post-tensioned beams. When anchorages are arranged at the end of beams, the maximum jacking stress in anchorages is the sum of the stress in reinforcement ahead of anchorage and the loss of prestress due to the anchor collar. If the prestress loss in the collar is large, the maximum jacking stress in anchorage should not exceed the maximum value specified in this clause. For example, the tensile stress limits of steel wire and strand should not exceed $80 f_{pk}$. The maximum jacking stress at the anchorage herein is the stress obtained from the total tensile force divided by the cross-sectional area of prestressing steel, in which the tensile force is obtained directly from the oil pressure gauge in the case that the gauge is not interfered by other factors when jacking.

6.1.6 The calculation equation of normal stress of concrete caused by prestressing force is provided in this clause.

(1) Pretensioned members are only used for simply supported structures, in which the pressure lines of prestressing force coincide with the gravity centroid lines of prestressing steel. Hence, the calculation equation for the concrete stress for an eccentrically loaded compression member can be employed to calculate its stress.

(2) For simply supported post-tensioned members with internal tendons, the calculation equations for eccentrically loaded compression members used in pretensioned members can still be employed. If it is a statically indeterminate structure like a continuous beam, the structural deflection due to the prestressing force is restrained by support. Then, secondary reaction force will be produced at support, which further causes a secondary bending moment, and makes the pressure line of prestressing force (the center of concrete stress in each section along the beam span) and the center line of prestressing steel not in the same plane. If the calculation equations for eccentrically loaded compression members are still

employed to calculate the concrete stress, the eccentricity of the pressure center of concrete with respect to the gravity axis of net cross-section shall be adopted, instead of the eccentricity of the center of prestressing steel with respect to the gravity axis of a net cross-section. Therefore, for statically indeterminate structures such as post-tensioned concrete continuous beams with internal tendons, not only the effect of secondary bending moment needs to be considered, but also the normal stress of concrete should be calculated separately.

(3) In post-tensioned members with both internal and external tendons, the normal stress of the concrete due to pre-load force is calculated by referring to calculation equations for post-tensioned members with internal tendons.

6.1.7 The calculation equations for resultant force, and the eccentricity of prestressing steel and reinforcing steel are provided in this clause. In prestressed concrete members with reinforcing steel, due to the influence of shrinkage and creep of concrete, the direction of internal force caused by reinforcing steel is opposite to that of prestressing force, which reduces the pre-stress on concrete in the tension zone and decreases the cracking resistance, this phenomenon needs to be considered in the calculation. In order to simplify the calculation, it is assumed that the stress in reinforcing steel is equal to the losses of prestress due to shrinkage and creep of concrete. In this simplified calculation, some errors will be produced when the centroid of prestressing steel is not coincident with that of reinforcing steel.

6.1.8 In the end anchorage zone of pretensioned members (the end ineffective length in Figure 5-1 is not included), due to the influence of the stress transferred by the anchorage of prestressing steel, the actual stress in the prestressing steel varies nonlinearly, but it can be approximately assumed to vary linearly within the transfer length of prestressing steel. The transfer length l_{tr} (mm) in Table 6.1.8 in this clause is obtained by calculation according to Eq. (6-1):

$$l_{tr} = \beta \frac{\sigma_{pe}}{f_{tk}} \tag{6-1}$$

where:

σ_{pe}——effective stress of prestressing steel during releasing;

β——surface shape factor of prestressing steel. For steel strand of 7 bundles, $\beta = 0.16$; for spiral ribbed steel wires, $\beta = 0.14$;

f_{tk}——the characteristics axial tensile strength of concrete;

d——nominal diameters of prestressing steel strand and steel wire, see Table 3-2 in the background to provisions of Clause 3.2.1 of the *Specifications*. The equivalent diameter $\sqrt{n}d$ is taken when steel strand bundles are used, where, d represents the diameter of a single steel strand or steel wire, n represents their number.

The prestress transfer length, l_{tr}, in Table 6.1.8 is calculated according to the concrete strength class and effective prestress, σ_{pe}, given in the table. In practical engineering, if the concrete strength class is not exact the value listed in the table when the prestressing steel is released, and the transfer length shall be calculated by linear interpolation. And if the effective prestress is not provided, after the calculation by linear interpolation, the transfer length shall be increased or decreased according to the ratio of actual σ_{pe} to the provided σ_{pe}.

6.2 Loss of Prestress

6.2.1 The $\sigma_{l1} \sim \sigma_{l6}$ listed in this clause are the losses of prestress often encountered in the calculation of prestressed concrete members, which have an important impact on the design of the serviceability limit state of a bridge. Each loss is affected by many factors and its calculation is relatively complex. Therefore, it needs to be pointed out that the data of prestress loss determined by test under the specific conditions of the project shall be firstly adopted in the calculation, especially for a large project. The data and calculation method provided in the *Specifications* may only be used when the test cannot be carried out or reliable test data are unavailable.

Other prestress losses are not provided in the *Specifications*, such as those due to friction between prestressing steel and anchorage, and the deformation of the jacking frame for prestressing. When they are required in calculation, they shall be determined by test in advance, or by the data accumulated over the years by fabricators or contractors.

6.2.2 In Eq. (6.2.2), for the spatial curves of the ducts in a parabola or circular arc and for the generalized spatial curves that can be superimposed after segmentation, the sum of included angles θ may be calculated according to the following approximate equation:

(1) Parabola andcircular arc curve

$$\theta = \sqrt{\alpha_v^2 + \alpha_h^2} \tag{6-2}$$

(2) Generalized spatial curve

$$\theta = \Sigma \sqrt{\Delta\alpha_v^2 + \Delta\alpha_h^2} \tag{6-3}$$

where:

α_v, α_h——angular change of parabola and circular arc curve formed by the projection of prestressing steel in vertical and horizontal directions, where prestressing steel varies in a spatial curve of parabola or circular arc curve;

$\Delta\alpha_v$, $\Delta\alpha_h$——increment of angular change of segmental curve formed by the projection of prestressing steel in vertical and horizontal directions, where prestressing steel varies in a generalized spatial curve.

In this revision, the parameters μ and k were appropriately enlarged, after comparing the parameters μ and k in literature in China and other countries, referring to the existing experimental data and the results from bridge construction monitoring, and considering the current construction technology level in China, including the influence of construction of corrugated duct by segmental splicing and positioning, flatness error and local slurry leakage. The friction coefficient of the corrugated plastic duct, μ, is adjusted from 0.14 ~ 0.17 in the former edition of the *Specifications* to 0.15 ~ 0.20 in the *Specifications*.

In this revision, μ and k of the external prestressing strand are supplemented with reference to the data provided by manufacturers.

6.2.3 The calculation for the losses of prestress due to anchorage set and shortening of reinforcement in this clause and Appendix G of the *Specifications* is basically the same as that in the former edition of the *Specifications*. Only the gap of nuts in the anchorage is modified as follows: if the anchorage with nuts is anchored after one-time jacking, 2 ~ 3 mm is taken for Δl; if it is anchored after two times jacking, 1 mm may be taken for Δl.

6.2.4 The equation for calculating σ_{l3} in this clause is the same as that in the former edition of the *Specifications*. It is a common equation in material mechanics, which is established by setting the coefficient of linear expansion $\alpha_c = 1.0 \times 10^{-5}/℃$ and elastic modulus $E_p = 2.0 \times 10^5$ MPa of prestressed steel. The elastic modulus of steel wire and steel strand are given as 2.05×10^5 and 1.95×10^5 in the *Specifications*, their average value is 2.0×10^5. Hence, it conforms to the former edition of the *Specifications*.

During the heat curing of pretensioned members, the prestress loss due to the temperature difference between reinforcement and jacking frame can only occur when steel and concrete have not been bonded; once steel and concrete are bonded and work together, the loss of prestress due to the temperature difference will no longer occur.

6.2.5 Straight prestressing steel is generally used in pretensioned members. All prestressing steel is cut off almost at the same time during release. In calculating the normal pre-compressive stress of concrete at the centroid of reinforcement in Equation (6.2.5-2), σ_{pc}, which is caused by prestress of all reinforcement, effective prestress in prestressing steel, σ_{pe}, is adopted as $\sigma_{con} - \sigma_{l2} - \sigma_{l3} - 0.5\sigma_{l5}$.

Post-tensioned members are often arranged with a lot of longitudinal prestressing steel bars, among which many prestressing steel bars need to be bent up, and they are always tensioned bundle by bundle. In calculating the loss of prestress by Eq. (6.2.5-1), the normal pre-compressive concrete stress $\Delta\sigma_{pc}$ at the centroids of earlier tensioned steel caused by the latter tensioned steel in each bundle should be calculated. It is complex and tedious to calculate by hand unless a computer program is used.

Hence, a simplified method to calculate the prestress loss of post-tensioned members due to elastic shortening of concrete is provided in Appendix H of the *Specifications*. The method is established according to the assumption that the prestressing force in each bundle of the tendon is equal to each other and the average loss due to elastic shortening is adopted, therefore, it is an approximate simplified method. It is assumed that the pre-compressive stress of concrete at the centroid of early tensioned steel caused by each bundle of the latter tensioned steel is $\Delta\sigma_{pc}$, and the corresponding compressive strain of concrete is $\Delta\sigma_{pc}/E_c$, obviously, the early tensioned steel has the strain same as that of concrete, thus the loss of prestress is caused as:

$$\sigma_{l4} = E_p \frac{\Delta\sigma_{pc}}{E_c} = \alpha_{EP}\Delta\sigma_{pc} \tag{6-4}$$

If there are m bundles of prestressing tendons, the loss of prestress due to elastic shortening of the ith bundle tendon will be caused by the $(m\text{-}i)$ bundles of tendons tensioned later. If the m bundles of tendons are of the same type and all the steel bars are assumed to be located at the centroid of all the tendons, then the prestress loss of the ith bundle of tendons is:

$$\sigma_{l4} = (m - i)\alpha_{EP}\Delta\sigma_{pc} \tag{6-5}$$

where $\Delta\sigma_{pc}$ is the normal compressive stress of concrete at the centroid of all tendons, which is caused by jacking a bundle of tendons.

It is obvious that prestress losses due to elastic shortening of m bundles of tendons are different. The prestress loss of the first bundle of tensioned tendons is the largest, $\sigma_{l4(1)} = (m-1)\alpha_{EP}\Delta\sigma_{pc}$, and the prestress loss of the last bundle of tensioned tendons is zero, $\sigma_{l4(m)} = (m-m)\alpha_{EP}\Delta\sigma_{pc} = 0$. In the simplified calculation, the average prestress loss of m bundles tendons (m tendons) induced by elastic shortening is adopted as follows:

$$\begin{aligned}\sigma_{l4} &= [\sigma_{l4(1)} + \sigma_{l4(2)} + \cdots + \sigma_{l4(m)}]/m = \sum_{i=1}^{m}\sigma_{l4(i)}/m \\ &= \sum_{i=1}^{m}(m-i)\alpha_{EP}\Delta\sigma_{pc}/m = \alpha_{EP}\Delta\sigma_{pc}\sum_{i=1}^{m}(m-i)/m \\ &= \frac{m}{2}\alpha_{EP}\Delta\sigma_{pc}\end{aligned} \tag{6-6}$$

The equation above is provided in Appendix H of the *Specifications* to calculate the prestress loss due to elastic shortening of tendons.

Here, two points are supplemented to explain how to calculate the normal pre-compressive stress of concrete at the centroid of tendons:

(1) In determining $\Delta\sigma_{pc}$ or σ_{pc}, the effective prestress of prestressing steel, σ_{pe}, is taken as σ_{con}- σ_{l1}- σ_{l2};

(2) After the prestressing steel is curved, $\Delta\sigma_{pc}$ is different at different cross-sections along the longitudinal direction of the beam, which can be taken by two approaches in calculation. In the *Specifications*, the stress at the cross-section where it is required to be controlled is taken, because data of geometric characteristics and prestress loss at the cross-section is ready-made. In Chinese railway specifications, $\Delta\sigma_{pc}$ at the $L/4$ cross-section is specified for simply supported beams, and average values of $\Delta\sigma_{pc}$ at several representative cross-sections are used for continuous beams.

6.2.6 The former Shanghai Institute of Railway Technology in China carried out an intensive study on the loss of prestress due to the relaxation of prestressing steel. After seven years of tests and analyses, the following equation for calculating the ultimate loss of prestress due to the relaxation of steel wire and steel strand was proposed:

$$\sigma_{l5} = \psi\left(0.52\frac{\sigma_{\text{pe}}}{f_{\text{pk}}} - 0.26\right)\sigma_{\text{pe}} \tag{6-7}$$

The equation is only applicable to steel wires and strands with general relaxation. According to the calculation of prestressed concrete voided slabs with span of 10 ~ 20 m and prestressed concrete simply supported beams with span of 25 ~ 50 m, when one-time jacking is used, $\sigma_{\text{pe}}/f_{\text{pk}} = 0.63 \sim 0.68$, and the ultimate loss of prestress $\sigma_{l5} = (0.07 \sim 0.093)\sigma_{\text{pe}}$. However, the relaxation rate of steel wire and strand at 1000 h with common relaxation in current national standard is not more than 8% when initial stress is $0.7f_{\text{pk}}$. By comparison, the Eq. (6-7) is acceptable, in which when $\sigma_{\text{pe}}/f_{\text{pk}} = 0.5$, $\sigma_{l5} = 0$.

Nowadays, a large number of steel wires and strands with low relaxation are used in practical projects. In order to be applicable to the actual situations, Eq. (6-7) is multiplied by the coefficient of reinforcement relaxation ζ in the *Specifications*, that is:

$$\sigma_{l5} = \psi\xi\left(0.52\frac{\sigma_{\text{pe}}}{f_{\text{pk}}} - 0.26\right)\sigma_{\text{pe}} \tag{6-8}$$

According to the introduction of *Modern Prestressed Concrete Floor Structures* (XU Jinsheng et al.) published by China Architecture & Building Press (in Chinese), the stress relaxation of steel wires and steel strands with low relaxation is about 1/4 of that in the prestressing steel with common relaxation. In the current national standard for steel wire and steel strand for prestressed concrete, it is stipulated that the stress relaxation of steel wire and steel strand with low relaxation is 0.31 of that in the prestressing steel with common relaxation. For the sake of safety, $\zeta = 0.3$ is adopted in the *Specifications*.

According to the previous design experience, the loss of prestress due to relaxation for prestressing threaded bars is taken as the specified value for cold-drawn steel bars in the *Specifications JTJ 023-85*.

When it is necessary to calculate the loss of prestress due to stage relaxation of prestressing steel, the ratio of intermediate value and ultimate loss of prestress due to relaxation of prestressing steel is provided in Appendix C of the *Specifications*. The ratio is referred from the relevant information of the China Ministry of Railways and is only applicable to steel wire and strand.

6.2.7 Eqs. (6.2.7-1) and (6.2.7-2) in this clause are applicable to calculate the loss of prestress for members with longitudinal prestressing steel in the tension and compression zones, respectively. If the member is concreted by pumping, the loss of prestress due to shrinkage and creep should be enlarged according to the actual situation.

6.3 Check for Cracking

6.3.1 For a long time, cracking resistance for prestressed concrete members of highway bridges is checked by investigating if the tensile stress of the concrete in the member exceeds or not the specified limit value, including cracking resistance on cross-section and inclined section of the members.

(1) Checking for crack resistance on cross-section

For Type A prestressed concrete members, $\sigma_{\mathrm{lt}} - \sigma_{\mathrm{pc}} \leqslant 0$ in the *Specifications* is used to check the resistance against cracking under long-term load, where σ_{lt} is the normal tensile stress of concrete at the extreme fiber of the member under the quasi-permanent combination of actions. The quasi-permanent combination of actions only includes the self-weight of the structure and the live load directly applied, other indirect actions are not taken into account.

For Type B members, although they are permitted to crack under the frequent combination of actions, their resistance against cracking on cross-section shall still be checked, because it is still expected that no tensile stress under the self-weight of structure will appear at the extreme tensile fibers of their critical sections.

(2) Checking forcrack resistance on inclined section

The inclined cracks in the web of prestressed concrete beam cannot be closed by themselves, unlike the cracks on cross-section which can be closed in most cases at the service stage. Therefore, resistance against cracking on the inclined section of members shall be checked strictly, and designers shall pay more attention to it. No matter what kind of flexural members, the inclined cracks are not expected to occur. Therefore, it is specified in the *Specifications* that checking for resistance against cracking on the inclined section of all flexural members is required to be carried out. Due to the existence of prestress in the beam, especially in the post-tensioned members, the prestressing steel could be reasonably arranged. For long-span bridges, vertical prestressing steel is often arranged in the beam segment with large principal tensile stress, which can greatly counteract the principal tensile stress caused by actions, hence, it is also easy to satisfy the requirements of resistance against cracking on the inclined section.

However, it should also be pointed out that it is found from the statistics of test results that the tensile strength of concrete dispersed largely. The characteristic tensile strength of concrete with 95 percent level of confidence is adopted in the *Specifications* (it is adopted with 85 percent level of confidence in the previous *Specifications JTJ 023-85*), if the quality of concrete is not well controlled in construction or the design is not reasonable, the probability will be greatly increased that the principal tensile stress in the actual bridges exceeding the limit value specified in the *Specifications*. The limits of principal tensile stress of concrete in codes of China and other countries are close. For example, $\sigma_{\mathrm{tp}} < 0.85 f_{\mathrm{tk}}$ is required in *Code GBJ 10-89* for members strictly required not to crack; while $\sigma_{\mathrm{tp}} < 0.95 f_{\mathrm{tk}}$ for members generally required not to crack; $\sigma_{\mathrm{tp}} < f_{\mathrm{tk}}$ is specified in railway specifications, while $\sigma_{\mathrm{tp}} < 0.8 f_{\mathrm{tk}}$ is specified in the previous *Specifications JTJ 023-85*. The specified values in the highway specifications are lower than those in the other two Chinese specifications. According to the above provisions, no adverse results have been reported in railway bridges or building structures for many years, and also in the highway bridges generally. However, in recent years, it was reported from time to time that inclined cracks occurred on the highway long-span continuous beam bridges, and it even occurred in the construction stage for some bridges. The inclined cracks occurred in a regular case, indicating that the principal tensile stress of the actual bridge exceeded the ultimate tensile stress of concrete. Although these inclined cracks are mostly stable and will not go so far as to cause bridge failures, but they damage the appearance of the bridge and may cause people's psychological anxiety. Therefore, according to the investigation and analysis data of those bridges, some supplementary provisions are specified or stricter provisions are put forward in the *Specifications* to eliminate or reduce the causes of defects.

The second item of this clause specifies the limit of the principal tensile stress on the inclined section, where σ_{tp} is the principal tensile stress of concrete in the members under frequent combination of actions, and is calculated in accordance with equations in Clause 6.3.3 of the *Specifications*. For prestressed concrete continuous beams, in addition to the loads directly applied to the bridge, such as dead load and vehicular load, the influence of indirect effects such as temperature difference due to sunlight, shrinkage and creep of concrete shall also be taken into account in the calculation of σ_{tp}. However, not all unfavorable factors are considered in the design calculation; therefore, the actual stress of the bridge is often larger than the calculated value. In recent years, a large number of simply supported beams have been built, no matter whether they are pretensioning or post-tensioning, regular inclined cracks were rarely reported, which indicates that this kind of bridge can resist the principal tensile stress. Therefore, two cases in determining the limit of principal tensile stress are specified in the *Specifications*. Most of the cast-in-place bridges are long span continuous beam bridges, based on the design experience of some bridges built in recent years without inclined cracks, the principal tensile stress of concrete in fully prestressed concrete bridges is limited to about 1.0 MPa, that is, $\sigma_{tp} \leqslant 0.4 f_{tk}$, and the requirement for principal tensile stress of prestressed concrete members of Type A and B are appropriately relaxed, to $\sigma_{tp} \leqslant 0.5 f_{tk}$. Most of the precast bridges have small spans, according to various opinions, the limit of the principal tensile stress in these bridges is also reduced to a certain extent compared with the previous *Specifications JTJ 023-85*.

6.3.3 The provisions in this clause are basically the same as those in former edition of the *Specifications*. In recent years, many regular inclined cracks have been found in the newly-built long span prestressed continuous box girder bridges. These bridges were designed with vertical prestressing steel, which could play an important role in resisting the principal tensile stress in design. However, analysis results showed that if these vertical prestressing steel were not fully used, the principal tensile stress of the web would exceed the limit value specified in the *Specifications*, and inclined cracks might occur. The investigation showed that the construction quality of vertical prestressing steel was generally not so good, and even all the prestressing was almost lost in some cases, so the induced vertical compressive stress in concrete could not reach the calculated value. In view of the current reality and the difficult to prestress the vertical prestressing steel in construction, the calculation method in the former edition of the *Specifications* to multiply a reduction factor of 0.6 in the calculation equation of σ_{cy} is still used in the *Specifications*. Meanwhile, it is stipulated in Clause 9.4.1 of the

Specifications that the longitudinal spacing of vertical prestressing steel should be 500 ~ 1000 mm. It is also found in the investigation that inclined cracks also occurred in the end span cast-in-place segment of continuous beam bridges and continuous beam bridges with fixed superstructure-pier connection, in which most of them are arranged with straight prestressing steel for the convenience of construction in the past design. Therefore, the *Specifications* stipulates in Clause 9.4.7 that curved prestressing steel should be designed in these beam segments to play a role in reducing the principal tensile stress.

The second item on the right-hand side of Eq. (6.3.3-2) is the normal stress of concrete at the location of principal tensile stress caused by frequent combination of actions, where M_s represents the effect induced by the frequent combination of actions. However, the effects of some actions are not only the bending moment M_s, but also the axial force N, both of them affect the normal stress of the principal stress point. Therefore, the normal stress at the principal stress point shall be calculated separately for these actions, and then superimposed with the normal stress caused by other actions.

According to the former edition of the *Specifications*, only the prestressing force effect of vertical prestressing steel is taken into account when vertical stress is calculated. In this revision, refereeing to the *Technical Guide for Design and Construction of Prestressed Concrete Beam Bridge* (BAO Weigang, 2009) published by China Communication Press (in Chinese), the prestressing force of transverse prestressing steel, transverse temperature gradient, and vehicular load are also taken into account, all these are calculated according to frequent combination of actions.

6.4 Check for Crack Width

6.4.2 The limit of crack width in this clause refers to the vertical crack of members under the frequent combination of actions together with the consideration of the influence of long-term effect, excluding other non-mechanical cracks caused by excessive shrinkage of concrete, improper curing, and excessive penetration of chloride salt during construction. The following factors are considered for the limit of the crack width: good structural durability, no reinforcement corrosion, and not excessive crack width which may induce influence on structural appearance and people's psychological uneasiness. However, practical measures are much more important in controlling crack width than design calculations, such as the compactness of concrete ensured during construction, and the necessary thickness of cover adopted in design.

Type B prestressed concrete members shall be used selectively. They shall not be used for bridges in areas that are seriously affected by aggressive environment.

6.4.3 Eq. (6.4.3) in this clause is developed from the calculation equation in the former edition of the *Specifications*, after analyzing the factors affecting the crack width of concrete members and considering the international relative consensus. In the developed equation, the reinforcement ratio is changed from the longitudinal tension reinforcement ratio within the member section in the former edition of the *Specifications*, ρ, to the effective reinforcement ratio for the longitudinal tension *reinforcement*, ρ_{te}.

For the calculation of the crack width of reinforced concrete members with circular cross-sections, the equation different from rectangular sections, T-sections and I-sections was adopted in the former edition of the *Specifications*. Considering that the formula for crack width in the *Specifications* is an empirical formula obtained from tests, Eq. (6.4.3) is also adopted to calculate the crack width of members with a circular section in the revision of the *Specifications* for consistency, in which the calculation formulae for longitudinal tension reinforcement and effective reinforcement ratio are different, see Clause 6.4.5.

Eq. (6.4.3) is also adopted to check the cracks of prestressed concrete flexural members in order to consist it with that for reinforced concrete members. However, E_s is replaced by E_p in the equation, and the stress in prestressing steel, σ_{ss}, is adopted as the stress increment of reinforcement after decompression.

6.4.4 The calculation equation of stress in longitudinal tension reinforcement at the cracked section of the member, σ_{ss}, is provided in this clause.

(1) Stress of reinforced concrete members with rectangular sections, T-sections and I-sections

The calculation equation of stress in reinforcement for reinforced concrete flexural members in the previous *Specifications JTJ 023-85* is still adopted in the *Specifications*, and the internal lever arm of the cracked section is adopted as $0.87h_0$, which has been proved to be effective by practical application for many years. The provisions in the former edition of the *Specifications* are adopted for the calculation equation of stress in the reinforcement of other reinforced concrete members and prestressed concrete flexural members.

The calculation equation of reinforcement stress of eccentric tension members is established by taking the moment of axial tensile force N_s and internal force on the section to the point of the resultant force in reinforcement in the compression zone. The equation is used for both tension members with large and small eccentricity, and the internal lever arm of the cracked section are all taken as $Z = h_0 - a_s'$, which is an approximate one.

The calculation equation of reinforcement stress of eccentric compression members is established by taking the moment of axial compressive force N_s and internal force on the section to the point of the resultant force of the compression zone. For eccentrically loaded reinforced concrete compression members, when $l_0/h < 14$, the test results show that the influence of member deflection on the eccentricity of axial force shall be taken into account, and Eq. (5.3.9-1) in the *Specifications* may be approximately used. However, when members have cracks, the influence coefficients ζ_1 and ζ_2 of the load eccentricity and slenderness ratio of the member in the equation may not be taken into account, thus ξ_1 and ξ_2 are adopted as 1.0. The curvature at the ultimate limit state cannot be taken as the curvature of the critical section, and 1/2.85 is taken according to *Code GBJ 10-89*.

(2) Eccentrically loaded reinforced concrete compression member with the circular section

The equation in the former edition of the *Specifications* for calculating the maximum stress in the reinforcement of reinforced concrete members with the circular section subjected to compressive force with large eccentricity was provided according to the test results. It was found that the concrete strength class had a great influence on the calculation results. In fact, the elastic modulus of concrete varies little with its strength class; hence, the stress in longitudinal reinforcement shall not vary significantly with the concrete strength class. In Clause 6.4.4 of the *Specifications*, the simplified calculation equation for reinforcement stress of reinforced concrete members with rectangular sections, T-sections and I-sections is independent of the concrete strength class. The assumption of the plane sections remain plane after deformation was adopted in the analysis of cross-section of reinforced concrete members, equations for calculating the maximum stress of reinforcement of reinforced concrete members with circular section subjected to compressive force with large eccentricity was studied and proposed by the editing team. However, the equations are two nonlinear transcendental equations, which are difficult to be applied in engineering. After analyzing the analytical solution for the equations, the simplified equations were proposed, and their parameters were corrected by the tested crack widths of circular members, then the equations for calculating the stress of reinforcement were finally presented in the *Specifications*.

(3) Prestressed concrete flexural members

The equation for calculating the stress in tension reinforcement of prestressed concrete flexural members is similar to that of eccentrically loaded reinforced concrete compression members. It is also established by balancing the bending moments of internal and external forces with respect to the point of the resultant force in the compression zone, but the calculated result is stress increment in the reinforcement. Therefore, the external force M_s needs to be subtracted by the moment $N_{p0}(Z - e_p)$ formed by the resultant force N_{p0} at the point of the resultant force in prestressing steel and reinforcing steel when the normal prestress of concrete equals zero. In addition, the secondary bending moment M_{p2} caused by the prestressing force shall be taken into account for statically indeterminate structures like continuous beams. The same mode is adopted in the *Specifications* for the internal lever arm Z of the cracked section for eccentrically loaded reinforced concrete compression members and prestressed concrete flexural members.

6.4.5 In calculating the effective reinforcement ratio for reinforced concrete members with rectangular sections, T-sections and I-sections, the depth of concrete area in tension, a_s, is determined as twice the distance from the centroid of longitudinal tension reinforcement to the extreme tension fiber of the member. However, for eccentrically loaded compression members with circular section, the tensile stress in longitudinal reinforcement is different, as shown in Figure 6-1. The larger the distance from the neutral axis is, the greater the tensile stress of reinforcement is, and the stronger the confinement effect on cracks is, while the section on the other side of the neutral axis is in compression. According to the principle that the confinement effect of reinforcement evenly distributed along a perimeter on cracks is proportional to the distance from the neutral axis, the effective area of reinforcing steel for confining cracks is obtained by theoretical analysis and then divided by the shadow area in Figure 6-1 to obtain Eq. (6.4.5-2) for calculating effective reinforcement ratio for reinforced concrete members with circular section. The coefficient β in the equation reflects the contribution of longitudinal tension reinforcement to restrain crack, which is related to the half pressure angle ϕ in Figure 6-1, while ϕ is related to the stress state of the member. After numerical analysis, a simplified equation for calculating β is given.

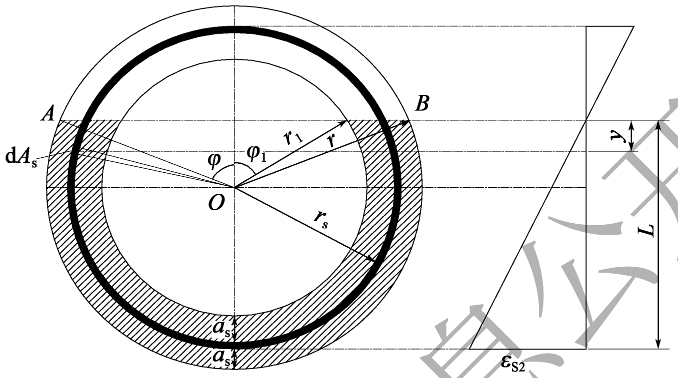


Figure 6-1　Effective tensile area of circular member

6.5　Check for Deflection

6.5.2　When the deflection of reinforced concrete flexural members is calculated in the *Specifications*, Eq. (6.5.2-1) is adopted to calculate the stiffness of the equivalent section, which was developed by the relevant research data from Southeast University in China. According to the statistics analysis of 198 reinforced concrete flexural members, the average ratio of the test results to the calculated results is $\mu = 1.106$, with a standard deviation $\sigma = 0.153$ and variation coefficient $\delta = 0.138$.

A flexural member with cracks is regarded as a member with variable stiffness [Figure 6-2a)], where the stiffness at the cracked section is the smallest, and the stiffness of the sections between two cracks is the largest. The solid line in Figure 6-2b) shows the variation law of the sectional stiffness. In order to facilitate the analysis, a cracked segment with a length of l_{tr} is taken and approximately decomposed into a segment of integral section $\alpha_1 l_{tr}$ and a segment of cracked section $\alpha_2 l_{tr}$ [Figure 6-2c)]. According to experimental study, α_1 and α_2 are related to the ratio of cracking moment M_{cr} to bending moment M_s, which may be determined by the following equations:

$$\alpha_1 = \left(\frac{M_{cr}}{M_s}\right)^2 \tag{6-9}$$

$$\alpha_2 = 1 - \left(\frac{M_{cr}}{M_s}\right)^2 \tag{6-10}$$

The member with variable stiffness in Figure 6-2c) is equivalent to the member with constant stiffness in Figure 6-2d). The equivalent stiffness B of flexural members with constant stiffness may be obtained by using the structural mechanics' method according to the principle of an equal rotation angle of the member under the action of end bending moment.

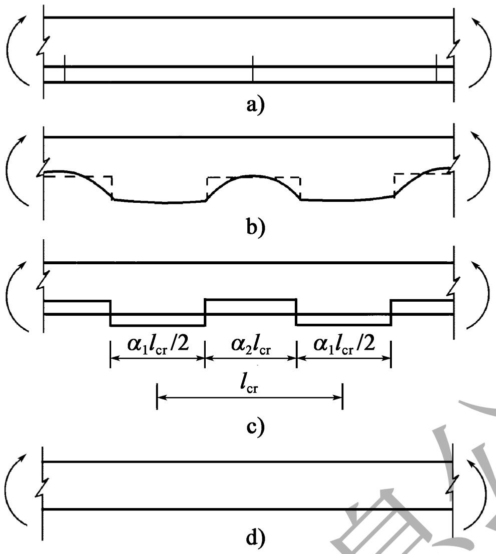


Figure 6-2 Diagram of the equivalent section of the member

According to the member with variable cross-sections shown in Figure 6-2c), the relative rotation angle θ_1 of the section at both ends of the cracked segment is calculated:

$$\theta_1 = \frac{\alpha_1 l_{cr} M_s}{B_0} + \frac{\alpha_2 l_{cr} M_s}{B_{cr}} \tag{6-11}$$

According to the member with constant cross-sections shown in Figure 6-2d), the relative rotation angle θ_2 of the section at both ends of the cracked segment is calculated:

$$\theta_2 = l_{cr} \frac{M_s}{B} \tag{6-12}$$

Let $\theta_1 = \theta_2$, then:

$$\frac{1}{B} = \frac{\alpha_1}{B_0} + \frac{\alpha_2}{B_{cr}} \tag{6-13}$$

Substituting Eqs. (6-9) and (6-10) into Eq. (6-13), it can be rewritten as:

$$B = \frac{B_0}{\left(\frac{M_{cr}}{M_s}\right)^2 + \left[1 - \left(\frac{M_{cr}}{M_s}\right)^2\right]\frac{B_0}{B_{cr}}} \tag{6-14}$$

where the cracking moment $M_{cr} = \gamma f_{tk} W_0$.

For prestressed concrete flexural members permitted to crack, according to the research data from Southeast University in China, the flexural stiffness of the equivalent section may also be obtained by the following method, which can be unified with those of reinforced concrete members: substituting $M_{cr} = M_0 + M_{cr,r}$ into Eq. (6-14) (M_0 is the bending moment for decompression, $M_{cr,r}$ is the cracking moment of non-prestressed concrete flexural members corresponding to prestressed concrete flexural members with cracking permitted), then appropriately correcting the results according to the test data. However, according to the trial calculation of the existing flexural members of highway bridges, the calculated deflection using the equivalent stiffness is much larger than that calculated according to the previous *Specifications JTJ 023-85*. Therefore, the calculation method of the previous *Specifications JTJ 023-85* is still used in the *Specifications*, and only the member stiffness is adjusted according to the field test results of several prestressed concrete highway bridges.

6.5.3 In the *Specifications*, the long-term effect of load is taken into account when deflection of flexural members is calculated, and it is expressed by the elastic deflection multiplied by the factor for long-term deflection. That is, with the increase of time, the stiffness of the member is decreased and the deflection is increased. The causes are as follows: the creep of concrete occurs in the compression zone; the bonding between the concrete and the reinforcement in the tension zone is gradually out of work, and the average strain of reinforcement increases; the shrinkage of concrete in the compression zone is inconsistent with that in the tension zone, and the curvature of the member increases; the elastic modulus of concrete decreases, etc.

The factor for long-term deflection, η_θ, in this clause, is adopted from the following equation in *Design Proposal for Partially Prestressed Concrete Structures* edited by China Civil Engineering Society (1985):

$$\eta_\theta = \frac{M_l\theta + (M_s - M_l)}{M_s} \tag{6-15}$$

For the convenience of calculation, it is simplified as stated in follows:

The ratio of M_l to M_s (M_l/M_s) in the equation is about 0.56 on average under the ratio of dead load to frequent live load in highway bridges, that is, the average value $M_l = 0.56\,M_s$.

In the equation, θ is the influence coefficient of the development factor of deflection under long-term load. For reinforced concrete flexural members, θ is related to longitudinal reinforcement ratio ρ' in the compression zone, and $\theta = 2.0$ when $\rho' = 0$. For reinforced concrete flexural members of highway bridges, they are usually designed without or with a small amount of longitudinal primary reinforcement in the compression zone, thus, $\theta = 2.0$ may be approximately taken. Generally, $\theta = 2.0$ is also taken for the prestressed concrete flexural members, which is large but safe for Type B members which are permitted to crack. The creep in high-strength concrete of C50 and above is small, and the increase of deflection due to creep is also small. However, the reducing effect of reinforcement in the compression zone to long-term deflection is poor. According to the *Guide for Design and Construction of High Strength Concrete Structures*, $\theta = 1.85 \sim 1.65$ is taken when $\rho' = 0$.

By substituting $M_l = 0.56M_s$ and θ into Eq. (6-15) and after adjusting, the factor for long-term deflection of reinforced concrete and prestressed concrete flexural members, η_θ, may be obtained. The limit of deflection specified in the *Specifications* still follows the provisions in the previous *Specifications JTJ 023-85*. However, the calculated deflection is increased after considering the long-term effect of load, especially in prestressed concrete members. Therefore, in the calculation of η_θ for high-strength concrete members, the lowest strength of concrete in the calculation was changed from C50 in the previous *Specifications* to C40 in the *Specifications*. When the concrete strength class is C40, $\eta_\theta = 1.45$; when it is C80, $\eta_\theta = 1.35$; while when it is between C40 ~ C80, η_θ is obtained by linear interpolation.

6.5.5 The beam deflection shall be checked not exceeding the limit value specified in the *Specifications*, which aims to evaluate whether the beam has sufficient stiffness. The pre-camber is set for the beam to have a smooth driving condition after the bridge is completed. Therefore, the beam pre-camber also needs to be considered by designers.

The principle of pre-camber setting for reinforced concrete bridges is as follows: when the deflection caused by structural self-weight and static live loads exceeds $l/1600$, the pre-camber shall be set, and its value is the sum of deflection caused by the structural self-weight and half of the deflection caused by live loads. Hence, after the bridge is completed, the mid-span of the beam can have a camber with a value of half the deflection caused by live loads, and the side view of the bridge is also visually appealing. The long-term effect of the load is taken into account in the pre-camber setting in the *Specifications*.

It is generally regarded that the camber of a prestressed concrete beam is always upward; hence, it is unnecessary to set a pre-camber for it. It may be right for a fully prestressed concrete beam, because its prestress degree $\lambda = M_0/M_s \geqslant 1$, and the bending moment for decompression, M_0, always greatly exceeds the bending moment caused by structural self-weight. However, for partially prestressed concrete beams, especially prestressed concrete beams permitted to crack, the camber of the beams will be greatly reduced. If the ratio of dead load to live load is large, the beam may gradually bend downward. Therefore, pre-camber shall be set for prestressed concrete beam when it is necessary. Two situations are considered in the *Specifications*:

(1) When the long-term deflection caused by prestress is greater than the calculated long-term deflection under frequent combination of actions, the camber of the beam is already very large. After the long-term deflection of structural self-weight is eliminated, the camber still remains in the beam with a value larger than the long-term deflection caused by the frequent live loads. In this case, the pre-camber is not necessary to be set, and the adverse effect of excessive pre-camber due to pre-applied stress is necessary to be considered. If the ratio of dead load to live load is small, such adverse effects will be more likely to occur. Therefore, this situation needs to be fully predicted during the bridge design stage, and appropriate measures are to be adopted, such as reducing the prestress degree or inverting the camber.

(2) When the long-term camber caused by pre-applied stress is less than the calculated long-term deflection under the frequent combination of actions, the beam camber is very small after it is eliminated by the long-term deflection under structural self-weight, which is generally small compared with the span length of the bridge, pre-camber needs to be set. The difference between the long-term deflection under the frequent combination of actions

and the long-term camber caused by pre-applied stress is adopted as the pre-camber, *i. e.*, to keep the camber with a value the same as the long-term deflection induced by the frequent live load.

6.5.6 Generally, the member deflection of a highway bridge during the construction stage is not calculated. However, the construction period of some long-span bridges is relatively long. When the cantilever method with cast-in-place or precast segments is adopted, the deflection of the cantilever end needs to be calculated for control. Meanwhile, shrinkage and creep strains of concrete have not reached their ultimate values, and need to be calculated according to the loading age and the age when the deflection is calculated.

7 Stress Analysis for Members in Persistent and Transient Situations

7.1 Stress Analysis for Prestressed Concrete Members in Persistent Situations

7.1.1 Since the stress state of cross-sections in prestressed concrete members after prestressing is complex, according to the design tradition for highway bridges, besides the load-carrying capacity of the members, the stresses in the members at the elastic stage also need to be checked. These stresses include normal compressive stress of concrete on a cross-section, tensile stress of reinforcement and principal compressive stress of concrete on an inclined section. Stress analysis for members is essentially strength analysis for members and is a supplement to the load-carrying capacity analysis for the members. In the analysis, characteristic actions are taken, impact effect shall be included for vehicular loads, the effect of prestress shall be considered, and partial factors for all actions are taken as 1.0. For simply-supported prestressed concrete structures, only the principal effect due to the prestress is analyzed. Besides that, secondary effects due to prestress, action of temperature and other variable actions shall be analyzed for continuous prestressed concrete beams and other statically indeterminate structures.

7.1.4 In the analysis of stresses on cracked sections in prestressed concrete flexural members, the flexural members subjected to external bending moment M_k and the resultant force N_{p0} in prestressing steel and non-prestressing steel may be transformed into eccentrically loaded compression members only subjected to an axial force N_{p0} with a distance e_{0N} away from the gravity axis of the section. For continuous post-tensioned concrete beams and other statically indeterminate structures, the above external bending moment M_k shall include the moments due to all actions and the secondary moment M_{p2} induced by the prestress from Figure 7.1.4:

$$N_{p0}(h_{ps}+e_N) = M_k \pm M_{p2} \tag{7-1}$$

$$e_N = \frac{M_k \pm M_{p2}}{N_{p0}} - h_{ps} \tag{7-2}$$

where:

h_{ps}——distance from the resultant force in prestressing steel and non-prestressing steel to the extreme compression fiber of the section;

e_N——distance from the eccentrically compressive force N_{p0} to the extreme compression fiber.

Eqs. (7.1.4-1) and (7.1.4-5) in this clause are general formulas in mechanics of materials.

For non-prestressing steel arranged in the tension zone, their tensile stresses are not high after the precompression is overcome. Hence, its calculation formula is not given in this clause.

7.1.5 Superposition of stresses in prestressed concrete flexural members calculated in accordance with Clauses 7.1.2, 7.1.3 and 7.1.4 as well as the stress limits after superposition are given in this clause.

The maximum compressive stress of concrete in the compression zone of members at the service stage shall be the algebraic sum of compressive stress σ_{kc} due to the combination of characteristic actions and tensile stress σ_{pt} of concrete in pretension zone due to prestress for uncracked members; and it shall be the compressive stress σ_{cc} of concrete on cracked sections due to the combination of characteristic actions for cracked members.

The maximum tensile stress of prestressing steel in tension zone of the member shall be the sum of effective prestress σ_{pe} in prestressing steel after subtracting all the losses of prestress and stress σ_p in reinforcement due to the combination of characteristic actions for uncracked members; and it shall be the sum of the effective prestress in prestressing steel when the normal stress σ_{p0} of concrete at the point of the resultant force in prestressing steel is zero and the stress increment σ_p in prestressing steel on cracked section due to the combination of characteristic actions for cracked members. For external prestressing strands, the effective prestress $\sigma_{pe,ex}$ in prestressing steel after subtracting all the losses of prestress shall be taken.

The compressive stress limit of concrete is 0.5 times the characteristic compressive strength of concrete. The tensile stress limit in steel wires and strands is 0.65 times their characteristic tensile strength. The above stress limits are comparable to those in *Specifications JTJ 023-85*.

In this revision, the stress limit for external prestressing strands has been supplemented. Fatigue performance of the external prestressing system is a primary factor in determining the stress limit for external prestressing strands at the service stage. Not only the fatigue stress range of external prestressing strands under vehicular loads but also the uncertainties of local stresses in anchorages and deviators need to be considered. Table 7-1 lists different stress limits in the specifications of different countries. By referring to the tensile stress of internal prestressing steel strands and considering the fatigue performance of external prestressing strand systems, the limit values have been specified as $0.60f_{pk}$; the tensile stress limit of prestressing threaded bars has been moderately reduced from $0.80f_{pk}$ in the former edition of the *Specifications* to $0.75f_{pk}$ in this revision.

Table 7-1　Tensile stress limits of external prestressing strands in persistent situations in some specifications in the world

Specification	USA	Japan	Germany	France
Tensile stress limit	$0.72f_{pk}$	$0.70f_{pk}$	$0.70f_{pk}$	$0.60f_{pk}$

7.1.6　The following two provisions are given in this clause:

(1) Calculation and limit value for the principal compressive stress on an inclined section of prestressed concrete flexural members are specified. This provision is to prevent the compression failure of webs under prestess in jacking and under actions at the service stage, and is the supplement for the check of shear resistance on inclined sections. Too high principal compressive stress may also lower the cracking resistance on inclined sections.

(2) Calculation of principal tensile stress in webs of the member at the elastic stage, requirements for stirrup arrangement and calculation for the number of stirrups are specified. These are the supplements for the analysis of shear resistance on inclined sections of the member. The quantity of stirrups calculated by this clause shall be compared with that calculated by shear resistance on inclined sections, and the larger one is adopted in design.

7.2　Stress Analysis for Members in Transient Situations

7.2.1　The stress analysis of members in transient situations required in this section, is essentially the strength analysis of members at the elastic stage. Characteristic construction loads are used for the action combination unless otherwise specified. Generally, serviceability limit state analysis is not performed for transient situations. Generally, construction measures or detailing arrangements can be employed to prevent excessive deformation or unwanted cracks occurring in members.

7.2.3　During the prestressing of members, concrete is required to achieve a certain strength and modulus of elasticity.

(1) At the initial stage of concrete shrinkage and creep, early prestressing will cause a large loss of prestress. On one hand, the prestress cannot be fully used, and on the other hand, micro-cracks may be produced in members and the cracking resistance may be reduced.

(2) At present, in order to speed up the construction, early strength agents are usually added to the concrete mixture by contractors. In this way, the concrete can achieve the required strength quickly, but the modulus of elasticity cannot be improved simultaneously, which

causes excessive deformation after prestressing of the members.

By referring to the provisions in Clauses 7.7.4 and 7.8.5 of *Technical Specification for Construction of Highway Bridge and Culvert* (JTG/T F50—2011), the required concrete strength during prestressing is adjusted to be 80% in this clause, and the requirement for the modulus of elasticity is also added.

7.2.4 ~ 7.2.6 Provisions on stress analysis and stress limits for reinforced concrete flexural members in transient situations are specified in these clauses. They are explained as follows:

(1) The calculation formulas for reinforced concrete structures at the elastic stage are used in the stress analysis for all cross-sections and inclined sections, and the basic assumptions related to flexural members at the elastic stage are applicable.

(2) When longitudinal reinforcement is provided in the compression zone, in calculating the depth of compression zone x_0 and the moment of inertia I_{cr}, the stress in compression reinforcement shall satisfy the condition of $\alpha_{ES}\sigma^{t}_{cc} \leqslant f'_{sd}$. If $\alpha_{ES}\sigma^{t}_{cc} > f'_{sd}$, the term $\alpha_{ES}A'_s$ in each formula shall be replaced by $\frac{f'_{sd}}{\sigma^{t}_{cc}}A'_s$, in which f'_{sd} is the design strength of compression reinforcement, σ^{t}_{cc} is the corresponding compressive stress in concrete at the point of the resultant force in compression reinforcement. When multiple layers of reinforcement are provided in the tension zone, the term $\alpha_{ES}A_s(h_0 - x_0)^2$ in the calculation formula for the moment of inertia of the cracked section shall be replaced by $\alpha_{ES}\sum_{i=1}^{n}A_{si}(h_{0i} - x_0)^2$, in which n is the number of layers of tension reinforcement, A_{si} is the cross-sectional area of all reinforcement at the ith layer, and h_{0i} is the distance from the centroid of the ith layer of reinforcement A_{si} to the extreme compression fiber.

7.2.8 Normal stress limits of concrete in the precompression zone and the pretension zone during prestressing are specified for prestressed concrete flexural members in this clause. The precompression and pretension zones refer to the compression and tension zones formed during prestressing.

The compressive stress limit of concrete at the extreme fiber of the precompression zone has been determined by gathering construction experiences in China and referring to national and international specifications. It is also the specified value in *Specifications JTJ 023-85*, and a limit value combined for high-strength concrete and normal concrete. Excessive stress in the precompression zone of concrete causes not only large camber but also the possible occurrence of cracks in the longitudinal direction of members. This provision is demonstrated to be proper with years of practice.

The tensile stress σ^{t}_{ct} at the extreme fiber of concrete in pretension zones shall not exceed $1.15f'_{tk}$, which is the value specified in *Specifications JTJ 023-85*. When $\sigma^{t}_{ct} \leqslant 0.7f'_{tk}$, pretension zones generally will not crack. However, a certain number of longitudinal reinforcements shall still be provided in the pretension zone to distribute potential cracks evenly because the tensile strength of concrete scatters seriously. This requirement is somewhat higher than that in *Specifications JTJ 023-85.* When $\sigma_{ct}^{t}=1.15f_{tk}'$, it is still smaller than $\gamma f_{tk}'$, theoretically, cracking does not happen in general. But the tensile stress is already very high. If the prestress is not accurate or the tensile strain in concrete is too discrete, it is prone to cracking, and more longitudinal reinforcement is required.

The diameter of longitudinal reinforcement arranged in pretension zones can be as small as possible to facilitate the uniform distribution of the cracks.

8 Requirements for Member Analysis

8.1 Composite Flexural Members

8.1.1 Composite flexural members in this section refer to the flexural members in which precast members are used as supports during construction and combined with concrete layers poured on them.

8.1.3 In the analysis of composite flexural members, stresses caused by the different shrinkage between the cast-in-place concrete layers and the precast members shall be considered. They can be calculated by lowering the temperature of the cast-in-place concrete layers to some degree, in which the principle is similar to that in the analysis of thermal stresses when the temperature in the cast-in-place concrete layers is lower than that in the precast members.

8.1.6 Experimental studies showed that the main factors that affect shear resistance on interfaces of composite members were concrete strength, stirrup reinforcement ratio and tensile strength of reinforcement. Based on the experimental data, when the stirrup reinforcement ratio is equal to or larger than 0.001, it has a significant effect on the shear resistance, and the approximate regression is:

$$\frac{\tau_u}{f_{cd}} = 0.14 + \rho_{sv}\frac{f_{sd}}{f_{cd}} \tag{8-1}$$

where:

τ_u——ultimate shear stress on interface;

f_{sd}——design tensile strength of stirrup;

f_{cd}——design axial compressive strength of concrete;

ρ_{sv}——reinforcement ratio of the stirrup, $\rho_{sv} = A_{sv}/bs_v$, in which A_{sv} is the total cross-sectional area of each leg of the stirrup in one cross-section of the composite beam, s_v is the stirrups spacing, and b is the width of the interface in the beam.

From Figure 8-1, the shear force on the interface of the inclined section is F, the reaction force of support is V, the shear stress on the interface is τ, the distance from the support to the section under investigation is a, the width of the beam interface is b, the lever arm of force is z, and the equilibrium of all forces is:

$$Va = \tau abz$$

Then:

$$\tau = \frac{V}{bz} \tag{8-2}$$

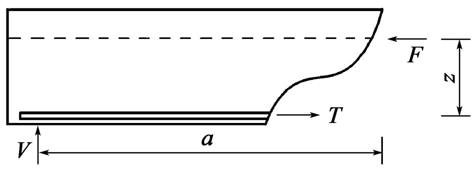


Figure 8-1 Shear on the interface of a composite beam

F-horizontal shear force on the interface; V-vertical shear; T-tension of primary reinforcement; z-internal lever arm

Substituting $\tau = \tau_u$, $V = V_d$, $z = 0.85h_0$ into Eq. (8-2), and then substituting the equation into Eq. (8-1), introducing $\rho_{sv} = A_{sv}/bs_v$ and the importance factor of structure γ_0, we can obtain:

$$\gamma_0 V_d \leqslant 0.12 f_{cd} bh_0 + 0.85 f_{sv} \frac{A_{sv}}{s_v} h_0 \tag{8-3}$$

The above formula is Eq. (8.1.6) in this clause.

8.1.7 Eqs. (8.1.7-1) and (8.1.7-2) in this clause are established by reference to Clause 8.16.6.5.3 in *US AASHTO Specifications 14 th Edition* (the Imperial units have been converted into the SI units).

8.1.11 Considering the features of composite members, C_2 expressed by Eq. (8.1.11-1) in this clause is different from that in Clause 6.4.3 in the *Specifications*. Eq. (8.1.11-2) in this clause is established by reference to Eq. (7.5.9-1) in *Code for Design of Concrete Structures* (GBJ 10-89). The limit value in Eq. (8.1.11-2) is taken as $0.75f_{sk}$ by considering that the stress in tension reinforcement of the precast member, which has a small section depth to carry all the loads at the construction stage, is greater than that when assuming the full section of the composite member to carry the same loads. After the cast-in-place concrete reaches the design strength to form the composite member, the whole section is subjected to M_{2s}, and the stress increment is produced in the tension reinforcement again. At this time, the stress in tension reinforcement of the composite member is still larger than that of a general integral member with the same section, and the tension reinforcement is likely to yield prematurely under the action of $M_s = M_{1Gk} + M_{2s}$. Hence, σ_{ss} is limited to $0.75f_{sk}$. In Eq. (8.1.11-2), when $M_{1Gk} < 0.35M_{1u}$, $h_1 = h$ is used, which is specified by reference to Clause 7.5.9 in *Code GBJ* 10-89 and Subsection 9.3.1 in *Concrete Structures* (*Basic Members*) authored by WANG Yijun and published by China Architecture & Building Press (in Chinese).

8.1.12, 8.1.13 Deflections of composite flexural members are larger than those of general integral members, and hence, the calculated stiffness of the composite flexural members shall be smaller than that of the general integral members. By referring to the calculation formula for the short-term stiffness of general flexural members in Clause 5.3.3 and that of composite flexural members in Clause 7.5.15 of *Code GBJ 10-89*, after comparsion and analysis, the stiffness of reinforced concrete or prestressed concrete composite flexural members is specified to be the product of the stiffness calculated according to Eqs. (6.5.2-1) or (6.5.2-4) in the *Specifications* multiplied by a reduction factor of 0.9 or 0.85, respectively.

8.1.14 By referring to the long-term stiffness of general flexural members in Clause 5.3.2 and that of composite members in Clause 7.5.14 of *Code GBJ 10-89*, after comparsion and analysis, the long-term development factor of deflection for composite flexural members is determined in this clause according to the situations of general flexural members and composite members in the *Specifications*.

8.1.15 This clause is made by reference to Clause 7.5.13 in *Code GBJ 10-89*.

8.2 Post-tensioned Concrete Anchorage Zones

8.2.1 Post-tensioned concrete anchorage zones, which are subjected to concentrated prestressing anchor force and have problems of local bearing and stress diffusion, are the typical disturbed regions in concrete bridges. By reference to *AASHTO LRFD Bridge Design Specifications*, the post-tensioned anchorage zones are further divided into local zone and general zone (Figure 8-2), so that separate analysis is conducted according to their mechanical behaviors.

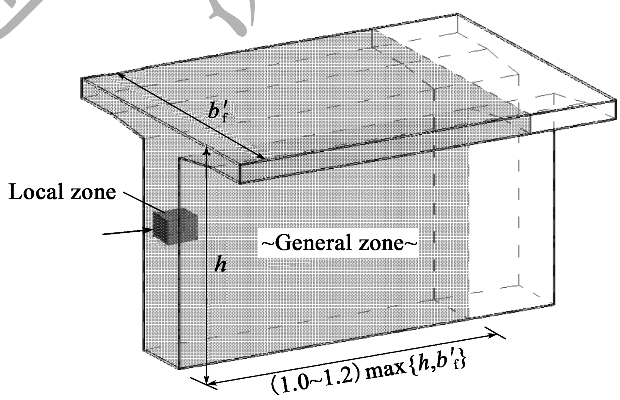


Figure 8-2 Division of general and local zones

The local zone is the region ahead of the anchorage which is directly subjected to anchor force, where triaxial compression is the main concern. Besides the local bearing ahead of the anchorage shall be checked, the tested force transmission performance of the anchorage device products shall meet the requirements in the current standard *Prestressing Strand Anchorage, Grip and Coupler for*

Highway Bridge(JT/T 329).

The general zone is the anchorage zone excluding the local zone, where tensile stresses due to the spreading of the prestressing force shall be concerned, and reinforcements against cracking shall be designed. It has been demonstrated from studies that the reinforcement meeting the conditions for the resistance check of ties in Clause B. 3. 3 can satisfy the requirements for crack resistant design. With reference to *Code for Design of Concrete Structures* (GB 50010—2010) and *AASHTO LRFD Bridge Design Specifications*, in the calculation for the design tensile force of each tensile part in the general zone, the design anchor force P_d is taken as 1. 2 times the maximum jacking force; meanwhile, it is required that the stress in tension reinforcement is not larger than the design tensile strength of reinforcing steel under the action of the design anchor force. According to a great number of experiments, the reinforcement design based on the above method enables the crack width in the anchorage zone to satisfy the requirements for the service performance of the structure, that is, the crack width does not exceed 0. 15 mm. Thereinto, the reference for *AASHTO LRFD Bridge Design Specifications* is the experimental research outcomes from University of Texas (totally more than 100 specimens), and the reference for *Code for Design of Concrete Structures* (GB 50010) is the experimental research outcomes from Tsinghua University and China Academy of Building Research (totally more than 50 specimens).

There are several tension zones in the general zone of the post-tensioned concrete anchorage zone:

(1) During the spreading of anchor force from the bearing plate to the full section, transverse tensile stress (or bursting stress) will be produced and form the resultant called the bursting force. In the view of the principles of mechanics, two load paths can be used to reflect the spreading of the anchor force. According to the equilibrium conditions of forces, transverse bursting forces are inevitable in the transition regions of compressive forces, as shown in Figure 8-3a). Moreover, from the three-dimensional finite element analysis, transverse stress distribution along the line of the anchor force can be obtained, and then the bursting force can also be obtained by the integral of the area of transverse tensile stresses, as shown in Figure 8-3b).

(2) When an anchor force is applied beyond the section core (if the force acted on it, only the longitudinal compressive stress is produced), longitudinal tensile stresses still exist along the edges at the tensile sides of the anchorage zones [Figure 8-4a)], and their resultant is the edge tension force.

(3) The spalling stresses in the vicinity of an anchorage plate are induced by the indentation of the anchorage surface, and their resultant is called spalling force [Figure 8-4b)].

8.2.2 The calculation method for bursting forces acting ahead of the anchorage devices at the end

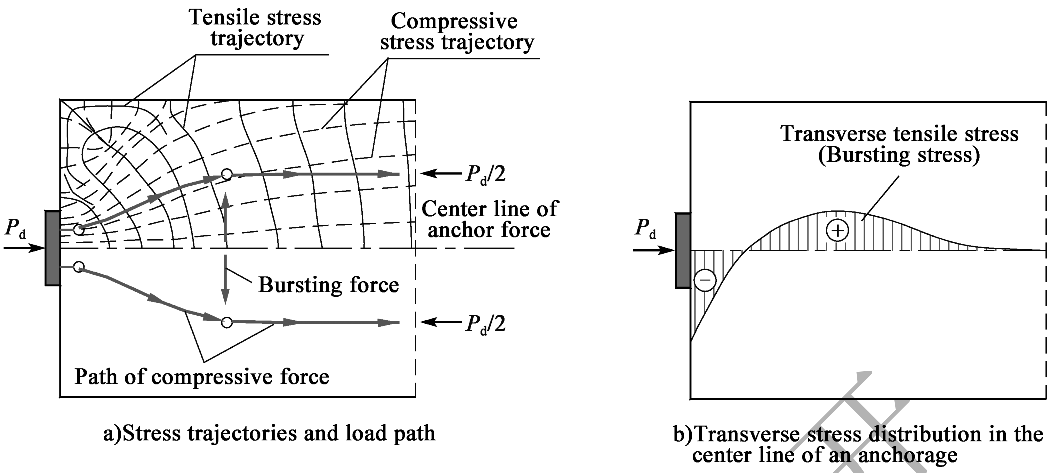


Figure 8-3 Principle for generation of bursting force in the end anchorage zone

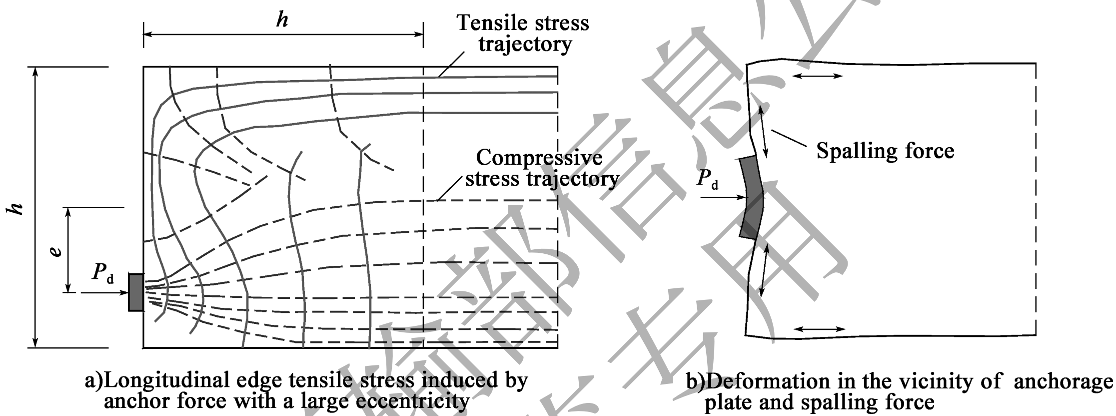


Figure 8-4 Principles for generation of longitudinal edge tension force and spalling force in the end anchorage zone

anchorage zones is given in this clause (Figure 8-4). For a T-beam (or box girder), its web in the end anchorage zone may be regarded as a rectangular cross-section to resist the end anchor force, and the eccentricity e is still counted from the section centroid of the original T-beam (or box girder).

Based on the relevant provisions in *AASHTO LRFD Bridge Design Specifications*, the calculation formula (8.2.2-1) for the bursting force induced by a single anchorage device is derived by considering the influence of eccentricity of anchorage. This formula is used for the calculation of bursting force in the principal plane of anchorage and has adequate computational accuracy when the depth-to-width ratio of the section is larger than 3. Research has shown that the bursting force caused by the spreading of anchor force along the direction of width is small and distributed in the local zone ahead of the bearing plate, and hence does not affect the bursting force in the principal plane.

The effects of the eccentricity of anchorage and the angle of inclination of the tendon are taken into account as the main factors in determining the point of application of the bursting force d_b. The sign of the tendon inclination is decided according to Figure 8-5: it is positive if the anchor force points toward the centroid of the section, and it is negative if the anchor force points away from the centroid of the section.

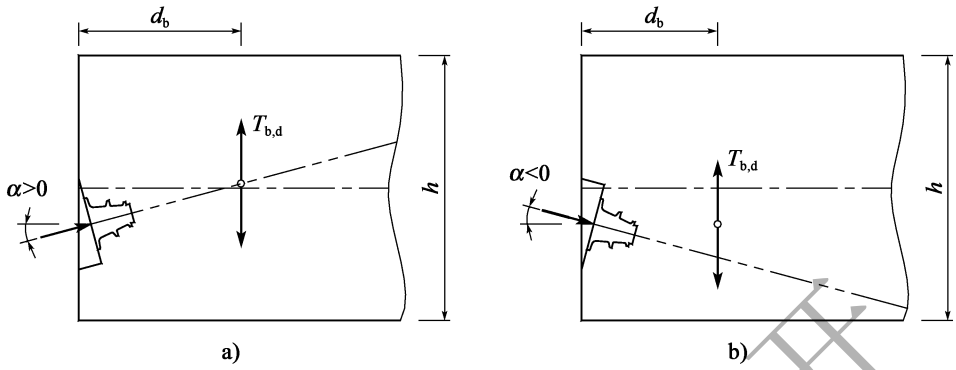


Figure 8-5 Positive and negative inclination of a tendon

Finite element analysis results have shown that the spreading effect caused by a single equivalent anchor force may be approximately regarded to be the same as that caused by a group of anchor forces when the spacing among multiple anchor forces is small. According to *AASHTO LRFD Bridge Design Specifications*, when the center-to-center spacing s between adjacent anchorage devices is smaller than 1.5 times the width a of the bearing plate, i.e., $s<1.5a$, they can be considered as a group of closely spaced anchorages. Research has shown that the requirement for this spacing may be extended to be $s<2a$.

8.2.3 At the end anchorage zones, the local depression due to anchor force is required to be compatible with the deformation in its vicinity, which causes the bursting stress in the surface. The peak stress can be as large as 0.5 times the average compressive stress in the full section induced by the anchor force, but it is reduced rapidly from the surface to the inside. From the integral of the bursting stress across its distribution area, the bursting force on the surface can be obtained (Figure 8-6). According to the previous studies, the bursting force is normally not larger than 2% of the anchor force.

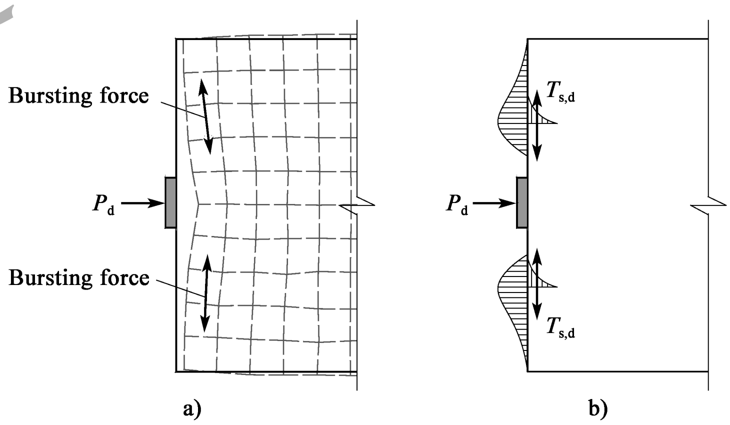
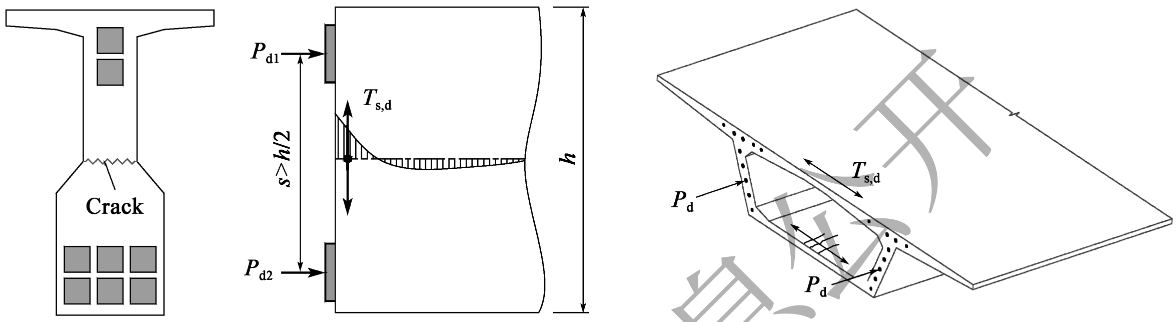


Figure 8-6 Deformation and bursting force in the vicinity of the anchorage plane

8.2.4 At the end anchorage zones, the spreading of anchor forces with a large spacing can cause bursting force at the end surface, for example, when the spacing between two groups of anchorages on the upper and bottom of a T-beam end is large, bursting stress can occur on the end surface of the anchorage, and bursting cracks may be developed [Figure 8-7a)]; for anchor forces applied to webs of a box beam, a portion of forces spread toward the bottom and top slabs, which can cause transverse bursting forces at the front ends of the bottom and top slabs, and may result in longitudinal cracking [Figure 8-7a)].



a)Bursting force in the web of T-beam b)Bursting force in slabs of box girder

Figure 8-7 Bursting forces between anchorages with a large spacing as well as the potential cracks

8.2.5 The calculation method for the edge tension force in the case of anchorages with large eccentricity is given in this clause. During the jacking of prestressing steel in batches and in sequence for post-tensioned beams, the case of anchorages with large eccentricity may be present, and hence, the most unfavorable tensile forces along the edges at the tensile side need to be calculated.

8.2.6 Due to the action of concentrated anchor force, the abrupt change in geometric shape and the action of radial force induced by the bending of prestressing strands, the anchorage zones of post-tensioned anchor blisters become typical disturbed regions with very complicated mechanical behaviors, where reinforcement needs to be provided to satisfy the requirement of cracking resistance and load-carrying capacity.

Three-dimensional finite element analysis results have shown the following typical local stresses existing in the blisters (Figure 8-8):

(1) Transverse tensile stress distributed ahead of the anchorage of the blister, which is called as "bursting effect ahead of anchorage".

(2) Concentration of tensile stress in the concave angle region at the root of the blister end surface, which is called as "tensile effect at the cantilever blister root".

(3) Concentration of tensile stress behind the anchorage, which is called as "tensioning effect

behind anchorage".

(4) Tensile stress at the lower edge of the bottom plate, which is called as "local bending effect".

(5) Concentration of tensile stress at the deviation region of prestressing tendon, which is induced by the "radial force effect".

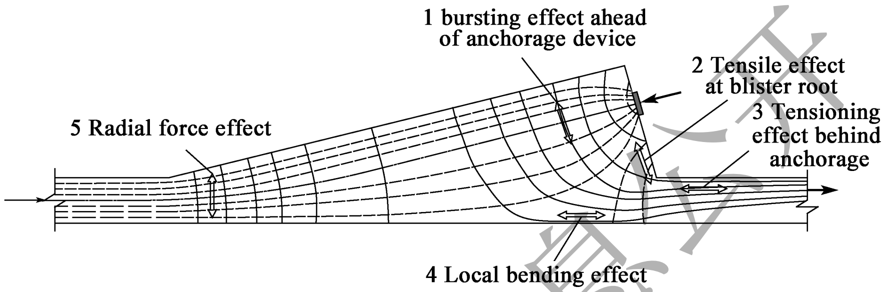


Figure 8-8 Five local tensile actions in anchorage zone of post-tensioned anchor blister

The calculation formulas for design tensile forces in the five tensile locations given in this clause are applicable to general anchorage zones of blisters. Provisions for the reinforcement to resist these forces are given in Clause 9.4.20. The sources of these formulas are:

(1) The tensile force $T_{s,d}$ at the root of a blister is mainly caused by the indentation in the vicinity of the anchorage device and the concentration of stress at the concave angle. Finite element analysis results have shown that its value is larger than the bursting force, hence it is approximately taken as 4% of the anchor force.

(2) Tensioning stress behind the anchorage of the blister is mainly distributed in the internal surface behind the blister. Its distribution range is small, but the resultant $T_{tb,d}$ is large, and hence the portion behind the anchorage is susceptible to tension cracking. Although the longitudinal compressive stress in the wall plate at this portion can counteract a certain tensioning stress behind the anchorage, it is generally neglected for the sake of safety. In this clause, it is expressed by the design prestressing anchor force P_d and taken as $0.2P_d$.

(3) The design tensile forces in the other three locations are used for isolated blisters and derived based on the case of in-plane forces. Among which, the bursting force $T_{b,d}$ ahead of the anchorage device of the blister and the edge tension force $T_{et,d}$ due to local bending are calculated with similar methods for the bursting force and the edge tension force in the end anchorage zones with large eccentricity; the calculation formula for the tensile force $T_{R,d}$ due to radial force is obtained based on the self-equilibrium condition of equivalent load for prestressing force, $T_{R,d} = 2P_d \sin(\alpha/2) \approx P_d \alpha$.

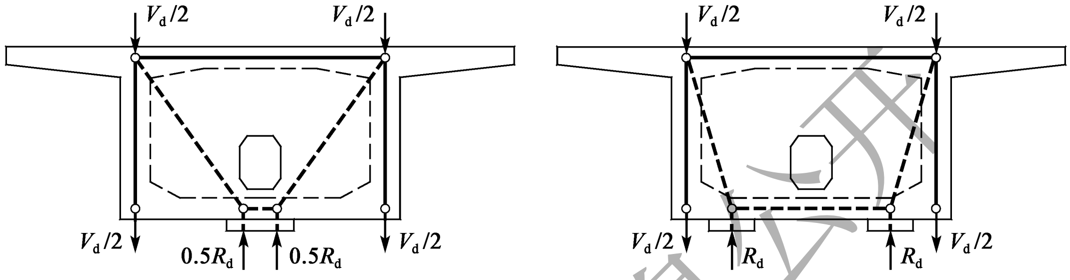
It is worth noting that the area of primary reinforcement calculated by the formulas in this clause is the minimum theoretical value to resist these five local tensile force effects, instead of the value added to the existing construction reinforcement arrangement in these locations. For example, for a blister on the bottom slab of a box beam, if the longitudinal reinforcement in the upper and lower layers of the bottom slab considering the overall design is already more than the required reinforcement to resist "tensioning force behind anchorage" and "local edge bending effect", it may not need to be increased. If it is smaller than the calculated value, additional reinforcement is required for strengthening.

8.3 Diaphragms at Supports

8.3.1 Diaphragms at supports are generally dominated by stresses in the transverse direction of the bridge. For the diaphragm in a single-cell box girder, if the center lines of the supports on the diaphragm coincide with that of the webs in the transverse direction of the bridge, loads within the span are directly transmitted to the supports without passing through the diaphragm, then the reinforcement for the diaphragm may be determined only according to the detailing requirements; if the center lines of the supports deviate from those of the webs, the reinforcement for the diaphragm shall be determined through analysis.

Studies have shown that loads within the span are primarily transmitted to the diaphragm by the distributed shear force in the webs. In the design of reinforcement for the diaphragm, the tensile force at the top of the diaphragm due to transverse bending is mainly concerned. Analyses have indicated that, if the span-to-depth ratio of the diaphragm $B_w/h > 2$, the normal stress in the mid-span section conforms to linear distribution, and the diaphragm may be designed as a shallow beam; if the span-to-depth ratio of the diaphragm $B_w/h \leq 2$, it may be regarded as a disturbed region, the top tensile force at the central section of the diaphragm should be calculated in accordance with Clause 8.3.2, or may be calculated by using the strut-and-tie model or the three-dimensional finite element model. Figure 8-9 shows strut-and-tie models of diaphragms on single support or double supports, and reflects the load paths in the diaphragms and the induced tensile effects. In general, joints in the diaphragms at the supports are dispersive joints (joints without clear geometric boundaries), the design is not controlled by the resistances of struts and joints, and only the reinforcement in the tie needs to be checked.

8.3.2 It is shown by studies that if the span-to-depth ratio of diaphragms $B_w/h > 2$, the transverse tensile force in the central section of this type of deep beam may be calculated as shallow beam; if the span-to-depth ratio $B_w/h \leq 0.5$, the transverse tensile force approaches to be a constant of $0.2V_d$, in addition, such narrow and deep diaphragms are seldom used in actual bridges. Diaphragms in a single-cell box with a span-to-depth ratio $0.5 \leq B_w/h \leq 2$ are widely used in practice, and the design for their top reinforcement for cracking resistance or the design for the transverse prestressing shall be the focus. The equation in this clause is derived according to the force diagram for deep beams with two cantilever ends subjected to uniform shear forces in the webs. When the span-to-depth ratio is in the range of 0.5 ~ 2, the top transverse tensile force approximately presents linear variation. By comparison with the elastic finite element analysis results (Figure 8-10), it is shown that this equation can better take into account the effects of the span-to-depth ratio of the diaphragm and the spacing of supports on the top transverse tensile force.



a)Diaphragm on single support

b)Diaphragm on double supports

Figure 8-9 Strut-and-tie models of diaphragms at supports

It is shown through three-dimensional finite element analysis results that support reactions have the characteristics of spatially spreading in the regions of pier tops. The transverse tensile effect exists not only in the extent of the diaphragm thickness (b) but also in the top slab of the box girder with a certain range at its two sides. Viewing along the longitudinal direction of the bridge, top reinforcing steel and/or transverse prestressing steel within the extent of $3b$ in the region on the pier top are effective to resist the transverse tensile force $T_{t,d}$ (Figure 8-11); viewing along the direction of girder depth, reinforcing steel and/or transverse prestressing steel in the top slab of the box girder are effective to resist the transverse tensile force.

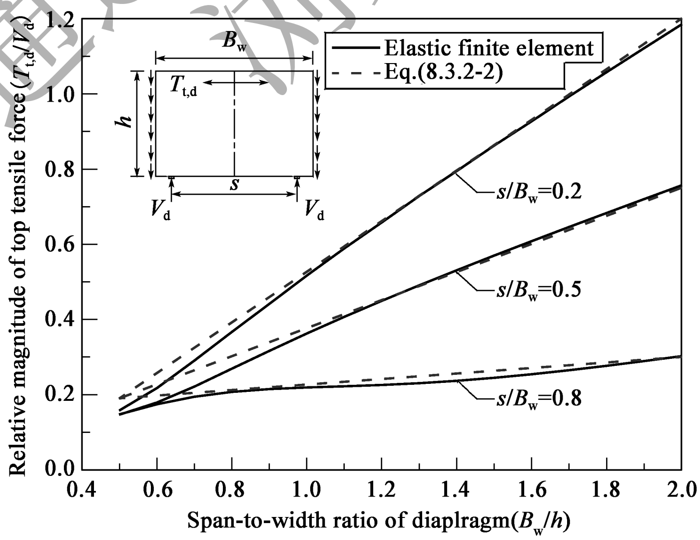
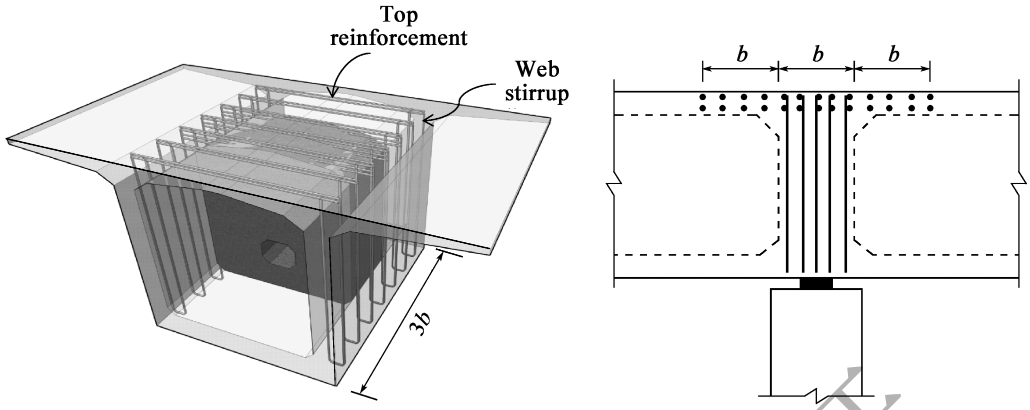


Figure 8-10 Comparison of calculation results from the equation and finite element analysis



a)3D layout b)Elevation layout

Figure 8-11 Layouts of reinforcing steel in diaphragm to resist top horizontal tensile forces

8.4 Cap Beams on Piers and Abutments

8.4.1 Bent piers and abutments in the transverse direction of bridges are frame structures formed by cap beams and columns (piles). It was specified in the former edition of the *Specifications* that, if the ratio of the flexural stiffness per unit length (EI/l) of the cap beam to that of the column is larger than 5, the cap beam can be simplified as a simply supported beam for analysis; and it can be simplified as a continuous beam if it has multiple spans, in which EI is the flexural stiffness, l is the length of the cap beam or the column and may be taken as the axial length between structural nodes. With the wide applications of pile foundation and column-type piers, the size of columns (piles) is increased and their number is decreased, so that the ratio of EI/l of the cap beam and the column is generally not larger than 5, and it is changed in this revision that bent piers shall be analyzed as rigid frames.

To facilitate the analysis, each pile may be simulated as an equivalent pile with fixed bottom as shown in Figure 8-12, and the depth of the fixed point is normally taken as $1.8/\alpha$, where α is the deformation coefficient of the pile taken in accordance with the provisions in *Specifications for Design of Foundation of Highway Bridges and Culverts* (JTG 3363).

8.4.2 Experimental studies worldwide in the recent two decades have shown that, simply supported beams with $2.0 < l/h \leqslant 5.0$ and continuous beams with $2.5 \leqslant l/h \leqslant 5.0$ can be regarded as "short beams", their mechanical behaviors are similar to those of deep beams but are different from ordinary shallow beams. Therefore, all the beams with $l/h \leqslant 5$ are called deep flexural members (including short beams and deep beams) in the hydraulic department [see *Design Code for Hydraulic Concrete Structures* (DL/T 5057—1996) (hereinafter referred to as *Code DL/T 5057—1996*)] and the building department in China. The section analysis for deep flexural members is different from that for ordinary flexural members. There are special requirements for the detailing of deep beams.

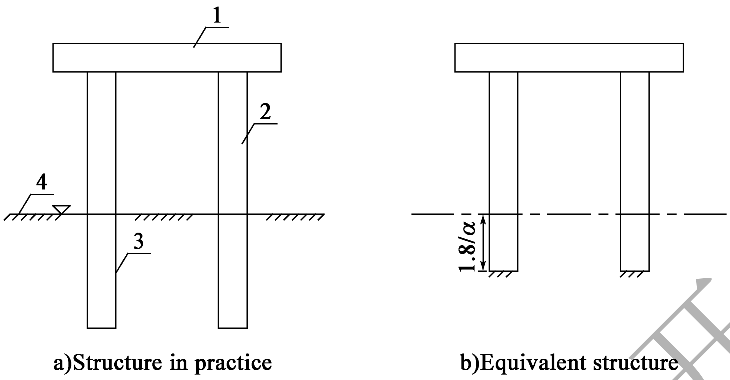


Figure 8-12 Structural simulation of pile in pile bent pier

1-cap beam; 2-column; 3-pile; 4-ground

According to the investigation, the span-to-depth l/h of cap beams on piers and abutments of highway bridges is mostly between 3 ~ 5. They are deep flexural members, and categorized as short beams but not deep beams. Hence, they shall be analyzed as deep flexural members while their detailing needs not to comply with the special requirements for deep beams.

Deep flexural members are different from ordinary flexural members, and the reasons for setting their span-to-depth ratio l/h not to be larger than 5 (refer to *Code DL/T 5057—1996*) are:

(1) According to the finite element analysis and measured data from structural tests, it can be found from the strain distribution at the mid-span section on both simply supported beams and continuous beams, and that at the sections at interior supports on continuous beams, as well as the average strain distribution after cracking of the above mentioned sections, that the assumption of the plane sections remain plane is not applicable to the beams with $l/h \leq 5$.

(2) According to the tests of beams subjected to shear, diagonal tension failure cannot occur in the beams with $l/h \leq 5$.

(3) According to the elastic analysis for beams with $l/h \leq 5$, the shear effect on their deflection is only about 7.8%, which can be neglected.

(4) Similar provisions can be seen in US standard ACI 318-89, Canada standard CAN 3-A23-3-M84 and New Zealand standard NZS 3101 1982, etc.

8.4.3 The expression with the internal lever arm is still used for the flexural resistance on the cross-section of deep flexural members. The two formulas in this clause are specified by reference to relevant information from the building department in China. Since the assumption of the plane sections remain plane after deformation is not applicable to deep flexural members, the internal lever arms z are smaller than those of ordinary flexural members, and hence, they are multiplied by correction factors.

8.4.4 The control conditions for the dimensions of sections resisting shear in reinforced concrete cap beams are determined in accordance with Eq. (5.2.11) of the *Specifications* and by reference to relevant information from the building department in China. According to the equation in this clause, the calculation result for beams with $l/h \leqslant 5$ is consistent with that from Eq. (5.2.11); the calculation result for beams with $l/h = 2$ is 0.8 times that from Eq. (5.2.11), this ratio is consistent with that obtained from the corresponding formula in the building department of China.

8.4.5 The calculation formula for the shear resistance on inclined section of reinforced concrete cap beams is established in accordance with Eq. (5.2.9-2) and by reference to Clause 10.6.4 in *Code DL/T 5057—1996* and relevant information from the building department in China. According to this formula, the calculation result for beams with $l/h \leqslant 5$ is consistent with that from Eq. (5.2.9-2), and increases with the decreasing span-to-depth ratio; the calculation result for beams with $l/h = 2$ is 1.33 times that from Eq. (5.2.9-2), this ratio is close to that obtained from the corresponding formula in the building department of China.

8.4.6 When the overhangs of cap beams on piers are subjected to concentrated force, and the distance from the force to the column edge is smaller than or equal to the section depth of the cap beams, they belong to deep cantilever beams and may be analyzed by the strut-and-tie model. The calculation methods for the internal forces in ties and struts as well as their load-carrying capacities are given in this clause.

Circular columns are transformed to square columns with reference to Clause 4.2.8 in *Specification for Design of Reinforced Concrete Pile Caps* (CECS 88:97) (hereinafter referred to as *Specification CECS 88:97*). In the calculation for the punching shear resistance and the shear resistance on an inclined section of pile caps, circular piles are transformed to square piles with a side length equal to 0.8 times the diameter. According to the plastic resistance of slabs to the punching shear of circular columns and square columns, the diameter d of the circular columns and the side b of the square columns have the transformation relationship of $b = (\pi/4)d$; according to the experimental data of other countries, they have the relationship of $d = 1.2b$; and hence, $b = 0.8d$ is adopted. This value is used for the transformation between circular and square columns hereinafter.

8.4.7 Cap beams (pier caps) on single-column piers with double bearings are common in practice. The mechanical behaviors in this type of cap beam are similar to those in deep beams or corbels. The suggestion on the configuration of strut-and-tie model and the calculation method for internal forces in top ties are given in this clause. In general, in the strut-and-tie models of pier caps, the effective cross-sectional area of diagonal ties is large, the design is not controlled by the resistances of struts and joints, and hence, only the reinforcements in ties are required to be checked.

8.4.8 The characteristic crack width of a reinforced concrete cap beam may be calculated using the formula for cracks in ordinary members, but the coefficient related to internal forces of the members [equivalent to C_3 in Eq. (6.4.3)] is taken as $\frac{1}{3}\left(\frac{0.4l}{h}+1\right)$. When $l/h=5$, it becomes the formula for ordinary flexural members. The above formula is established by reference to Clause 10.6.10 in *Code DL/T 5057—1996*, which was verified by tests on 34 simply supported beams.

8.4.9 Reinforced concrete cap beams with a span-to-depth of $l/h \leqslant 5$ have large flexural stiffness per unit length, and their deflections may satisfy the requirement specified in Clause 6.5.3, then they may not need to be checked.

8.5 Pile Caps

8.5.1 Eq. (8.5.1) in this clause is a simplified formula. For pile caps in super-large or large bridges, especially elevated pile caps under the action of strong horizontal forces like earthquakes, more accurate calculation methods shall be used, for example, considering the deformation of soil (m method, etc.) in the calculation.

(Translator's note: In m method, the horizontal reactions of the soil are assumed to be proportional to the lateral deflections of the pile, and the reaction coefficient of the soil is assumed to be proportional to the deep of the stratum. The proportionality coefficient of the soil is presented by m and the method is named after it.)

8.5.2 Two methods with "beam-like model" and "strut-and-tie model" are used for the calculation of the ultimate load capacity of pile caps. The former is adopted in two Chinese specifications, and the latter is specified in Clause 5.6.31 of *AASHTO LRFD Bridge Design Specifications*. For corbels and brackets that have similar properties to pile caps, it is specified in Clause 5.13.2.4.1 of *AASHTO LRFD Bridge Design Specifications* that, if the length of the cantilever is greater than the depth of brackets or corbels, the component is analyzed as a cantilever beam; if the length of cantilever is less than the depth of brackets or corbels, the strut-and-tie model is applied for the analysis. In the *Preliminary Recommendations on Design of Reinforced and Prestressed Concrete Structures* published by the Fédération Internationale de la Précontrainte (FIP) in 1982, it was specified that the component was considered as a deep beam if the ratio of the length of the cantilever to the depth of the beam was smaller than or equal to 1. Accordingly, it is specified in the *Specifications* that the beam-like model is applied for the analysis if the distance from the center of the exterior row of piles to the edge of the pier or abutment is larger than the depth of the pile cap; and the strut-and-tie model is applied for the analysis if the distance from the center of the exterior row of piles to the edge of pier or abutment is less than or equal to the depth of pile cap.

The beam-like model is traditionally used in the analysis of pile caps. According to the relevant commentary in *Technical Code for Building Pile Foundation* (JGJ 94-94), the failure mode of pile caps is similar to that of beams, that is, flexural cracks present at two edges parallel to piers or abutments, which indicates that a pile cap subjected to loads in two directions behaves as two beams subjected to load in each direction rather than as a two-way slab. Considering that the horizontal forces and bending moments are simultaneously applied to highway bridges in two directions, vertical forces in the piles of the same row are different. When the direction of the horizontal forces and bending moments change to the opposite one, the pile with maximum vertical force will subject to the minimum vertical force, vise versus. Therefore, the maximum vertical force in one pile is taken as the calculated vertical force for all piles in the same row.

8.5.3 Generally, stirrups or bent bars are not provided in pile caps of highway bridges in China. The shear on the inclined section is mainly resisted by concrete and is expressed by reference to the first item in the right-hand side of Eq. (5.1.10-2) in *Specifications JTJ 023-85* as:

$$Q_h = \frac{0.008(2+P)\sqrt{R}bh_0}{m} \tag{8-4}$$

where:

Q_h——shear force resisted by concrete (kN);

R——concrete (strength) mark in *Specifications JTJ 023-85*;

b——minimum web width at the section on the top of compression zone through inclined section (cm);

h_0——effective depth of member (cm);

m——shear span-to-depth ratio.

In this formula, P is the reinforcement percentage of longitudinal tension reinforcement, which shows that longitudinal reinforcement has a certain contribution to resist shear, can hinder the development of inclined cracks and stop the rising of the neural axis. In this revision, $(2+P)$ is changed to $(2+0.6P)$ (see the commentary on Clause 5.2.9), concrete "strength mark" is changed to "strength class" (Translator's note: concrete strength mark is expressed by kgf/cm^2), and all units of measurement are changed to SI system, finally the importance factor of structure γ_0 is introduced after synthesizing various factors, then Eq. (8.5.3) in this clause is obtained as:

$$\gamma_0 V_d \leqslant \frac{0.9\times10^{-4}\times(2+0.6P)\sqrt{f_{cu,k}}}{m}bh_0 \tag{8-5}$$

The provision that $m=0.5$ is adopted when $m<0.5$ was determined after comparing with Eq. (5.6.8-2) in *Code JGJ 94-94* to avoid large shear resistance resulting from Eq. (8-5) for cap beams with very small shear span-to-depth ratio.

Pile caps are deep beams with short overhangs, and their shear span-to-depth ratio is much smaller than that of ordinary beams. Since few experimental data were available, a comparison was made with the calculation formula in *Code JGJ 94-94*, as shown in Table 8-1.

Table 8-1 Comparison of shear resistance on inclined section

Shear span-to-depth ratio	JGJ 94-94 ($\times bh_0$)N	The *Specifications* ($\times bh_0$)N		Shear span-to-depth ratio	JGJ 94-94 ($\times bh_0$)N	The *Specifications* ($\times bh_0$)N	
0.3	2.50	$P=0.20$	1.91	1.1	1.07	$P=0.60$	0.97
0.5	1.88	$P=0.30$	1.96	1.3	0.94	$P=0.70$	0.84
0.7	1.50	$P=0.40$	1.44	1.5	0.83	$P=0.80$	0.74
0.9	1.25	$P=0.50$	1.15	1.7	0.78	$P=0.90$	0.67

Notes: 1. Concrete strength class is C25 in this table; b is the effective width of pile cap(mm); h_0 is the effective depth of pile cap (mm), see *Code JGJ* 94-94. h_0 is irrelevant to P value since the contribution of longitudinal reinforcement on shear resistance is not considered.

2. According to this formula, the shear span-to-depth ratio is taken as 0.5 when it is less than 0.5.

3. P value is taken by considering its relationship with the shear span-to-depth ratio in the conventional design.

In calculating V_d, the design vertical force in each row of piles is taken as the maximum one multiplied by the number of piles in this row, as shown in Figure 8-13.

Design vertical force in the first row of piles:

$$V_{1d}=5\max(N_{11d}, N_{12d}, N_{13d}, N_{14d}, N_{15d}) \tag{8-6}$$

Design vertical force in the second row of piles:

$$V_{2d}=5\max(N_{21d}, N_{22d}, N_{23d}, N_{24d}, N_{25d}) \tag{8-7}$$

Design vertical force in a single pile N_{ijd} (N_{11d}, N_{21d}, $\cdots$) is calculated in accordance with the provision in Clause 8.5.1.

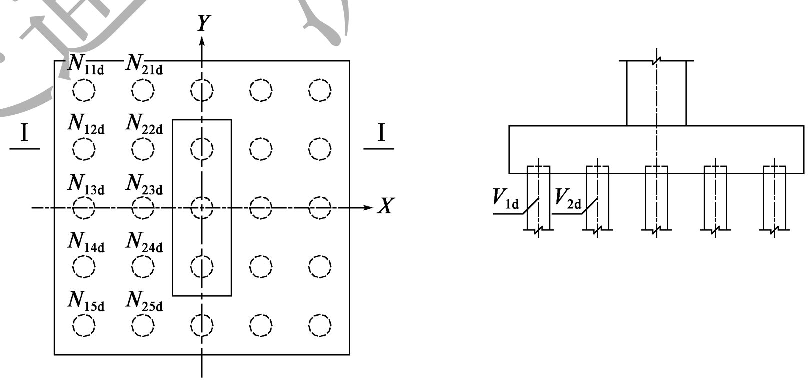


Figure 8-13 Schematic diagram for calculation of V_d of piles

8.5.4 The depth of pile caps in highway bridges is usually large. If the distance from the center of the exterior row of piles to the edge of the pier or abutment is less than or equal to the depth of the pile cap, the design analysis shall be carried out by the strut-and-tie model. For the analysis of pile caps with two piles, the strut-and-tie model suggested in the former edition of the *Specifications* is adopted in this clause (Figure 8.5.4). The force applied on the top surface of the pile cap from the pier is replaced by two concentrated forces at a distance a away from the edges of the pier column, and the reaction points of piles are located at the centers of the piles.

In the study for the locations of the action points in the pier in the former edition of the *Specifications*, the distance from the center of piles in the exterior row to the edge of the pier was firstly selected to be equal to the depth of the pile cap, then the tensile forces at the underside of the pile cap were calculated by the beam-like model and the strut-and-tie model, respectively. Trial computation was carried out to meet the condition that the amount of reinforcement in the bottom of the pile cap by the two models was approximately equal. It was recommended that $a = 0.15h_0$ from the trail computation result. As regards the pile cap of two piles with unequal reactions, a "strut-and-tie model with vertical web members" was suggested in the commentary of the former edition of the *Specifications* (Figure 8-14).

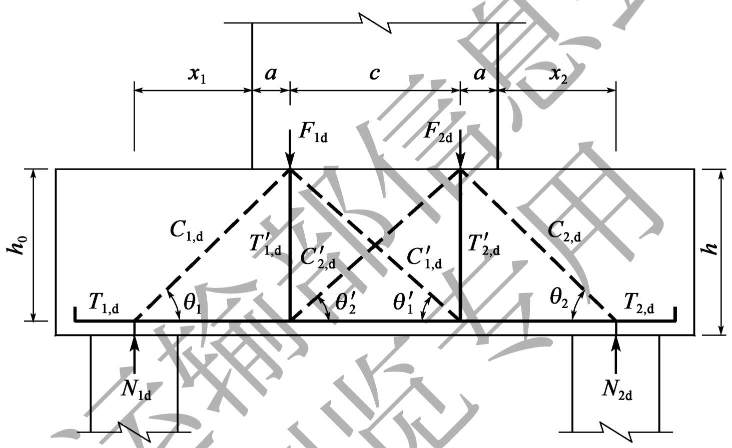


Figure 8-14 Strut-and-tie model with vertical web members

The configuration and force analysis of that model may be explained by the following example. Assume $N_{1d} = 6000$ kN, $N_{2d} = 5000$ kN, $x_1 = x_2 = 1250$ mm, $h_0 = 1880$ mm, $c = 3000$ mm, $a = 0.15h_0 = 282$ mm. According to the geometric relationship among the members in the model, we have: $\theta_1 = \theta_2 = \tan^{-1}\dfrac{h_0}{x_1 + a} = \tan^{-1}\dfrac{1880}{1250 + 282} = 50.82°$, $\theta'_1 = \theta'_2 = \tan^{-1}\dfrac{h_0}{c - 2a} = \tan^{-1}\dfrac{1880}{3000 - 2 \times 282} = 27.66°$. According to the equilibrium condition of the joint at the top of the left pile, we obtain: $C_{1,d} = \dfrac{N_{1d}}{\sin\theta_1} = 7740.3$ kN, $T_{1,d} = \dfrac{N_{1d}}{\tan\theta_1} = 4890.0$ kN, $C'_{1,d} = \dfrac{T_{1d}}{\cos\theta'_1} = 6177.0$kN. According to the equilibrium condition of the joint at the top of the right pile, we obtain: $C_{2,d} = \dfrac{N_{2d}}{\sin\theta_2} = 6450$ kN, $T_{2,d} = \dfrac{N_{2d}}{\tan\theta_2} = 4075.0$ kN, $C'_{2,d} = \dfrac{T_{2d}}{\cos\theta'_2} = 5147.5$ kN. Then, the equilibrium of forces of two intermediate joints at the bottom edges of the pile cap gives: $T'_{1,d} = C'_{1,d}\sin\theta_1 = 3774.0$ kN, $T'_{2,d} = C'_{2,d}\sin\theta_2 = 3145.0$ kN. Accordingly, the concentrated forces on the top of the pile cap may be obtained as: $F_{1d} = C_{1,d}\sin\theta_1 + C'_{1,d}\sin\theta'_1 - T'_{1,d} = 6629.0$ kN; $F_{2d} = C_{2,d}\sin\theta_2 + C'_{2,d}$

$\sin\theta_2' - T_{2,d}' = 4371.0$ kN.

The computed results show that: $F_{1d} + F_{2d} = N_{1d} + N_{2d} = 11000$ kN, indicating that the strut-and-tie model can satisfy the condition for equilibrium of forces. Finally, the reinforcement in the bottom of the pile cap is calculated according to the tensile force in ties $T_d = \max(T_{1,d}, T_{2,d}) =$ 4890 kN. Also, the width of diagonal struts may be determined by Figure 8.5.4, and the load carrying capacity of struts can be calculated.

In fact, if the pile reactions in two rows of piles are not equal, the strut-and-tie model for the pile cap may also be established by referring to deep beams subjected to two unequal vertical forces (Figure 8-15). Compared with the trapezoidal strut-and-tie model subjected to two equal vertical forces, to satisfy static equilibrium, the strut at the top of the pile cap has an angle of inclination to the horizontal.

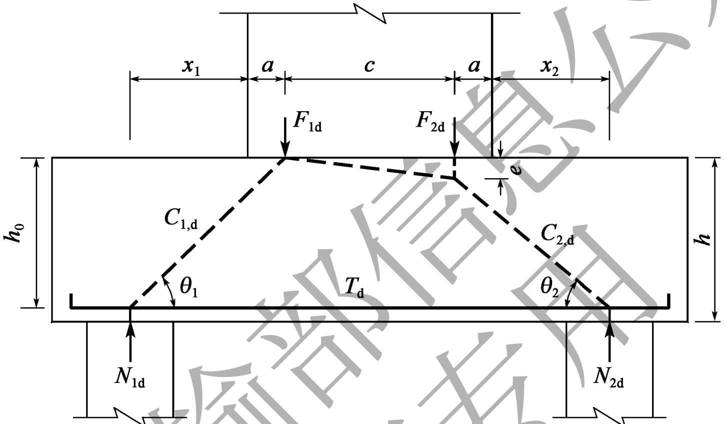


Figure 8-15　Strut-and-tie model with an inclined horizontal strut

In the above example, according to the geometric relationship and the condition of force equilibrium, we may obtain: $\theta_1 = \tan^{-1}\dfrac{h_0}{x_1 + a} = 50.82°$; $C_{1,d} = \dfrac{N_{1d}}{\sin\theta_1} = 7740.3$ kN; $T_d = \dfrac{N_{1d}}{\tan\theta_1} = 4890.0$ kN; $\theta_2 = \tan^{-1}\dfrac{N_{2d}}{T_d} = 45.67°$, $C_{2,d} = \sqrt{N_{2d}^2 + T_d^2} = 6993.7$ kN; $e = h_0 - (a + x_2)\tan\theta_2 = 314$ mm.

Comparing the calculation results using the above two strut-and-tie models, it can be seen that the design tensile forces of the ties in the bottom of pile caps are consistent, and the design compressive forces in the diagonal struts are relatively close.

Moreover, by reference to Clause 8.5.2 in *Code for Design of Concrete Structures* (GB 50010—2010), the requirement of the minimum reinforcement ratio for tension reinforcement in pile caps is reduced properly from 0.2% in the former edition of the *Specifications* to 0.15% in this revision.

8.5.5　The formula for punching shear resistance in Eq. (5.6.1) of the *Specifications* is:

$$\gamma_0 F_{ld} \leqslant 0.7\beta_h f_{td} U_m h_0 \tag{8-8}$$

For pile caps, $\beta_h = 0.85$.

This formula is applicable to the case that the angle between the inclined planes of the truncated pyramid of failure and the horizontal plane is 45°. As shown in Figure 8.5.5a), the angle between the inclined planes of the truncated pyramid of failure due to downward punching and the horizontal plane is not smaller than 45°, the punching shear resistance shall be multiplied by coefficients α_{px} and α_{py} to consider the influence induced by different angles. If the punching shear span-to-depth ratio λ_x or λ_y is 1, namely the angle is 45°, α_{px} or α_{py} is 1, so that the formula in this clause is consistent with Eq. (8.5.5-1) and Eq. (8-17). Similarly, as shown in Figure 8.5.5b), the angle between the inclined planes of the truncated pyramid of failure due to the upward punching and the horizontal plane is not smaller than 45°, the punching shear resistance shall be multiplied by the coefficients α'_{px} and α'_{py}. The coefficients for punching shear resistance were specified by referring to the relevant provisions in *Specification CECS 88:97*.

Item 1 in this clause is developed for the downward punching shear due to the compressive forces of columns, piers or abutments, and the truncated pyramid formed by the connection lines from the edges of the column, pier or abutment to the corresponding top edges of piles is adopted. The angle between the inclined planes of the truncated pyramid and the horizontal plane shall not be smaller than 45°. If it is smaller than 45°, lines shall be drawn down at an angle of 45° with the horizontal plane. At this point, the downward lines from the edges of columns, piers or abutments are likely to be unable to intersect with the top edges of piles but intersect with some points at the bottom edge of the pile cap, then piles are located beyond the truncated pyramid.

Item 2 in this clause is developed for the upward punching shear induced by the reaction forces of corner and edge piles beyond the truncated pyramid due to the downward punching shear of the column, pier or abutment, and the truncated pyramid formed by the connection lines from the edges of corner piles or edge piles to the edges of column, pier or abutment is adopted. The angle between the inclined planes of the truncated pyramid and the horizontal plane shall not be smaller than 45°. If it is smaller than 45°, lines shall be drawn up at an angle of 45° with the horizontal plane. At this point, the upward lines from the edges of piles are likely to be unable to intersect with the edges of the column, pier or abutment, but intersect with some points at the top surface of the pile cap. In Figure 8.5.5b), taking a length of h_0 away from the edges at two sides of the corner pile results in the punching shear angle of 45°.

8.6 Hinges

8.6.1 According to the Hertz formula, the maximum compressive stress of cylindrical hinges in line contact is:

$$\sigma_{max} = 0.564\sqrt{\frac{P}{l} \times \frac{\left(\frac{1}{r_1} - \frac{1}{r_2}\right)}{\frac{1-\upsilon_1^2}{E_1} + \frac{1-\upsilon_2^2}{E_2}}} \tag{8-9}$$

where:

P——pressure on the contact surface of a hinge;

E_1, E_2——modulus of elasticity of concrete for the upper and lower cylinders, respectively, $E_1 = E_2 = E_c$;

υ_1, υ_2——Poisson's ratio of concrete for the upper and lower cylinders, respectively, $\upsilon_1 = \upsilon_2 = \upsilon_c$.

Other symbols are explained in Clause 8.6.1.

According to Clause 3.1.8, $\upsilon_c = 0.2$, which is substituted into Eq. (8-9) to obtain:

$$\sigma_{max} = 0.407\sqrt{\frac{PE_c}{l}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)} \tag{8-10}$$

The average pressure on the cylindrical hinges is:

$$f_{cm} = \frac{\pi}{4}\sigma_{max} = 0.32\sqrt{\frac{PE_c}{l}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)} \tag{8-11}$$

When load-carrying members reach the ultimate limit state, their stresses may not be satisfied to the above Hertz formula. Therefore, the load-carrying capacity should still be determined by the elastic state and the method of allowable stress. According to *Specifications JTJ 023-85* and referring to *Specifications for Design of Highway Bridges and Culverts* issued in 1975, the allowable axial compressive stress of concrete is taken to be $0.75f_{cd}$, and the pressure of hinge is taken as $P = \gamma_0 F_{hd}/1.3$. Considering the increase coefficient β for local compression resistance of concrete, and its correction factor η_s (see Clause 5.7.1), $f_{cm} = 0.75\beta\eta_s f_{cd}$, in which f_{cd} is the design compressive strength of concrete, γ_0 is the importance factor of structure, and F_{hd} is the design pressure of hinges, substituting them into Eq. (8-11) and introducing "$\geqslant$" sign:

$$0.75\beta\eta_s f_{cd} \geqslant 0.32\sqrt{\frac{\gamma_0 F_{hd} E_c}{1.3l}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)}.$$

It is solved to obtain:

$$\gamma_0 F_{hd} \leqslant \frac{7.14\,(\beta\eta_s f_{cd})^2 l}{E_c\left(\frac{1}{r_1} - \frac{1}{r_2}\right)} \tag{8-12}$$

The width of the contact surface is:

$$b = \frac{P}{f_{cm} l} = \frac{\gamma_0 \frac{F_{hd}}{1.3}}{0.32l\sqrt{\frac{\gamma_0 F_{hd} E_c}{1.3l}\left(\frac{1}{r_1} - \frac{1}{r_2}\right)}} = 2.74\sqrt{\frac{\gamma_0 F_{hd}}{E_c\left(\frac{1}{r_1} - \frac{1}{r_2}\right)l}} \tag{8-13}$$

8.6.2 The formula for transverse tensile force in hinges is established with reference to *Handbook of Highway Design*, *Arch Bridges* (*Volume I*) published by China Communication Press in 1978 (in Chinese). According to the formula proposed by E. Morsch, the transverse tensile force in hinges is:

$$z = 1.5\,\frac{a-b}{4h}P \tag{8-14}$$

where:

z——transverse tensile force in hinge;

P——pressure on the contact surface of hinge;

a, b, h——see Figure 8.6.1.

The E. Morsch formula is applicable to hinges at elastic state. According to *Specifications JTJ 023-85* and referring to *Specifications for Design of Highway Bridges and Culverts* issued in 1975, the average allowable stress for reinforcement of each grade is taken to be $0.68f_{sd}$, the pressure of hinge is taken as $P = \gamma_0 F_{hd}/1.3$, in which f_{sd} is the design tensile strength of reinforcement, γ_0 is the importance factor of structure, and F_{hd} is the design pressure of hinges. Assuming the cross-sectional area of reinforcement to be A_s, substituting the above formulas into Eq. (8-14) and introducing "≥" sign, we obtain $0.68f_{sd}A_s \geq 1.5\dfrac{a-b}{4h} \times \dfrac{\gamma_0 F_{hd}}{1.3}$, then it is solved to give:

$$\gamma_0 F_{hd} \leq \frac{h}{0.425(a-b)} f_{sd} A_s \tag{8-15}$$

Based on the experiments and research information from Beijing Municipal Engineering Second Company and other three organizations in 1977, the transverse tensile force in hinges was about 0.25 ~ 0.3 times the pressure on the contact surface, and the lateral tensile force (in the length direction of the hinge) could be taken as 0.1 times the pressure on the contact surface. The ratio of transverse tensile force to lateral tensile force was 2.5 ~ 3.0. Therefore, after determining the cross-sectional area of the transverse reinforcement for hinges, the cross-sectional area of the lateral reinforcement may be taken as 0.4 times that of the transverse reinforcement.

The height of hinges is taken to be 0.8 ~ 1.25 times their width, which follows the provisions in Clause 329 of *Technical Specifications for Design of Railway, Highway, Urban Roads* (CH 200-62) in the Soviet Union. This value has been adopted by the highway and railway departments in China since 1950s.

8.7 Bearings

8.7.1 Bridge bearings are devices that link superstructures and substructures, and transmit the reactions of superstructures down to the substructure, and ultimately, to the ground. They are also key components to form the structural system, and call for proper selection and arrangement because inadequate design in type selection and arrangement of bearings will influence the behaviors of the structural system.

For example, spherical bearings can adapt to large rotation angles but have small rotational stiffness, so they are only used for the cases with large rotation angles or other special requirements; while pot rubber bearings are usually used in curved bridges to increase the rotational stiffness of the main girder.

In the structural design, the temperature during the installation of bearings and other factors like jacking at the construction stage shall be fully considered. Otherwise, "excessive" load and deformation in bearings are prone to occur after the bridge completion, which will result in excessive shear deformation and other hazards in bearings.

8.7.3 Laminated elastomeric bearings are analyzed for the serviceability limit state and the service stage. This is also applied in some other countries, for example, in Clause 14.7.5.3.2 of *US AASHTO LRFD Bridge Design Specifications*, the compressive resistance of bearings is checked by the average compressive stress in the bearings.

In Item 1, the effective compression area is taken as the area of laminated elastomeric bearings, namely, the compression area is limited to the partial area of the reinforcing steel plates provided for bearings.

For Δ_l in Eq. (8.7.3-2) ~ Eq. (8.7.35) of Item 2, the shear deformation of bearings due to the temperature change in superstructure as well as shrinkage and creep of concrete may be calculated directly by the length change of the superstructure. For the shear deformation due to the longitudinal force and the bearing force along the profile grade on the top surface of the bearing that is directly placed beneath the bottom of the beam, the shear forces distributed to the bearings need to be calculated, then the shear deformation is calculated. The relationship among characteristic longitudinal force F_k, total thickness of elastomeric layers in the bearing t_e, shear deformation due to longitudinal force Δ_l, shear modulus of bearing G_e, gross area of bearing surface A_g, and tangent of shear angle of bearing $\tan\alpha$ is $\tan\alpha = \dfrac{\Delta_l}{t_e} = \dfrac{F_k}{A_g G_e}$. As long as one of three values of Δ_l, F_k and $\tan\alpha$ is known, other values can be obtained, as shown in Figure 8-16.

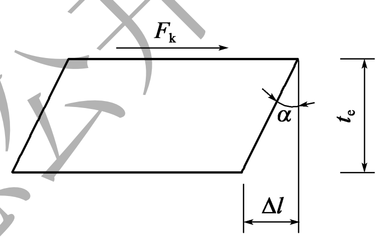


Figure 8-16 Shear deformation of elastomeric layer

The parameters of a single bearing can be used in the calculation.

For laminated elastomeric bearings used for slab superstructure, if the transverse direction of the bridge is set to be parallel to the cross slopes of piers and abutments or cap beams, the shear deformation in the transverse direction of the bridge induced by the component of compressive forces of supports parallel to the cross slopes shall be considered. Since bearings are arranged to be parallel to the cross slopes of piers and abutments or cap beams, there is no nonuniform compressive deformation in the transverse direction, and only the shear deformation occurs in the transverse direction, which is calculated in the same way for the longitudinal shear deformation.

The relationship between the total thickness t_e of a bearing and its circumferential dimension is specified in Eqs. (8.7.3-6) and (8.7.3-7) of Item 2. Considering that too thick bearings will affect the driving smoothness while too thin bearings will affect the shear deformation and rotation angle, a

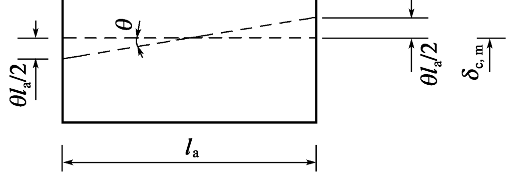


Figure 8-17 Compressive deformation of bearing

proper range for the thickness shall therefore be specified.

Item 3 concerns the calculation of the average vertical compressive deformation of elastomeric bearings. The elastic bulk modulus of elastomer was not considered in Clause 3.5.6 of *Specifications JTJ 023-85*. It is considered to be 2000 MPa in this revision with reference to the specifications and standards of the United States and Europe. In Eq. (8.7.3-9), $\delta_{c,m} \geqslant \theta \frac{l_a}{2}$ is specified to satisfy the requirement for rotation angle and to avoid separation of the bearing from the loading surface of the structure; while $\delta_{c,m} \leqslant 0.07t_e$ is specified to restrain the vertical compressive deformation so that the stability of bearings is not affected, see Figure 8-17.

In Eq. (8.7.3-10) of Item 4, reinforcing steel plates in elastomeric bearings are subjected to tensile force due to vertical and transverse loads, which is related to vertical load, shear force, and the thickness of elastomeric layers on and beneath the reinforcing steel plates. K_p in this equation is the correction factor for stress and is taken as 1.3 in the European standard, 2 in International Union of Railways, and 1.3 in the *Specifications*.

8.7.4 The checking for stability against sliding in this clause follows the provision in *Specifications JTJ 023-85*. The checked object is the bearing in service stage. The vehicular load (including impact force) for Eq. (8.7.4-5) in this clause is taken as 0.5 times its characteristic value, which is determined by trial computation to select the smaller reaction force.

8.7.5 The friction force in a PTFE slide bearing shall not result in a deformation larger than the allowable shear deformation of the interior elastomeric layers of the bearing. The item $G_e A_g \tan\alpha = F_k$ at the right-hand side of "$\leqslant$" sign in Eqs. (8.7.5-1) and (8.7.5-2) of this clause refers to the commentary on Clause 8.7.3, in which F_k is the allowable horizontal force for the limit of $\tan\alpha$. A_g is controlled to make F_k to be not smaller than the friction force. If A_g of a slide bearing is small and the provision in this clause can not be satisfied, excessive shear deformation will occur in interior layers for the bearing under a large friction force.

8.8 Deck expansion joints

8.8.1 From the built bridges, it is found that many expansion joints are damaged before the end of their design life. Besides materials, construction, installation and other factors that cause premature damage, the absence of relevant data or incomplete representation for construction and installation in construction drawings also affect the installation quality of expansion joints. Points for attention in their design, model selection and drawings are specified in this clause.

8.8.2 There are many factors that influence the movement range and the deformation of expansion joints during the operation. Besides those factors listed in this clause, other possible factors that affect the deformation of expansion joints include: rotation angle at the beam end, vertical and transverse deformation of expansion joints induced by vertical and transverse temperature gradients due to sunshine and sunset, transverse dislocation of expansion joints caused by vehicular centrifugal force on curved bridges, unequal deformation of joint gaps due to the unequal span length of beams at the end spans of some skewed bridges, etc. The amplification factor for the movement range β shall be multiplied in the calculation of the movement range. The value of β is determined to be 1.2 ~ 1.4 in the *Specifications*, and may be selected by the designers according to various potential unfavorable and favorable factors.

Movements of expansion joints caused by the shear deformation of laminated elastomeric bearings due to braking force are related to the direction of braking force, continuous length and arrangement of deck or beam, arrangement of laminated elastomeric bearings, etc. Hence, the most unfavorable closing and opening ranges for expansion joints need to be calculated according to the actual situations.

If laminated elastomeric bearings are provided on the tops of flexible bent piers, the stiffness of the piers and the laminated elastomeric bearings are connected in series. Their displacements induced by the braking force may be calculated by the serial stiffness, which may refer to *Case Studies for Analysis of Piers and Abutments in Simply-supported Beam Bridges with Continuous Deck* (*Revised Edition*) authored by Yuan Lunyi and published by China Communications Press in 1997.

9 Detailing Requirements

9.1 General

9.1.1 According to the investigation on the durability of concrete structures in China, and by reference to relevant provisions in *Code for Durability Design of Concrete Structures in Highway Engineering* as well as other corresponding specifications and standards in China and other countries, the following adjustments are made for the thickness of concrete cover:

(1) The requirement that the thickness of the concrete cover is not smaller than the diameter of the reinforcement is specified to ensure the anchorage of reinforcement in the bond layers of concrete.

(2) In terms of durability due to the carbonization of concrete, depassivation and rusting of reinforcement, the minimum thickness of concrete cover is no longer calculated based on the outer surface of longitudinal primary reinforcement, and it is changed to be calculated based on the outer surface of the extreme reinforcement (longitudinal primary reinforcement, stirrups or distribution reinforcement).

(3) According to the environment categories in Section 4.5, the requirements for the thickness of the concrete cover are specified in terms of the category of member and the design life.

9.1.2 The thickness of concrete cover should not be too large either. If the concrete cover is too thick, effective measures shall be taken to reduce its thickness or the thick concrete cover shall be tied to prevent its cracking, spalling and falling. Fiber-reinforced concrete or additional wire fabric is usually used for the thick concrete cover. To avoid wire fabrics to become the source of rusting, effective insulation and locating measures shall be taken for them, and then the thickness of the concrete cover over the wire fabric may be moderately reduced.

9.1.3 With reference to Section 18.11 in the former *German Standard Concrete and Reinforced Concrete Design and Construction DIN 1045: 1978* (hereafter referred to as *German Standard DIN 1045*), detailed provisions are made in this clause for wire fabric provided in concrete cover over bundled bars with an effective diameter larger than 36 mm. The provisions in this clause are not related to the thickness of concrete cover and are applicable to concrete cover with any thickness.

The limit values for the diameter and number of single bars in a bundled bar are specified by referring to US and German specifications.

9.1.4 The minimum development length l_a of reinforcement is calculated by Eq. (9-1):

$$l_a = f_{sk}\frac{\pi d^2}{4} \times \frac{1}{\pi d \tau} = \frac{f_{sk} d}{4\tau} \tag{9-1}$$

where:

f_{sk}——characteristic tensile strength of reinforcement;

d——diameter of reinforcement;

τ——ultimate anchorage bond stress between reinforcement and concrete, which is taken from *British Standard Steel, Concrete and Composite Bridges - Part 4: Code of Practice for Design of Concrete Bridges BS 5400: 1984* (hereafter referred to as *British Standard BS 5400*), as shown in Table 9-1.

Table 9-1 Ultimate anchorage bond stress between reinforcement and concrete (MPa)

Type of reinforcement	Concrete strength class			
	C25	C30	C35	≥ C40
Plain rebars in tension	1.4	1.6	1.8	1.9
Plain rebars in compression	1.7	1.9	2.1	2.1
Deformed rebars in tension	2.5	2.8	3.1	3.3
Deformed rebars in compression	3.1	3.5	3.8	4.1

For compression reinforcement, it is considered in US and German specifications that the hook at the end of a bar does not work for compression reinforcement, thus it behaves as a straight end of a bar. For tension reinforcement with a hook at the end, the development length is taken to be the value in accordance with 18.5.2.2 in *German Standard DIN 1045* multiplied by 0.7.

9.1.5, 9.1.6 Dimensions of hooks and stirrups are specified by reference to the current *Code for Quality Acceptance of Concrete Structure Construction* (*GB 50204*) (hereinafter referred to as *Code GB 50204*) and *Technical Specification for Construction of Highway Bridge and Culvert* (JTG/T F50).

9.1.7 Limit values of the diameter of reinforcement for lap splices are specified with reference to Clause 8.4.2 in the current *Code GB 50001*.

9.1.8 Provisions related to welded splices are made with reference to Clause 5.4.5 in the current *Code GB 50204*.

9.1.9 Provisions related to tension lap splices are made by reference to Clause 5.4.6 in the current *Code GB 50204*. Lap splices of bundled bars are specified by reference to 18.11.5 in *German Standard DIN 1045*. According to Clause 5.4.6 in *Code GB 50204* as well as US and German specifications, if there are difficulties in construction, the area percentage of lap splices is allowed to be larger than 25%, but the length of lap shall be increased, for which relevant provisions are made by reference to Table 8.4.4 in the current *Code GB 50010*.

9.1.12 This clause follows the provisions in the former edition of the *Specifications*.

(1) Sudden brittle crushing failure needs to be avoided for compression members, which requires minimum reinforcement ratio of the longitudinal reinforcement and appropriate arrangement of the stirrups. In addition, a part of the compressive force will transfer from the concrete to the reinforcement in the compression members due to the creep of concrete. Therefore, the reinforcement ratio for compression reinforcement should be somewhat high. The reinforcement ratio for compression members in US and German specifications is specified to be not smaller than 1% and 0.8%, respectively. However, if the cross-sectional area of concrete used in the design is larger than that required in practice, the reinforcement may be designed with the reinforcement ratio for the reduced cross-sectional area of concrete required in practice. It is specified in Article 5.8.4.1 of *British Standard BS 5400* that the cross-sectional area A_s of reinforcement in a column should not less than 1% of the cross-sectional area of the columns, or $0.15N/f_y$ (in which N is the ultimate axial load, and f_y is the yield strength of reinforcement. For a column with a diameter of 1.5 *m*, N = 25000 kN, f_y = 335 MPa, then A_s = 11194 mm^2, which is equivalent to 0.6%), whichever is the lesser. Due to some specified constraints, the minimum reinforcement ratios for compression members specified in the specifications of other countries are actually around 0.6%. Axially or eccentrically loaded compression members are all called as compression members or columns in specifications of other countries. They are deemed as the same type in the *Specifications*, and hence, the provisions for the minimum reinforcement ratio are also the same. In the *Specification JTJ 023-85*, the minimum reinforcement ratio for longitudinal reinforcement in compression members is specified to be 0.4%, which was increased to 0.5% in the former edition of the *Specifications* (*JTG D63-2007*) and it was increased to 0.6% for high-strength concrete with a strength class of C50 or above. This clause follows the provisions in the

former edition of the *Specifications* ~~are followed~~.

(2) The minimum reinforcement ratio for tension reinforcement in flexural members is determined based on the bending moment at which the concrete cracks, which is equal to the bending moment that a reinforced concrete beam of the same size can withstand. The purpose is to ensure that when cracks appear at the tension edge of the concrete, the beam will not suffer brittle failure due to insufficient reinforcement. Based on the above requirement, the minimum reinforcement percentage for tension reinforcement in reinforced concrete members is taken as $45 f_{td}/f_{sd}$.

9.1.13 Eq. (9.1.13) in this clause is the requirement for the minimum reinforcement ratio in prestressed concrete flexural members, which is similar to that for the above-mentioned reinforced concrete flexural members. It may be expressed as $M_{ud} \geqslant M_{cr}$.

9.2 Slabs

9.2.6 The arrangement of reinforcement in skewed slabs follows the provisions in the former edition of the *Specifications* that were made with reference to relevant specifications in other countries. The explanations are as follows:

(1) It is required in Article 5.8.10.1 of *British Standard BS 5400* that the primary reinforcement in skewed slabs should be aligned as close as the principal moment directions.

(2) It is explained in C9.7.1.3 of *US AASHTO LRFD Specifications* (*1994 Edition*) that, if the skew angle does not exceed 25°, the primary reinforcement in monolithic skewed slabs may be placed parallel to the longitudinal axis of the bridge, which only affects the stress as high as 10%. It is indicated in Article 5.8.10.2 of *British Standard BS 5400* that reinforcement (primary reinforcement and distribution reinforcement) in monolithic solid skewed slabs are usually placed parallel and perpendicular to the supports in combination with bands of reinforcement positioned parallel to the free edges.

(3) A single precast skewed slab is a narrow slab with a small width-to-span ratio. Its flexural behavior is close to that of a right slab with a span of skew length. It is indicated in Article 5.8.10.2 of *British Standard BS 5400* that, only for combinations of a large skew angle and a low ratio of skew width to skew span, it is preferable to arrange reinforcement in directions parallel and perpendicular to the free edges.

(4) According to studies, negative moments exist at the obtuse corners of simply-supported skewed slabs, and their directions are perpendicular to the bisectors of the obtuse angles; the end support reactions at the obtuse corners of simply-supported skewed slabs are several times larger than those in orthotropic slabs. Therefore, top reinforcement layers are placed perpendicular to the bisectors of the obtuse corners to sustain negative moments, and bottom reinforcement layers are placed parallel to the bisectors of the obtuse corners to sustain tensile force in the bottom of slabs.

9.3 Beams

9.3.1 End diaphragms must be provided in prefabricated T-beam bridges. According to design drawings in China, additional intermediate diaphragms need to be provided in prefabricated T-beam bridges. It is specified in Clause 8.12.2 of *US AASHTO Specifications 14th Edition* that one intermediate diaphragm is recommended at the point of maximum moment in T-beams with an effective span larger than 12 m. This provision is similar to that used in China.

End diaphragms must be provided in box girders. Intermediate diaphragms shall also be provided in curved box girders. It is specified in Clauses 8.12.1, 8.12.3, 9.10.3.3 and 9.10.3.5 of *US AASHTO Specifications 14th Edition* that intermediate diaphragms are required in curve box girders with an inner radius smaller than 240 m, the center spacing between them in reinforced concrete curved box girders is not larger than 12 mm, and that in prestressed concrete curved box girders needs to be determined according to the stress states of the structures.

9.3.2 For T- and I-beams, the depth of flanges at the connection with webs is specified to be no less than 1/12 of the beam depth in *Specifications JTJ 023-85*, which is smaller than that in the current design drawings. It is herein changed to be no less than 1/10 of the beam depth by referring to Clause 5.3.15 in *Code for Design on Railway Bridge and Culvert* (TBJ 2-85) (hereafter referred to as *Code TBJ 2-85*). When haunches are provided, their additional depth may be included.

The depth at the middle of the top and bottom flanges of box girders shall be no less than 1/30 of the clear span of the slabs and no less than 200 mm. By comparing the dimensions of 17 rectangular box girders in China, most of them are close to the above provisions.

It was specified in the former edition of the *Specifications* that the web width of T-beams, I-beams or box girders shall not be less than 140 mm, while it is required in this revision that the web width shall not be less than 160 mm. When the web width is varying, the length of the transition segment should not be less than 12 times the difference of the web width.

In order to avoid dramatic changes in sections, the bottom slope of haunches in continuous beams should not be larger than 1/6.

9.3.3 The minimum clear spacing of reinforcement follows the provisions of Clause 6.2.12 in *Specifications JTJ 023-85*.

9.3.4 If the span length of a cantilever slab is large, a positive moment is likely to be produced under the points of application of wheel loads. Its calculation method can refer to the commentary in Clause 4.2.5.

9.3.5 For reinforcement in bottom slabs of box girders, it was specified only for prestressed concrete structures in *Specifications JTJ 023-85* (Clause 6.2.36), that is, reinforcement with the cross-sectional area no less than 0.25% ~ 0.30% of that of the concrete section should be provided in the longitudinal and transverse directions of the bridge. By referring to the provisions of Clauses 8.17.2.3 and 9.2.4 in *US AASHTO Specifications 14 th Edition*, it was specified in the former edition of the *Specifications* that reinforcement with a cross-sectional area no less than 0.4% of the concrete section should be provided for reinforced concrete bridges, while reinforcement with the cross-sectional area no less than 0.3% should be provided for prestressed concrete bridges. These provisions are adopted in this revision.

9.3.6 Beyond the effective width of flanges in reinforced concrete T-beams or box girders, stresses are small under bending, and the contribution of primary reinforcement is insignificant. Hence, primary reinforcement is required to be arranged within the effective width. Beyond the effective width, construction reinforcement with 0.4% of the cross-sectional area may be provided, which is specified by referring to Clause 8.17.2.1 in *US AASHTO Specifications 14 th Edition*.

9.3.7 Longitudinal construction reinforcement is arranged on both sides of webs of beams, mainly to prevent cracking in webs, especially in tension zones of webs. It was specified in Clause 6.2.10 of *Specifications JTJ 023-85* that the cross-sectional area of longitudinal reinforcement should not be less than $(0.0005 \sim 0.0010)bh$ for cast-in-place concrete beams, and $(0.0015 \sim 0.0020)bh$ for beams with thin-walled webs by using welded reinforcement frames. It is reported that cracks often occur at two sides of webs, which is caused by less reinforcement on both sides. In this clause, the provisions are not specified separately for cast-in-place beams and beams with thin-walled webs by using welded reinforcement frames, it is specified that their total cross-sectional area of reinforcement in both sides should be $(0.0010 \sim 0.0020)bh$, in which the upper limit should be adopted for beams with thin-walled webs. Emphasis is put on the side reinforcement in specifications of other countries, for example, reinforcement with at least $0.0005bh_0$ shall be placed on each side as specified in Article 5.8.4.2 of *British Specification BS 5400*; for the beams with a depth larger than 610 mm, side reinforcement with 10% of cross-sectional area of tension reinforcement shall be placed in tension zones, and reinforcement of 264 mm^2 per meter of depth shall be placed in other regions, as specified in Clause 8.17.2.1.3 of *US AASHTO Specifications*

14 th Edition. In addition, for segments with large shear forces near supports and anchored segments with prestressing steel, longitudinal reinforcements are beneficial to prevent cracking, and their spacing should be reduced.

9.3.8 This clause is made by reference to Clause 6.1.5 in *Code GBJ 10-89* and Section 8.6.12 in *Concrete Structures* edited by WANG Yijun and published by China Architecture & Building Press in 1993 (in Chinese) (hereafter referred to as *Concrete Structures*). This clause is mainly to ensure the enough development length of the cut-off reinforcement and the flexural resistance on inclined sections.

9.3.9 Complicated stresses of the beam on the supports are induced by the local reaction forces of the supports acting on the bottom surfaces of the beam. To improve the flexural and shear resistances on inclined sections near supports and the resistance to tensile stress in the beam bottom, at least 1/5 primary reinforcement or two tension reinforcements shall be inserted into the beam sections on the supports. This clause follows the provision of Clause 6.2.13 in *Specifications JTJ 023-85*.

9.3.10 This clause is specified by following Clause 6.2.17 in *Specifications JTJ 023-85*, and referring to Clause 7.2.5 in *Code GBJ 10-89* and Section 8.7 in *Concrete Structures*. This clause aims to ensure the flexural resistance on the inclined section not to be less than that on the cross-section.

9.3.11 This clause follows the provision of Clause 6.2.14 in *Specifications JTJ 023-85*, but adds restrictions for the welded reinforcement frame on the number of reinforcement layers and the diameter of the reinforcement.

9.3.12 Before the presence of inclined cracks in concrete, principal tensile stresses are mainly resisted by concrete, and stresses in stirrups are very small. However, once cracks occur, the stresses in stirrups are increased abruptly. If the stirrups are less, they do not have enough capacity to resist the inclined tensile stresses transferred from concrete in the cracked sections. Therefore, it is necessary to specify the minimum reinforcement ratio of stirrups. With reference to *US Code ACI 318-05*, the minimum reinforcement ratio ρ_{sv} of stirrups is taken as $1.2\dfrac{0.35}{f_{sv}}$, and it shall be no less than 0.14% for HPB300 reinforcement and 0.11% for HRB400 reinforcement.

In addition to resisting shear on inclined section, if stirrups are also used to support compression reinforcement to avoid buckling, they must be of the closed type, and their arrangement shall be the same as that of the stirrups in compression members (see Clause 9.6.1 and its commentary in the *Specifications*). It was specified in *Specifications JTJ 023-85* that, when stirrups were hooked with tension reinforcement, the number of the tension reinforcements hooked on each side shall not be larger than five. This is not specified in this clause, stirrups are used for positioning, as long as the correct location of tension reinforcements can be maintained during installation, no restriction should be made on the number of longitudinal tension reinforcements to be hooked.

In the case of too large stirrup spacing, some inclined cracks are likely to occur between two stirrups, and the cracks do not intersect with the stirrups. It was specified in *Specifications JTJ 023-85* that the stirrup spacing shall not be larger than 3/4 of the beam depth and not be larger than 500 mm. It is now changed to be not larger than 1/2 of the beam depth and not be larger than 400 mm by referring to Clause 8.19.3 in *US AASHTO Specifications 14 th Edition* and *Code GB 50010—2002*. In order to increase the anchoring force of reinforcement, it is required to reduce the stirrup spacing within the range of lap splices for reinforcement. Due to the larger shear forces near the supports of beams, the stirrup spacing shall be reduced in the beam portions near the supports to prevent the development of cracks.

9.3.13 The influence of interaction between shear and torsion shall be considered in the calculation of the minimum reinforcement ratio for stirrups in members subjected to combined flexure, shear and torsion. According to Sections 4.5.2 and 8.4.2 in *Concrete Structures*, the minimum reinforcement ratio for pure torsion members is $0.055f_{cd}/f_{sv}$. According to Clause 9.3.12 in the *Specifications*, the minimum reinforcement ratio for shear stirrups in flexural members is $0.11\% \sim 0.14\%$ (represented by c in this clause). The minimum reinforcement ratio for stirrups in members subjected to combined flexure, shear and torsion is determined between the above two values by considering its linear relationship with the reduction coefficient β_t (Clause 5.5.4) for load-carrying capacity of concrete in members subjected to shear and torsion, and is calculated by the formula of $\left[(2\beta_t - 1)\left(0.055\frac{f_{cd}}{f_{sv}} - c\right) + c\right]$ in Item 3 of this clause. When $\beta_t = 1.0$ for pure torsion members, it equals to $0.055f_{cd}/f_{sv}$. When $\beta_t = 0.5$ for shear members, it equals to c.

The minimum reinforcement ratio for longitudinal reinforcement in members subjected to combined flexure, shear and torsion shall not be less than the sum of the minimum reinforcement ratios for longitudinal primary reinforcement in flexural members and in members subjected to shear and torsion. The minimum reinforcement ratio for longitudinal reinforcement in flexural members is taken in accordance with Clause 9.1.12. The minimum reinforcement ratio for longitudinal reinforcement in members subjected to shear and torsion is taken to be $0.08(2\beta_t - 1)f_{cd}/f_{sd}$ by referring to Sections 4.5.2 and 8.4.2 in *Concrete Structures* and Clause 7.2.10 in *Code GBJ 10-89*. For pure torsion members, $\beta_t = 1.0$, then the minimum reinforcement ratio is $0.08f_{cd}/f_{sd}$. For pure shear members, $\beta_t = 0.5$, then the minimum reinforcement ratio is zero.

9.3.14 For the curved soffit of a beam, longitudinal primary reinforcement in tension zones has the trend of downward deformation under the action of tensile force, which may result in spalling of concrete cover. Hence, the stirrup spacing in the curved portion needs to be reduced. The stress state of the tension reinforcement arranged at corners is similar to that of the above-mentioned tension reinforcement close to the concave surface of the beam soffit. At this point, each intersected tension reinforcement may be extended separately an additional development length from the intersection point.

Assuming that the radius of the circular curve of the beam soffit is r and the tension force in primary reinforcement of the curved portion is F, then the radial compressive force per unit arc length along the center is $u = F/r$, and the radial compressive force within the arc length s_v (stirrup spacing) is $F_c = us = (F/r)s_v$. If the cross-sectional area of primary reinforcement is A_s and the design tensile strength is f_{sd}, then, the tensile force in reinforcement is $F = f_{sd}A_s$. Substituting F into the formula of F_c obtains $F_c = f_{sd}A_s s_v/r$. Assuming the cross-sectional area of one leg of stirrups is A_{sv1}, the design tensile strength of the stirrups is f_{sv}, then the tensile resistance in double-leg stirrups is $F_r = 2f_{sv}/A_{sv1}$. It shall be balanced with the radial compressive force induced by the tensile force in the primary reinforcement hooked by the double-leg stirrups. Let $F_r = F_c$, it may give $A_{sv1} = (f_{sv}/f_{sd}) \times (A_s s_v/2r) = m(A_s s_v/2r)$, in which $m = f_{sv}/f_{sd}$, then Eq. (9.3.14-1) is obtained. Eq. (9.3.14-2) is for a circular curve and may be used approximately for a non-circular curve.

9.3.15 Loop splicing has been used for the transverse connection between deck slabs on T-beams for several years and has also been adopted in general design drawings. All reinforcements of loop splices have 100% lap splices at the same section, the lap length (the distance between the ends of two semi-circular loops) is only around 20 times the diameter of reinforcement, and there are two very close lap splices in the connecting section between flanges of two adjacent beams. Measures are required to strengthen the splices, for example, longitudinal reinforcement of full length may be provided within semi-circular loops. In addition, the thickness of deck slabs within the connecting section shall meet the requirements for the thickness of concrete cover and the diameter of circular loops, and should not be too small in size.

9.3.16 This clause follows the provision of Clause 6.2.13 in *Specifications JTJ 023-85*, and the minimum thickness of cast-in-place slabs of composite beams is specified by making reference to the structures in the completed bridges. By referring to Clause 7.5.17 in *Code GBJ 10-89*, the top surfaces of the precast beams shall be roughened with unevenness no less than 6 mm.

9.3.17 For shear resistance on interfaces of composite beams, stirrups have no contribution when their reinforcement ratio for the interfaces is lower than 0.10% (see Section 9.2.3 in *Concrete Structures*). It is specified in Article 7.4.2.3 of *British Standard BS 5400* that anchored reinforcement of 0.15% contact area shall be arranged between the beam and the slab in composite beams, and the spacing of this reinforcement should not exceed the lesser of four times the thickness of slab or 600 mm.

9.4 Prestressed Concrete Superstructures

9.4.1 Anchorage devices are centralized at the ends of prestressed beams, which results in complicated stresses, hence, small stirrup spacing is required. Horseshoe-shaped portions of T-beams are equivalent to compression members during jacking, stirrups shall be provided additionally in the horseshoe-shaped portions because they are arranged with closely-spaced prestressing steel. Since there are large shear forces and tensile stresses in anchorage zones near the centers of beam supports, small stirrup spacing shall be adopted.

Stresses in prestressed concrete beam ends during jacking and other construction phases are very complicated, the diameter and spacing of stirrups shall simultaneously conform to the provisions for shear resistance on inclined section (Clause 5.2.9) and for reinforcement against bursting force ahead of anchorage devices (Clauses 8.2.1 and 8.2.2). In addition, it is specified in this clause that the stirrup spacing shall not be larger than 120 mm within the extent of the beam depth at the beam ends. It is specified in *Specifications JTG D 62—2004* that the stirrup spacing at the beam ends shall not be larger than 100 mm, and it is increased by 20 mm in this clause, mainly considering that the reinforcement at the beam ends is generally closely spaced. Based on the engineering practices, properly increased stirrup spacing is helpful for concrete casting and compaction by vibration.

9.4.3 This clause is adjusted in accordance with the provision of Clause 3.2.2. The cohesion between plain wires and concrete is poor. The failures of members tested by China Academy of Building Research were all caused by the slip of wires without the contribution from the wire strength. The failure bending moment of flexural members with plain wires of 5 mm diameter and 1100 MPa characteristic strength only reached 40% ~ 50% of the design value; while the failure bending moment of flexural members with plain wires of 3 mm diameter and 1600 MPa characteristic strength reached 90% of the design value. Therefore, indentation on plain wires shall be provided to improve the cohesion.

9.4.4 This clause follows the provision in the previous *Specifications*, and is adjusted in accordance with the provision of Clause 3.2.2 correspondingly.

9.4.5 In order to avoid concrete failure at the ends of members due to the impact caused by the releasing of prestressing steel, the concrete around the ends of reinforcement shall be strengthened locally. This clause follows the provision in the former edition of the *Specifications*.

9.4.6 Anchor plates with trumpet pipes are used in current anchorage devices generally.

Therefore, the provision on the required thickness of bearing plates is removed.

9.4.7 This clause follows the provision in the former edition of the *Specification*. When prestressing steel is mostly placed on the bottom or both the upper and bottom of the end sections, longitudinal cracks will develop at the ends of the members due to the tensile stresses at the beam ends perpendicular to the direction of the beam length caused by the prestress force. Therefore, it is required that a part of prestressing steel after bending up is placed along the depth of the ends as evenly as possible. According to testing data from other countries, tensile stresses ahead of anchorage devices are generally distributed in the beam segment with a length of 3/4 beam depth from the beam end. Besides, longitudinal cracks along ducts at the beam ends were found in the investigation from the railway department. Therefore, small stirrup spacing at the beam ends and additional wire fabric are required. In addition, a proper increase of the web width at the beam ends is also an effective way to prevent longitudinal cracks at the beam ends.

9.4.8 For curved ducts, such as ducts for vertical curved reinforcement in straight beams, ducts for reinforcement in curved beams and ducts for reinforcement in thickened blisters, their inner side of the curvature plane is squeezed by curved tendons, and concrete covers in and out of the curvature plane are subjected to shear, hence, the concrete covers at the bottom and sides of beams need to be increased or reinforced by tie reinforcement. By referring to Clauses 5.10.4.3.1 and 5.10.4.3.2 in *US AASHTO LRFD Specifications*, in-plane shear force F_{in}, out-of-plane shear force F_{out} and shear resistance V_c (all of N/mm) are:

$$F_{in} = \frac{P}{r} \leqslant V_c \tag{9-2}$$

$$F_{out} = \frac{P}{\pi r} \leqslant V_c \tag{9-3}$$

$$V_c = 0.33\varphi d_c \sqrt{f'_{ci}} \tag{9-4}$$

where:

ϕ——factor for shear resistance of material, $\phi = 0.9$;

P——jacking force for prestressing tendon (N);

f'_{ci}——compressive strength of concrete cylinder (ϕ150 mm ×300 mm) at the time of force transmission and anchorage of prestressing tendons (MPa);

d_c——distance from the center of the tendon duct to the surface of the concrete cover in the directions in and out of the curvature plane (mm).

The ratio of compressive strength of 150 mm cubes to that of ϕ150 mm × 300 mm cylinders made of the same concrete is 1/0.8. If the cube strength of the concrete at the time of force transmission and anchoring is f'_{cu}, then $0.8f'_{cu} = f'_{ci}$ shall be substituted into Eq. (9-4) to obtain the shear resistance $V_c = 0.33 \times 0.9 d_c \sqrt{0.8f'_{cu}} = 0.266 d_c \sqrt{f'_{cu}}$, where d_c is the thickness of concrete cover c_{in} or c_{out} plus the outer radius of duct $d_s/2$. By substituting it into Eq. (9-4) and introducing Eqs. (9-2) and (9-3), Eqs. (9.4.8-1) and (9.4.8-3) in this clause are obtained.

The jacking force for prestressing tendons is P, then the radial compressive force per unit arc length of tendon is $u = P/r$, and the radial compressive force for the arc length s_v (stirrup spacing) is $F_c = u\, s_v = (P/r)\, s_v$. If the cross-sectional area of one leg of stirrups is A_{sv1}, the tensile resistance of double leg stirrups is $F_r = 2f_{sd}A_{sv1}$. Let $F_c = F_r$, Eq. (9.4.8-2) is obtained.

9.4.10 This clause follows the provision in the former edition of the *Specifications*. If the radius of curved tendons is too small, a large friction force of ducts and radial compressive force will be induced during jacking. For special ducts and prestressing tendons, for example, semi-circular prestressing tendons with a radius of about 1.5 mm used for hooping in towers of cable-stayed bridges, they are not restricted by this provision as special measures have been taken.

9.4.11 By referring to the VSL product standard, the minimum tangent length of prestressing tendons is supplemented in the *Specifications*. Due to the detailing and stress requirements, if wedge-type anchorage devices for steel wires or strands are used, the curved parts of tendons are not allowed to enter into the anchor segment, hence, the tendons ahead of the anchorage device shall have a certain straight extension. The minimum straight extension L_{min} under different minimum breaking loads is shown in Figure 9-1, which provides a reference for designers.

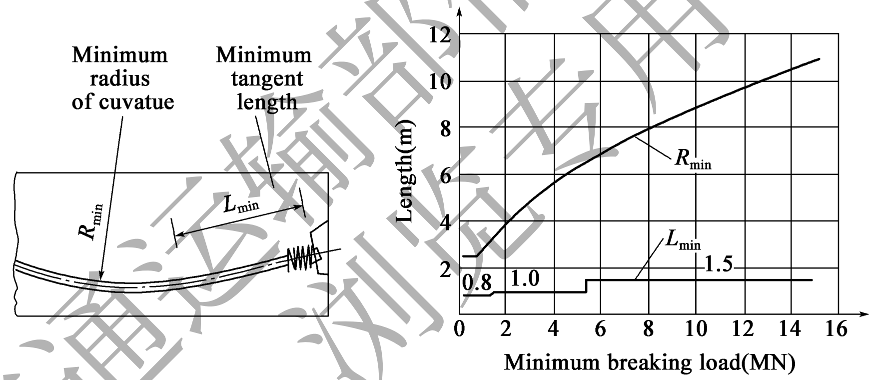


Figure 9-1 Relationship between minimum breaking load and R_{min}, L_{min}

9.4.14 This clause follows the provision in the former edition of the *Specifications*. Besides the deviation from the alignment, the prestress loss due to friction is mainly induced by the friction between the curved duct and the prestressing tendon. To reduce the loss due to friction, it is required to reduce continuous curves along the entire length of the prestressing tendon and to increase the radius of the curvature. Generally, two ways are used to achieve these objectives during the arrangement of prestressing tendons. One way is to jack, anchor and lengthen the tendons segment by segment, and repeat again, in which the tendons can be directly lengthened by couplers, and can also be segmentally anchored and spliced. The other way is to use varying beam depth to reduce the curvature of the curved tendons. These two ways can also be used in combination with a more proper arrangement of prestressing tendons.

9.4.15 This clause follows the provision in the former edition of the *Specifications*. The number of prestressing tendons shall not be suddenly increased or decreased, to prevent excessive variation of shear stress or principal tensile stress in webs due to sudden change of prestressing force, and also to prevent weakening of sections due to too many anchorages arranged within one section.

Within the 1/4 ~ 1/3 section of the span in a continuous beam, positive and negative moments occur alternatively under live loads. During the launching stage in incremental launching construction of continuous beams, positive and negative moments occur alternatively at most sections. In the above cases, prestressing tendons should be scattered in the upper and bottom areas of webs and their adjacent flanges, so that both positive and negative moments can be resisted by prestressing tendons.

Continuous beams at interior supports are subjected to concentrated reaction forces, and their stress states are complicated. Longitudinal horizontal tensile stresses are induced by the reaction forces at the bottom of their webs. Therefore, reinforcing steel along the bridge direction shall be arranged in their webs and bottom flanges near the interior supports.

9.4.16 This clause follows the provision in the former edition of the *Specifications*. Due to the precompression, tensile stress exists in the concrete surface layer around anchorage devices. Concrete within the angle of spreading ahead of anchorages is subjected to a compressive force, but within this range along the direction of force transmission there is a tensile force zone in the shape of a date pit. This indicates that stresses in the surface layer around the anchorage and concrete ahead of the anchorage are complicated. Therefore, anchorage devices should not be arranged at the tension zones of members and should be arranged at the centroid location of section or compression zones.

When the prestressing steel is extended beyond flanges for anchoring, the arrangement of tie reinforcement in anchor blisters may refer to Clause 9.4.8 in the *Specifications*.

9.4.17 Precast segmental structures have been widely used for bridges crossing rivers or seas and for bridges in urban road systems (as listed in Table 9-2). They have good overall benefits and can fulfill the development trend of industrialization, upsizing, mechanization and standardization for modern bridges. According to the structural characteristics of the built precast segmental bridges, the detailing requirements are specified in this clause.

Table 9-2 List of some precast segmental bridges in China

No.	Bridge	Completed year	Span (m)	No.	Bridge	Completed year	Span (m)
1	Shanghai Liuhe Bridge	2000	42	6	Approach of Jiangsu Chongqi Bridge	2011	50
2	Shanghai Humin viaduct	2002	35	7	Approach of Nanjing No. 4 Bridge	2012	50

continued

No.	Bridge	Completed year	Span(m)	No.	Bridge	Completed year	Span(m)
3	Approach of Sutong Bridge	2008	75	8	Approach of Xiazhang Bridge	2013	70
4	Xiamen Jimei Bridge	2008	100	9	Approach of Jiashao Bridge	2013	70
5	Approach of Shanghai Yangtze River Bridge	2009	60				

(1) Structures with both internal and external tendons and epoxy joints have been mostly used for building precast segmental box girder bridges. Epoxy joints are pressed with a compressive stress of 0.3 ~ 0.5 MPa. Wet joints made of fine aggregate concrete are generally provided at gaps upon the closure of box girders or compensation gaps for assembly error.

(2) Basic configurations and functions of composite shear keys are as follows:

① Shear key in the web: which consists of several rectangular keys (grooves) and resists the shear force of joint section at normal service stage.

② Shear key in top flange: which consists of several long and narrow keys (grooves) and is used for positioning during assembly of segments.

③ Shear key in bottom flange: which consists of several long and narrow keys (grooves) and is used for positioning during assembly of segments.

④ Shear key in haunch: which is arranged at the interface between the web and top flange (bottom flange) and is used for positioning during assembly of segments.

9.4.18 For end anchorage zones of post-tensioned members, tension reinforcement shall be provided in the corresponding regions to resist bursting force, spalling force and longitudinal edge tension force, as shown in Figure 9-2.

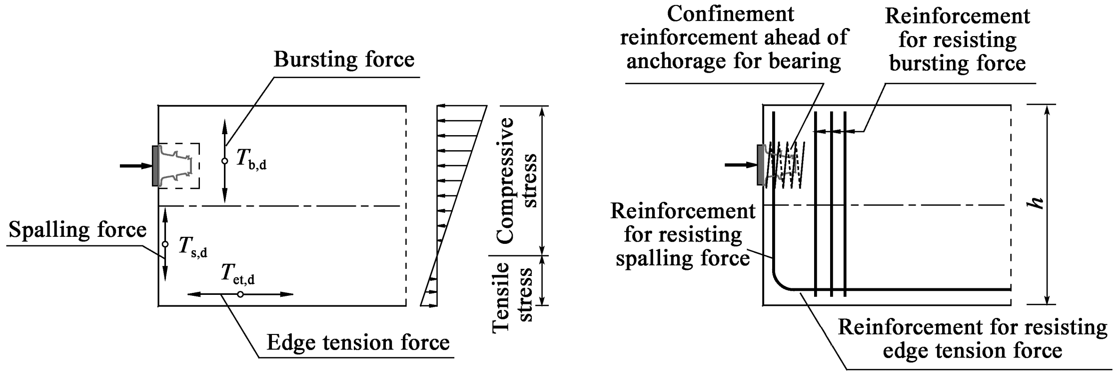
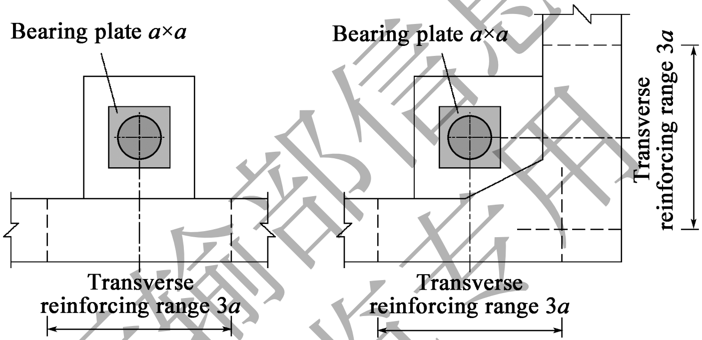


Figure 9-2 Local tensile forces and reinforcement in end anchorage zone

9.4.19 Requirements for elevation dimensions of prestressed anchor blisters are specified in this clause. Based on the width of blisters, they can be divided into independent blisters, corner blisters and blisters attached to whole wall plates. To ensure smooth and reliable force transmission, corner blisters at the junctions between flanges and webs are preferred in general.

9.4.20 This clause gives the provisions on detailing design of reinforcement after the quantity of local reinforcement in blisters and their attached wall plates (i. e., top flanges, bottom flanges or webs) have been calculated in accordance with Clause 8.2.6 and Appendix B in the *Specifications*. This clause is made by reference to engineering practices in China, *US AASHTO Highway Bridge Design Specifications*, *Japanese Concrete Bridge Design Specifications*, *VSL Post-tensioning Design Manual* and relevant research outcomes of the University of Texas at Austin in US.

In wall plates to which blisters are attached, transverse reinforcing ranges for longitudinal reinforcement to resist tension ahead of anchorages and local lateral flexure of edges are shown in Figure 9-3.



a)Independent blister b)Corner blister

Figure 9-3 Transverse reinforcing range for longitudinal reinforcement in wall plates of blisters

In addition, apart from resisting the five tension effects in blisters shown in Figure 8.2.6, transverse reinforcement should be provided in wall plates in front of blisters to resist bursting forces (Figure 9-4), considering that anchor forces continue to spread along the width of wall plates after transmitting to wall plates. Longitudinal cracks at this location have been found in both laboratory tests and existing projects.

9.4.24 Deviator structures are explained as follows:

(1) Block type deviators: which are used in the case with a few deviation tendons, or for positioning of tendons between deviators.

(2) Transverse rib type deviators: which are used in the case with large transverse deviation force, or for positioning of tendons between deviators.

(3) Vertical rib type deviators: which are used in the case with large vertical deviation force.

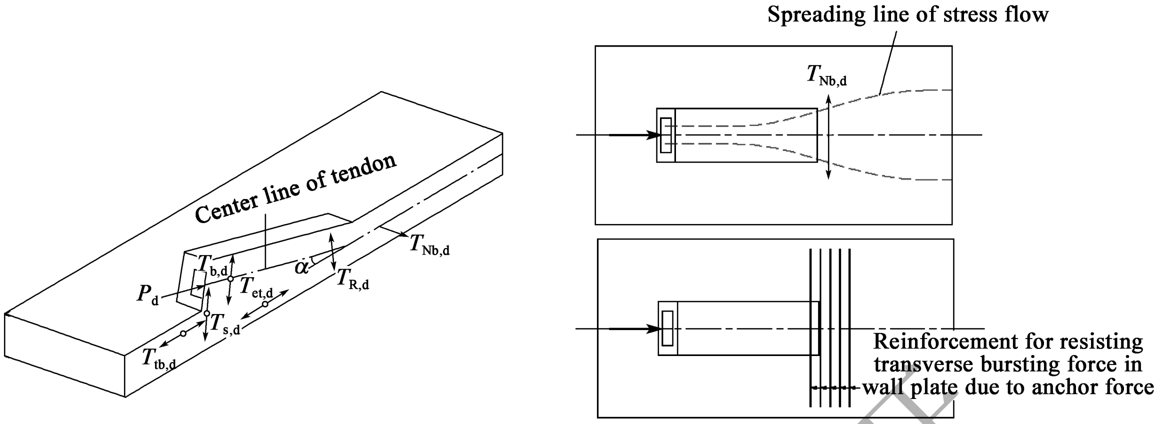
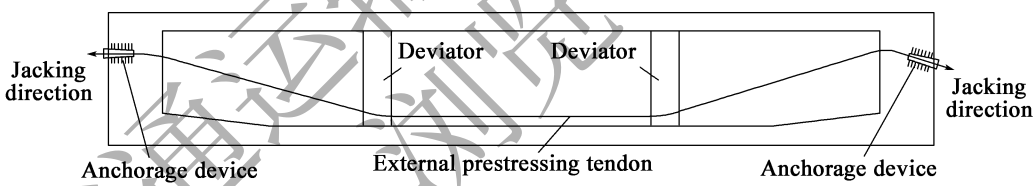


Figure 9-4　Bursting effect due to anchor force and reinforcement along the width of wall plate

(4) Crossbeam type deviators: which are used at the locations of crossbeams.

9.4.25 ~ 9.4.27　These clauses were developed by referring to engineering experiences of existing precast segmental bridges. When the free length of the external prestressing tendons among anchoring location, deviation structure, positioning structure and seismic device is longer than 8 m, the final length shall be determined by calculation, and its influence on structural stresses shall be considered.

External prestressing tendons should be deviated properly after entering anchorages (Figure 9-5) to avoid direct transmission of the stress fluctuation in prestressing steel to wedges in anchorages.



igure 9-5　Proper deviation of the external prestressing tendon in anchorage structure

9.5　Arch Bridges

9.5.1　Based on the statistical analysis from *Study on Design Optimization of SRC Arch Bridges and CFST Arch Bridges* authored by XU Fengyun, among 44 completed reinforced concrete arch bridges with a span of 100 m and above, there are four bridges with a rise-to-span ratio of 1/4, four bridges with 1/5, eleven bridges with 1/6, seven bridges with 1/7, thirteen bridges with 1/8, and one bridge with 1/10. Among 26 steel reinforced concrete (SRC) arch bridges (Melan arches) and concrete-filled steel tube (CFST) arch bridges with a span of 66 ~ 313 m in design, under construction and completed, there are eight bridges with a rise-to-span ratio of 1/4, ten bridges with 1/5, seven bridges with 1/6, and one bridge with 1/8. Among 11 reinforced concrete truss-type composite arch bridges with a span of 100 ~ 330 m under construction or completed, there are three bridges with a rise-to-span ratio of 1/6, one bridge with 1/7, six bridges with 1/8, and one bridge with 1/9. In this revision, the common rise-to-span ratio of reinforced concrete arch bridges between 1/5 and 1/8 is adjusted to be between 1/4.5 and 1/8.

The span of spandrel structures in open-spandrel arch bridges is usually taken as 1/8 to 1/15 of the bridge span, so that the main arches are stressed uniformly. But considering the harmony for aesthetics of the bridges, a bit larger value may be used properly.

As for the axes of catenary arches, the arch axis coefficient tends to be smaller with the light-weight trend and the decreasing rise-to-span ratio of spandrel structures. According to the analysis in the abovementioned *Study on Design Optimization of SRC Arch Bridges and CEST Arch Bridges*, among twelve arch bridges with a span from 100 m to 312 m, the arch axis coefficient m is from 2.24 to 1.347. For composite truss bridges, smaller m value or parabola may be taken for their bottom chords.

9.5.2 Wall piers or bent piers and simply-supported slab or beam structures are usually adopted for spandrel structures of open-spandrel arch bridges. Elastomeric bearings may be used. If continuous deck systems are used, sliding bearings and expansion joints shall be provided at the top surfaces of pier columns or abutments of the main arches. Near the arch crown, due to short wall piers or bent piers with large rigidity against thrust, sliding bearings and expansion joints should also be provided.

9.5.4 Lateral bracings and arch ribs form truss structures in bridge transverse direction, which increase the lateral stiffness of the arch structures. Lateral bracings under spandrel columns are also helpful for the transverse distribution of loads.

9.5.5 Lateral bracing systems are essential for the integration and stability of half-through arches and tied arches. During concreting arch ribs, the in-place lateral bracing systems may be used to improve construction stability. Lateral stud bracings are provided at the arch crown, lateral stud bracings or K-bracings are provided at the location near the zeros of bending moment influence lines at both sides of the arch crown. Crossbeams are provided for the arch ribs at the bridge deck level, and cross bracings are provided for the arch ribs beneath the bridge deck. For example, in the Yongjiang Bridge in Yongning, Guangxi, with a main span of 312 m, one lateral bracing is provided at its arch crown, two K-bracings are provided at the location near the zeros of positive bending moment influence line near the arch crown, two crossbeams are provided for the arch ribs at the bridge deck level, two cross bracings are provided for the arch ribs beneath the bridge deck, and four lateral bracings are provided for the arch ribs above the bridge deck.

9.5.6 Truss arch bridges have high end structures, and the end joints of top chords cannot connect with the top of piers or abutments, therefore vertical cross bracings need to be provided to keep out-of-plane stability. To improve the transverse horizontal stiffness of truss arches, horizontal cross bracings shall also be provided between end sections. Proper vertical and horizontal cross bracings shall also be provided between other sections.

9.5.7 Composite truss arch bridge is a new type of arch bridge constructed in Guizhou in 1982. The Jiangjiehe Bridge in Guizhou with a main span of 330 m is the longest composite truss arch bridge .

A composite truss arch bridge is a derivative developed by the synthesis of a truss arch bridge and a T-shaped rigid frame truss bridge (with suspended spans). In composite truss arches, the top and bottom end chords of the truss arch are rigidly connected with abutments; at the two ends of the mid span segment which has a length of 0.5 ~ 0.6 times the span, the top chords are interrupted, while the bottom chords of the arch remain continuous in the full span. In this way, a composite structure system is formed, in which the top chord works as a beam and the bottom chord works as an arch. The arch axis (of the bottom chord) is generally a parabola.

Members in composite truss arch bridges with large spans have box sections, for example, all the top and bottom chords in the Jiangjiehe Bridge with a main span of 330 m and Jingnan Bridge in Guangxi with a main span of 160 m are three-cell box beams, diagonal and vertical web members are two separated box members connected with lateral bracings.

At the ends of a composite truss arch bridge, top chords are rigidly connected with abutments, hence, short end spans shall be designed behind the arch springings. When the ratio of the length of the end span to that of the main span is close to 0.5, the end span has a long lever arm of force and small reaction at the ends, which are favorable for stresses at the construction stage and unfavorable for stresses during the service life. If the ratio is close to 0.2, the above situations are reversed. In the Jiangjiehe Bridge, the span ratio of end span to the main span is 0.24 for the left bank and 0.16 for the right bank. In the Jingnan Bridge in Guangxi, the span ratio of end span to main span is 0.24 for the left bank and 0.31 for the right bank. The above length of end spans includes the length of the abutment.

As for the interrupt positions of top chords in composite truss arch bridges, according to the analyses on the Daozhen Bridge and the Jianhe Bridge in Guizhou, each one at two ends of the mid-span segment should be 0.3 times the main span distant from the arch crown. It is 0.3 times for the Jingnan Bridge in Guangxi, and 0.25 times for the Jiangjiehe Bridge.

9.5.8 This clause follows some contents of Clause 6.3.3 in *Specifications JTJ 023-85*. To achieve a certain stiffness, the short side of the cross-section is not be smaller than 1/15 of the length for lateral stud bracings, K-bracings and cross bracings in arch bridges. At the intersections between arch ribs and lateral stud bracings, K-bracings or cross bracings, due to dramatic section change and large local stresses, chamfers shall be provided for smooth transition.

9.5.9 Joint blocks are formed by the connection of truss members. According to the photoelastic model test of joints for the Jiangjiehe Bridge, local stress concentration occurs at the joints of members. To reduce the stress concentration in the joints, transition lines shall be provided at the edges of the joint block, namely, between adjacent edges of members. Enveloping reinforcement arranged in joint blocks is helpful to reduce stress concentration and prevent splitting between adjacent members and pulling out of tension members from joint blocks.

9.5.11 This clause follows the provisions of Clause 6.3.2 in *Specifications JTJ 023-85*. For truss arch bridges or rigid frame arch bridges in soft soil areas or cold regions, due to ground settlement or temperature drop, the stresses in arch springings are unfavorable. Therefore, reinforcements shall be added properly in their bottom chords.

9.5.12 This clause is consistent with Clause 5.2.3 in *Code for Design of Highway Masonry Bridges and Culverts* (JTG D61). Piers of multi-span arch bridges are mostly flexible, and the interaction effect among arches is remarkable. Therefore, robust pier is required to be arranged for every 3 ~ 5 spans, or other measures that can resist thrust from the arch at one side of the pier.

9.5.13 Continuous longitudinal beams provided in the deck system can increase the integrity of the deck system, and prevent the progressive collapse of the deck system due to the failure of local hangers.

9.6 Columns, Piers and Abutments, Pile Caps

9.6.1 This clause is compatible with the general axially loaded compression members specified in Clause 5.3.1 of the *Specifications*, and differs from the axially loaded members with confinement reinforcement of closely spaced spiral circular reinforcement or welded circular stirrups specified in Clause 5.3.2 of the *Specifications*. The detailing requirements for the latter refer to the provisions in Clause 9.6.2 of the *Specifications*.

Stirrups mainly rely on their bends (the bend angle no larger than 135°) to restrain longitudinal reinforcement. The farther the longitudinal reinforcement is away from the bends, the weaker the restraint they receive from the stirrups. In addition to the provision in the former edition of the *Specifications*, considering the large dimensions of bridge members, it is specified in the *Specifications* that besides composite stirrups, tie reinforcement may also be provided for the longitudinal reinforcement located beyond the specified range, as shown in Figure 9.6.1.

9.6.2 This clause is compatible with the compression members arranged with spiral or welded circular confinement reinforcement specified in Clause 5.3.2 of the *Specifications*. Compared with common compression members in Clause 9.6.1 of the *Specifications*, they have higher load-carrying capacity for two reasons: (1) they have higher cross-sectional strength due to close stirrup spacing, and (2) they have higher ultimate load-carrying capacity beacuase the stability factor needs not to be considered in the calculation due to their limited slenderness ratio. This clause is consistent with the provisions in the former edition of the *Specifications*.

9.6.3 Eccentrically loaded compression members have the same detailing requirements as the axially loaded compression members. In specifications of other countries, both axially and eccentrically loaded compression members are referred to as compression members or columns, and they have the same detailing requirements. Primary reinforcements are required to be arranged in the bending direction of the eccentrically loaded compression members, and construction reinforcement shall be arranged in the direction without bending. Piers or abutments in highway bridges are usually subjected to eccentric loads in two directions, hence primary reinforcements are arranged in both directions. This clause is consistent with the provisions in the former edition of the *Specifications*.

9.6.4 Wire fabric in surfaces is specified by referring to Clause 8.20 in *US AASHTO Specifications 14 th Edition*, in which it is required to have an area of 264 mm^2 per meter. It is taken to be 250 mm^2 in the *Specifications*, which is equivalent to providing five reinforcing bars with a diameter of 8 mm per meter. This clause is consistent with the provisions in the former edition of the *Specifications*.

9.6.5 Cap beams generally have an effective span of 2.5 ~ 7.0 m with a span-to-depth ratio l/h of 3 ~ 5 (the span-to-depth ratio decreases with the increase of the span). The cap beams are considered as deep flexural members but not deep beams (deep beam is defined as follows: for simply-supported beam, $l/h \leqslant 2$; for continuous deep beam, $l/h \leqslant 2.5$). Since the span-to-depth ratio l/h is smaller than conventional beams, and the cap beams are rigidly connected with pier columns, the expansion and contraction of the (cap) beams are restrained. Therefore, a certain amount of construction reinforcement should be provided on their side faces. This clause is developed by referring to commonly used design drawings. Cap beams are subjected to concentrated loads, and the shear forces of the beams are quite large, hence high strength class is required for the concrete. This clause is consistent with the provisions in the former edition of the *Specifications*.

9.6.7 This clause is developed by referring to Clauses 9.3.4 and 9.3.5 in *Code GB 50010*.

9.6.8 This clause is developed by referring to Clause 9.3.6 in *Code GB 50010*.

9.6.9 Resistance against overturning of continuous highway box girder bridges is closely related to the structural forms of substructures. According to the investigations on structural types of highway box girder bridges, parametric analysis on resistance against overturning and bridge accidents, the structural types of substructures of highway box girder bridges are specified in this clause, to which attention shall be paid in structural design.

(1) Overturning resistance of box girders is closely related to the transverse spacing of bearings and the torsion spans of box girders. In the overturned bridges, single-column piers with one bearing were successively used in the interior supports (Figure 4-1), thus the box girders had large torsion spans, and their resistance against overturning could not be guaranteed effectively. Typical structures of single-column piers in box girder ramp bridges on highways are shown in Figure 9-6. Rigid connection systems of piers and girders are applicable to high piers, in which the requirement for resistance against overturning of box girders is converted into that for the load-carrying capacity of members. Single-column piers with double bearings reduce the torsion span of box girders and can significantly improve the resistance against overturning of the box girders.

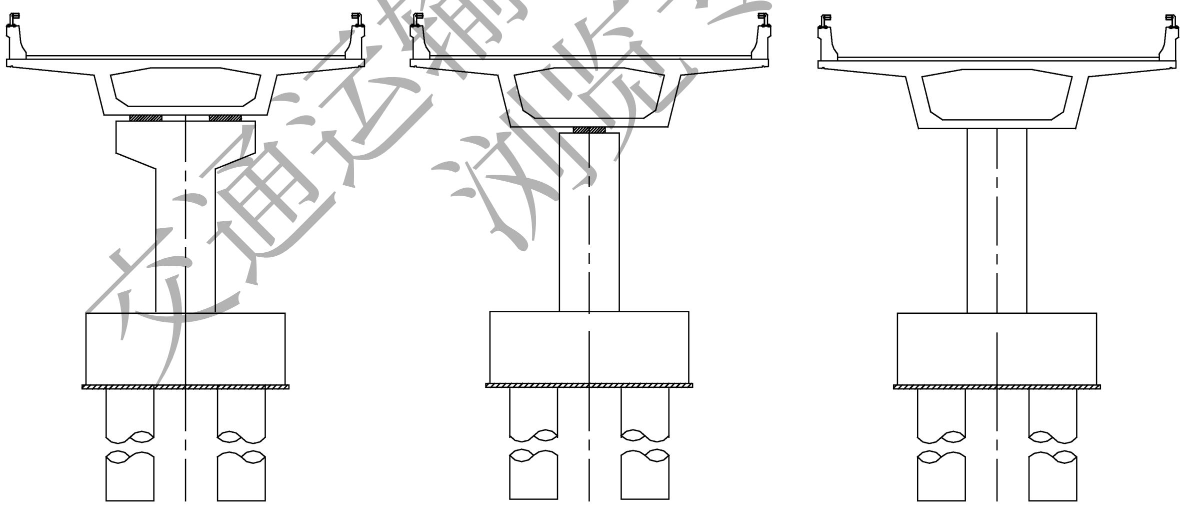
(2) Under special conditions in highways, single-column piers with one bearing are required. For example, the pier location is required to be set at the central divider in the case of crossing roads, due to the limitation of clearance under the bridge. However, in a serial continuous beam, the number of the single-column piers continuously used shall be limited, to guarantee that the overturning resistance of box girders can comply with the provisions of Clause 4.1.8 in the *Specifications*.

(3) Generally, "transverse restrainers" need to be provided at piers or abutments, especially for skewed, curved, or special-shaped bridges as well as bridges with superstructures employing PTFE sliding elastomeric bearings. According to their mechanical behaviors or the slip property of PTFE sliding elastomeric bearings, horizontal torsion and transverse displacement can occur at the ends of main girders. To maintain the planar alignment of girders and normal service of expansion joints and to ensure the safety of girders, transverse restrainer structures shall be provided in piers or abutments. For bridges with a large profile grade, the creeping of main girders is significant, hence, a rigid connection between pier and girder, longitudinal restrainers, and other measures are provided usually.

9.6.10 It was specified in Clause 5.2.5 of *Code for Design of Ground Base and Foundation of Highway Bridges and Culverts* (JTG D63—2007) (hereafter referred to as *Code JTG D 63—2007*) that the depth of pile caps should not be smaller than 1.5 m. Besides this requirement, the depth of the pile caps should usually be taken as 1.0 ~ 2.0 times the diameter of piles, and the multiplier should be increased for drilled piles with a large diameter (e.g., drilled piles with a diameter more than 2.5 m).

In *Specification for Design of Reinforced Concrete Pile Caps* (CECS 88:97) (hereinafter referred to as *Specification CECS 88:97*), it is considered that the arrangement of longitudinal reinforcement in pile caps is related to the understanding of the failure mechanism. When the yield line method ("beam-type system" in the *Specifications*) is used for the analysis, uniform orthogonal reinforcement is often adopted, as specified in Clauses 4.2.3.2 and 5.2.2.1 of

Technical Code of Building Pile Foundation (JGJ 94-94) (hereinafter referred to as *Code JGJ 94-94*). This is also adopted in current highway bridges. When the spatial truss model method ("strut-and-tie system" in the *Specifications*) is used for the analysis, longitudinal reinforcement is often arranged in the slab strips passing through the pile tops. The pros and cons of the above two reinforcement arrangement methods still make it difficult to reach conclusions. It is specified in the *Specifications* that primary reinforcement shall be placed in the range that is 1.5 times the pile diameter distant from the pile center, and construction reinforcement with a reinforcement ratio no less than 0.1% shall be provided beyond the above range, which combines two different types of reinforcement arrangement methods. When the pile spacing is increased to be larger than or equal to 3 times the pile diameter, by referring to Section 16.8 of *Theory of Reinforcement for Reinforced Concrete Structures* authored by F. Leonhardt, hanger reinforcement shall be provided in the middle regions between two piles starting from one pile diameter distant from each pile center. This is because the longitudinal primary reinforcements between two piles are not supported directly by piles, but are subjected to compressive forces from some "struts", so some longitudinal reinforcements in the middle between two piles may be compressed, and cracks may appear in concrete. Hence, hanger reinforcements shall be provided for pile caps with large pile spacing. Generally, the minimum center-to-center spacing of piles is taken as 2.5 times the pile diameter for pile foundations in highway bridges, except for bridges under special ground conditions or in order to avoid the obstruction of underground pipelines. Therefore, there are not many cases with the abovementioned arrangements of construction and hanger reinforcements.



a)Single-column pier with two bearings b)Single-column pier with one bearing c)Single-column pier fixed with girder

Figure 9-6 Typical structural forms of single-column pier

To facilitate construction, surface reinforcement is usually not arranged on the top faces and side faces of pile caps at present, which was required in the former edition of the *Specifications*, and has been removed in this revision.

According to Clause 5.2.5 in *Code JTG D 63—2007*, one layer of wire fabric with 1200 ~

1500 mm^2 per meter wide needs to be provided at the bottom of the pile caps, which is still adopted in this clause. If the top surface of the pile is 100 mm higher than the bottom surface of the pile cap (Clause 5.2.6 in *Code JTG D 63—2007*), primary reinforcements are located at least 130 mm above the bottom surface of the pile cap, which results in a very thick concrete cover for primary reinforcement. Therefore, wire fabric shall be arranged at the bottom of the pile caps.

9.7 Bearings and Expansion Joints

9.7.2 Neoprene has good aging resistance and has been used in most bridges in China. But it has poor resistance to low temperature. Therefore, according to the provisions in *Laminated Elastomeric Bearings for Highway Bridges* (JT/T 4—2004) and *Pot Bearing for Highway Bridge* (JT 391—2009), it is specified that the EPDM (Ethylene-Propylene-Diene Monomer) bearings or natural rubber bearings may be selected to be used in cold zones.

9.7.3 In determining the arrangement of bearings, attention needs to be paid to the following issues:

(1) At one support of beams, only one bearing can be provided in the longitudinal direction. If more than one bearing is provided, uneven stresses in bearings will be induced by deflection angles at the beam ends.

(2) In the transverse direction, uneven stresses can also be found in the case with more than two bearings, although there is no deflection angle in the transverse direction. Therefore, the number of bearings in the transverse direction should not be larger than two.

9.7.4 Measures like padstones set at the top of pile caps and/or pads set at the bottom of slabs are usually taken to keep the level of bearings and to avoid shear deformation of bearings.

9.7.6 Laminated bearings are often affected by the properties of elastomers, and have a design life of about 20 ~ 30 years. The service life of pot bearings and spherical bearings is longer than that of the laminated elastomeric bearings but is shorter than the design life of main structures. Therefore, in the structural design of bridges, maintenance and replacement of bearings during the service life shall be considered, and structural measures for inspection and replacement of bearings must be made for piers or abutments with bearings.

9.7.7 Expansion joints commonly used in current highway bridges include modular joints, finger plate joints, and flexible plug joints. They have a movement range of 20 ~ 3000 mm. Their detailed technical requirements are referred to the current *General Technical Requirements of Expansion and Contraction Installation for Highway Bridge* (JT/T 327).

9.8 Culverts, Lifting Loops and Hinges

9.8.2 This clause is developed by referring to Clause 9.7.6 in *Code for Design of Concrete Structures* (GB 50010—2010). Compared with the former edition of the *Specifications*, the following changes are made:

(1) R235 reinforcement is changed to HPB300 reinforcement;

(2) Stress limit of reinforcement under the action of self-weight is adjusted from 50 MPa to 65 MPa.

According to the durability requirements, for reinforcements of lifting loops bound with reinforcement frames under severe environments, insulation materials to isolate them or other reliable rust prevention measures shall be taken at the contact positions.

9.8.3 Detailing of hinges is specified by referring to *Highway Design Manual: Arch Bridges* (First Volume) (1978) and *Code for Design on Railway Bridge and Culvert* (1958).

Appendix A
Practical Refined Analysis Models for Bridge Structures

A.1 General

A.1.1 Practical refined analysis models are mainly used to solve the spatial effects for bridge structures and to make up for the deficiency of analysis by the single beam model. It has different applicability for different bridge structures.

(1) All the spatial effects of the bridge structures can be considered by the spatial grid model, and the checking index listed in Table 6.1.3 can be obtained completely.

(2) The shear lag effect of the box girder and nonuniform flexural deformation along the transverse direction of the bridge can be analyzed by the folding surface grillage model. However, the horizontal shear stress of the top and bottom slabs cannot be calculated, and the principal tensile stress of the top and bottom slabs in Table 6.1.3 cannot be checked. The longitudinal and transverse members can be analyzed simultaneously by the folding surface grillage model, which is applicable to analyze the longitudinal and cross beams in the wide box girder bridges.

(3) The single beam model with 7 degrees of freedom is applicable to analyze the box girder bridges with significant thin-wall effect, especially for the analysis of the integral behaviors of the curved box girder bridges. The principal tensile stress of the top slab, bottom slab and webs listed in Table 6.1.3 can be obtained. The shear lag effect cannot be calculated by the single beam model with 7 degrees of freedom, because it is a model with a single beam that needs to satisfy the assumption for the full section that plane sections remain plane after deformation. In addition, the behaviors of the bridge deck and the effect due to the excursion force of the prestressing tendon on the bottom slab of the box girder

need to be analyzed by other models.

A.2 Application Principles

A. 2. 1 In the spatial grid model, a complex bridge structure is discretized into multi-plate elements. Each plate element is further discretized into a crossed orthogonal beam grid, which is like a "net", and the stiffness of the plate is substituted by that of the crossed longitudinal and transverse beams. Thus, a bridge structure can be expressed by a spatial grid, as shown in Figure A-1.

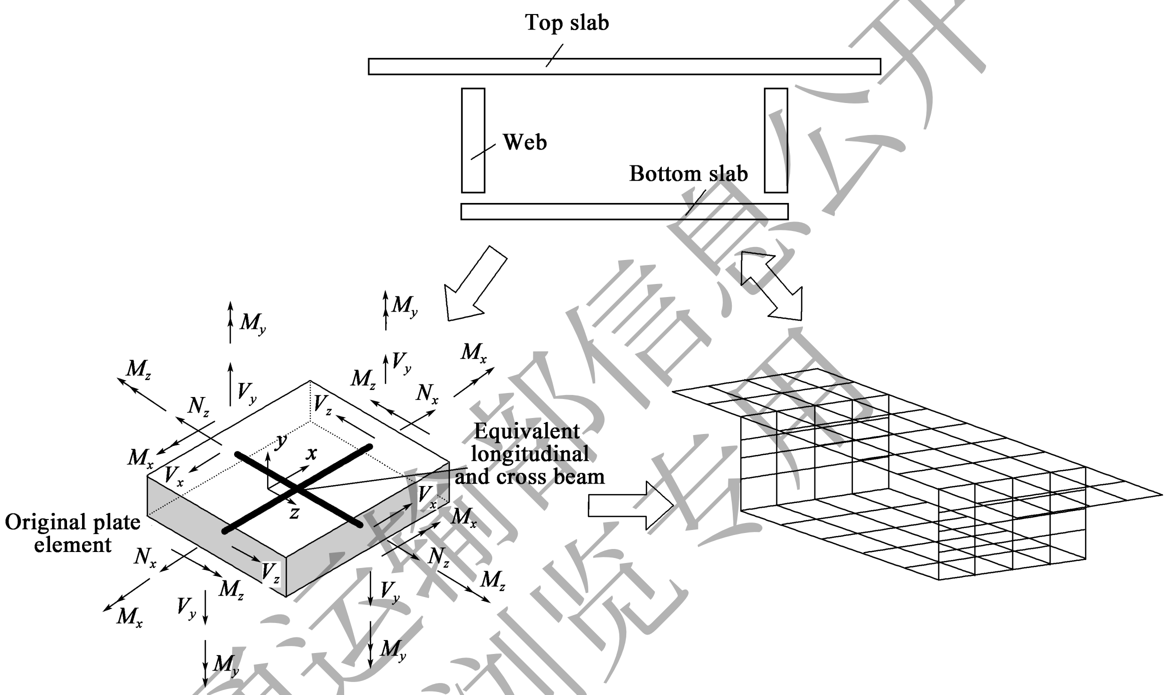


Figure A-1 Principle diagram of spatial gird model simplification

A. 2. 2 In the folding surface grillage model, the section of the box girder is divided with the cutting line perpendicular to the principal axis of the section, and the centroid position of each longitudinal beam element remains unchanged. The transverse beam element is connected to each longitudinal beam element to form a single-layer folding surface grillage model, as shown in Figure A-2.

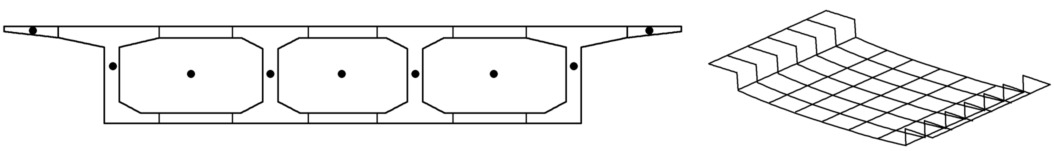
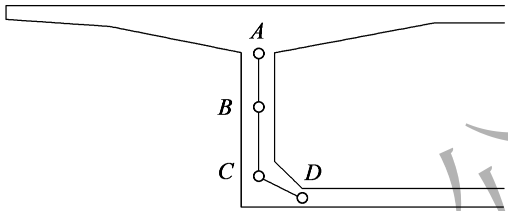


Figure A-2 Diagram of folding surface grillage model and structural discrete

A. 2. 3 The amplification effects of the normal stress and shear stress induced by the torsional effect are estimated when a beam element with 3 or 6 degrees of freedom is used. At present, it is not accurate to use 1. 15 as the stress amplification factor in all cases because the box girders today are widers than before. Three straight prestressed concrete box girder bridges (one with a span of 100 m and girder width of 11. 85 m; one with a span of 268 m and girder width of 16. 5 m; and one with a span of 130 m and a girder width of 16. 5 m) are taken as examples. The analysis results using the single beam model with 7 degrees of freedom indicated that the shear stress amplification factors under live load were 1. 5 ~ 2. 0 at Point *A* (upper edge of the web), Point *B* (centroid of section), Point *C* (lower edge of the web), and Point *D* (edge of bottom slab), as shown in Figure A-3.



igure A-3 Diagram of stress calculation in box girder

Appendix B
Analysis Method Using Strut-and-Tie Model

B.1 General

B.1.1 The disturbed stress flow inside the concrete structure is mainly induced by two factors: one is the action of concentrated force (Figure B-1), for which the range of the disturbed regions is taken as one time the beam depth; the other is the sudden change of the sectional geometry (Figure B-2), for which the range of the disturbed regions is taken as the beam depth in the adjacent area.

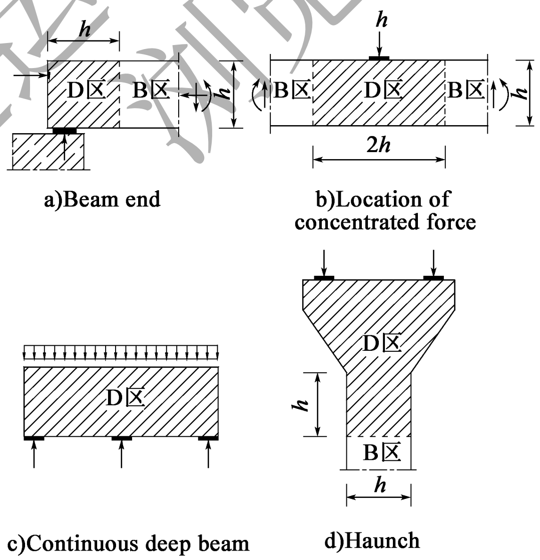


Figure B-1 Sections subjected to concentrated force

B.1.2 The strut-and-tie model is a simplified stress flow analysis model extracted from a continuous body of concrete structure, which is composed of struts, ties and nodes, reflecting the stress transmission path inside the structure. As an example, a strut-and-tie model for the stress transmission of a deep beam under concentrated force, and a strut-and-tie model for the concentrated anchor force spreading inside the anchorage zone of the beam end, are shown in Figure B-3.

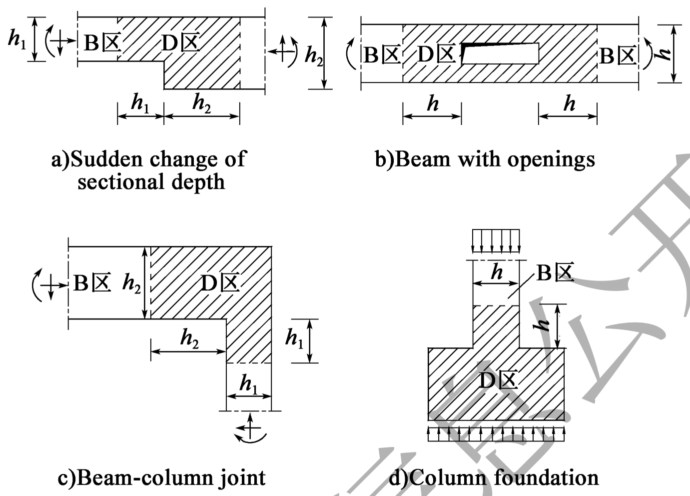


Figure B-2 Sections with sudden change of geometry

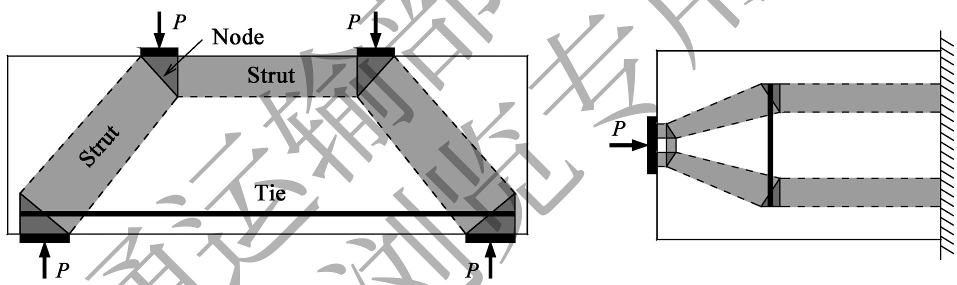


Figure B-3 Basic components of the strut-and-tie model

The tie is a tension member, which is generally composed of reinforcing steel or prestressing steel. The strut represents the resultant force in the compressive regions. The shape of the strut can be prismatic, bottle-shaped or fanshaped, depending on the diffusion feature of the compressive force. The node is located at the intersection of the concentrated force and the axis of the strut or tie, which is the zone where the stress flow is diverted. According to the type of components in the node zone, the nodes can be categorized as Type CCT (compression-compression-tension, a node zone surrounded by multi-struts and one tie), Type CCC (compression-compression-compression, a node zone surrounded by struts), Type CTT (compression-tension-tension, a node zone surrounded by one strut and two or more ties), and Type TTT (tension-tension-tension, a node zone surrounded by ties). According to the diversion feature of stress flow in the node zone, nodes can be categorized into centralized nodes (nodes in deep beam, as shown in Figure B-3) and diffused nodes (nodes in the middle portion inside the end anchorage zone, as shown in Figure B-3). At a centralized node, there is generally at least one definite surface acted by boundary force. The diffused node represents a zone of stress flow diversion, and its boundary is often not very clear. It is unnecessary to check its resistance because the intersection range of the strut and tie herein is relatively wide.

In the design process based on the strut-and-tie model, "developing a strut-and-tie model in the disturbed region" is a difficult step. Theoretically, the strut-and-tie model may be developed according to the method suggested in B. 2. 2 of the *Specifications*, such as those strut-and-tie models presented in the *Specifications*, including the model for the disturbed region of the cantilever pier cap beam, the pier cap and the pile cap. However, it is like research work with a heavy workload to develop the strut-and-tie models for the disturbed regions of the anchorage zone of the post-tensioned end, the anchorage zone of a triangular block, and the diaphragm at supports. Therefore, the formulas for calculating the internal force of ties in these disturbed regions are directly provided in the *Specifications*, and thus two steps for developing the strut-and-tie model and solving the model can be eliminated.

B.2 Development Method

B.2.1 The strut-and-tie model shall satisfy two essential conditions as follows:

(1) Static equilibrium condition, that is, each node in the model shall satisfy the equilibrium equation;

(2) Material yield criterion, that is, the stress of each member and node in the model shall be smaller than the yield stress of the material.

Theoretically, the strut-and-tie model is a lower-bound theorem of plasticity, which will give a conservative estimate of bearing capacity. A variety of models can be selected to analyze the same disturbed region.

However, not all arbitrary models are appropriate because the characteristics of concrete structures are different. In order to avoid that the stress redistribution beyond the plastic deformation capacity of concrete occurs in the structure, the position and direction of struts and ties shall reflect the transmission path of stress flow inside the concrete structures, which is the basic principle to be followed in developing a strut-and-tie model. Taking the deep beam shown in Figure B-4 as an example, the arrangement of ties and struts can reflect the trend of principal tensile and principal compressive stresses, and the internal forces of ties and struts are consistent with the resultant of elastic stress in key sections. Design of reinforcement based on such structural configuration together with reasonable construction reinforcement can not only meet the demand of bearing capacity but also effectively control the crack width at the service stage and reduce the structural stress redistribution during loading.

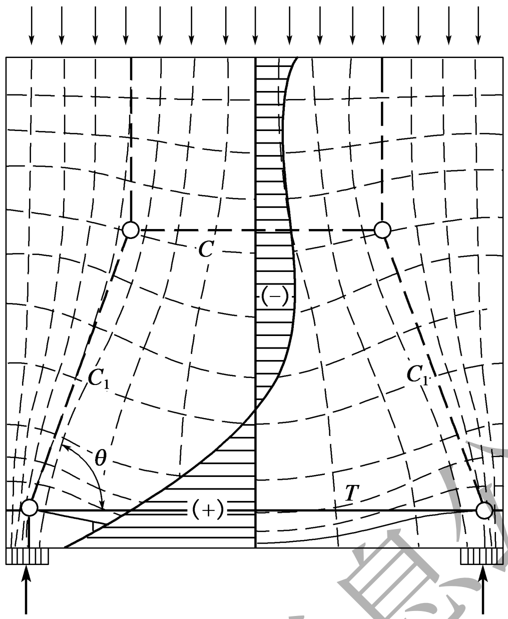


Figure B-4 Strut-and-tie model constructed based on elastic stress of the structure

B. 2. 2 Although the international academic community has made great efforts in the development method of the strut-and-tie model, there is still no universally applicable concise method available. The methods commonly used at present are the load path method, stress trace method, stress fluent-line method, the criterion of minimum strain energy, as well as the criterion of maximum stress. They are briefly introduced as follows:

(1) In the load path method, the transmission path of stress flow inside the structure is directly extracted by experience. Therefore, it requires the designer to have good concepts and engineering experience, and it is generally used for the structures with relatively simple structural geometry and load conditions.

(2) In the stress trace method, the directions of ties and struts are ensured to be consistent with the trace direction of the main stress, and then the exact location of the resultant force, that is, the location of ties or struts, is obtained by integrating stresses of some key sections.

(3) The stress fluent-line method is a quantitative load path method. In this method, the relatively accurate pressure transmission fluent-line is obtained by solving the analytical equation or algorithm of the stress fluent-line. The location of transverse tension or splitting force is obtained by analyzing the transverse component of the compression line, and thus the complete configuration of the strut-and-tie model is developed.

(4) In the criterion of minimum strain energy, it is considered that the real structural

deformation induced by load always consumes the minimum strain energy of the structure. This criterion should also be satisfied for the strut-and-tie model which is discretized from a continuum of structure. Several topology optimization methods based on this criterion are also approaches to obtain the strut-and-tie model .

(5) According to the criterion of maximum stress, the strut-and-tie model shall follow the lower-bound of plasticity, which means that among all possible models, the model with the maximum stress has the closest stress path to the real one. By using this criterion, the key parameters in the model configuration can be determined.

B.2.3 The research results show that the effective compressive strength of concrete in the strut decreases rapidly with the decrease of the angle between the tie and the strut. When the angle is too small(generally, the threshold value is 25°), the model is an inappropriate configuration and cannot truly reflect the transmission path of stress flow according to the maximum stress criterion.

B.3 Checking Contents

B.3.1 This clause presents the checking content for the strut-and-tie model, including the tensile resistance of the tie and the compressive resistance of the strut. Generally, under the condition that the local compression resistance of the supporting surface meets the requirements and the anchorage of the reinforcement is reliable, the resistance of the node is equivalent to that of the intersecting strut and thus is unnecessary to be checked.

The design axial force of ties and struts can be obtained from the static analysis by the developed strut-and-tie model, such as the strut-and-tie models of several typical disturbed regions presented in the *Specifications*(as shown in Clause 8.4.6, Clause 8.4.7, and Clause 8.5.4). In these cases, the design axial force of ties and struts may be calculated in accordance with the ascertained model configuration. For some complex disturbed regions where it is difficult to develop the strut-and-tie model, the design internal force of ties may be directly calculated according to the analytic formulas(such as Clause 8.2.2, Clause 8.2.6 and Clause 8.3.2) obtained from the theory of stress flow model.

B.3.2 ~ B.3.4 The calculation method for design resistance of ties and struts in the strut-and-tie model is specified mainly referring to the relevant provisions of *AASHTO LRFD Bridge Design Specifications*, in which the material index is converted according to that of the standard in China.

The tie is generally composed of reinforcing steel or prestressing steel, and the development length or anchorage detailing of the reinforcement must satisfy the requirements to avoid anchorage failure. The resistance of the strut is controlled by the compressive resistance of its end face with a definite boundary. For the strut with a diffusion node at one end, its boundary is often not very clear and the range is relatively wide, and its resistance is generally not controlled in the design. Two typical struts with definite boundaries are presented in the *Specifications*, namely: 1) the strut restrained by the supporting surface and the reinforcement, and 2) the strut restrained by the supporting surface and the adjacent strut.

The design equivalent compressive strength of the concrete strut $f_{ce,d}$ is reduced mainly by considering the influence of transverse tensile strain in the direction perpendicular to the strut. When the checked nodes adjacent to the strut end are Type CCC, the design equivalent compressive strength of the concrete strut $f_{ce,d}$ is taken as $0.85\beta_c f_{cd}$.

In *AASHTO LRFD Bridge Design Specifications*, the design equivalent compressive strength of concrete strut is expressed as:

$$f_{ce,d} = \frac{0.7f'_c}{0.8 + 170\varepsilon_1} \leqslant 0.85 \times 0.7f'_c \tag{B-1}$$

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + \varepsilon_2)\cot^2\alpha_s \tag{B-2}$$

$$\varepsilon_s = \frac{T_{i,d}}{A_s E_s} \tag{B-3}$$

where:

f'_c——characteristic compressive strength of $\phi150$ mm × 300 mm concrete cylinder at 28d;

ε_s——tensile strain of the reinforcing steel in the tie;

α_s——angle between the compressive strut and adjoining tension ties;

$T_{i,d}$——tension force of the tie intersecting with the strut;

A_s——cross-sectional area of the reinforcing steel in the tie intersecting with the strut;

E_s——elastic modulus of the reinforcing steel in the tie;

ε_2——compressive strain in the direction of the strut, which is taken as 0.002.

According to the material index in Chinese standards, Eq. (B-1) is converted as follows:

The relation between the characteristic compressive strength of 150 mm cubes at 28 d, $f_{cu,k}$, and that of $\phi150$ mm × 300 mm cylinders, f'_c, is:

$$f'_c = 0.80 f_{cu,k} \tag{B-4}$$

In the commentary on Section 3.1 of the *Specifications*, the relation between the characteristic prism compressive strength, f_{ck}, and the characteristic cube compressive strength, $f'_{cu,k}$, is:

$$f_{ck} = 0.88\alpha f_{cu,k} \tag{B-5}$$

where α is recommended by the previous test data and *Guidelines for Design and Construction of High Strength Concrete Structures* (*second edition*) published by China Architecture & Building Press in 2001 (in Chinese) (hereinafter referred to as *High Strength Concrete Guidelines*), for C50 concrete and below, $\alpha = 0.76$. For C55 ~ C80 concrete, $\alpha = 0.78 \sim 0.82$. Considering the brittleness of the concrete above C40, the reduction coefficient for brittleness is taken as 1.00 and 0.87 for C40 and C80 concrete, respectively, and it may be taken by linear interpolation for concrete with intermedia strength grades.

The relation between the design compressive strength of concrete f_{cd} and the characteristic compressive strength of concrete f_{ck} is:

$$f_{cd} = \frac{f_{ck}}{1.45} \tag{B-6}$$

It can be obtained by combining Eq. (B-4), Eq. (B-5), and Eq. (B-6):

$$0.70 f'_c = \beta_c f_{cd} \tag{B-7}$$

Substituted Eq. (B-7) into Eq. (B-1), thus Eq. (B.3.3-2) can be obtained.

Parameter β_c is related to the concrete strength class. The value β_c corresponding to different strength classes of concrete is shown in Table B-1.

Table B-1 β_c corresponding to different strength classes of concrete

Concrete strength class	C25 ~ C40	C45	C50	C55	C60	C65	C70	C75	C80
β_c	1.29	1.32	1.33	1.34	1.34	1.35	1.36	1.36	1.37

For convenience in practice, the values in the above table are properly merged as 1.30 for C25 ~ C50 and 1.35 for C55 ~ C80.

Appendix C
Concrete Shrinkage Strain and Creep Coefficient Calculation, Ratio of Median to Ultimate Values of Prestress Loss due to Prestressing Steel Relaxation

C.1 Shrinkage Strain

C.1.1, C.1.2 Eq. (C.1.1-1) ~ Eq. (C.1.1-5) are compiled with reference to the provisions of *CEB-FIP Model Code 1990* (hereinafter referred to as *CEB-FIP Code*).

The data listed in Table C.1.2 can be approximately applied to the concrete with seasonal temperature variations between −20 ~ +40℃. More accurately, all listed values are only applied to the concrete with a mean temperature between 10 ~ 20℃, otherwise, the effect of the actual deviation on the concrete with temperature ranges from 0℃ to +80℃ to the mean temperature of 20℃ shall be corrected by the following method. The notional shrinkage coefficient and shrinkage development coefficient are corrected by the following equations:

(1) Notional shrinkage coefficient

$$\beta_{RH,T} = \beta_{RH}\beta_{sT} \tag{C-1}$$

$$\beta_{sT} = 1 + \left(\frac{8}{103 - 100\dfrac{RH}{RH_0}}\right)\left(\frac{T/T_0 - 20}{40}\right) \tag{C-2}$$

where:

$\beta_{RH,T}$——coefficient depends on temperature, which is used to substitute β_{RH} in Eq. (C.1.1-2);

β_{RH}——coefficient calculated by Eq. (C.1.1-4);

RH——annual mean relative humidity (%), which is taken as $RH = 55\%$ when $40\% \leq RH < 70\%$, and taken as $RH = 80\%$ when $70\% \leq RH < 99\%$;

T——actual temperature (℃);

$T_0 = 1$℃;

$RH_0 = 100\%$.

(2) Shrinkage development coefficient

$$\alpha_{st}(T) = 350\left(\frac{h}{h_0}\right)^2 \cdot e^{-0.06(T/T_0 - 20)} \tag{C-3}$$

where:

$\alpha_{st}(T)$——coefficient depends on temperature, which is used to substitute the product of 350 $(h/h_0)^2$ in Eq. (C.1.1-5);

T——actual temperature (℃);

$T_0 = 1$℃;

C.1.3 In accordance with this clause, the ultimate shrinkage strain of concrete is obtained as listed in Table C-1.

Table C-1 Ultimate shrinkage strain of concrete $\varepsilon_{cs}(t_u, t_0)$ ($\times 10^{-3}$)

Loading age t_0 (d)	$40\% \leq RH < 70\%$				$70\% \leq RH < 99\%$			
	Theoretical thicknessh (mm)				Theoretical thicknessh (mm)			
	100	200	300	≥600	100	200	300	≥600
3 ~ 7	0.50	0.45	0.38	0.25	0.30	0.26	0.23	0.15
14	0.43	0.41	0.36	0.24	0.25	0.24	0.21	0.14
28	0.38	0.38	0.34	0.23	0.22	0.22	0.20	0.13
60	0.31	0.34	0.32	0.22	0.18	0.20	0.19	0.12
90	0.27	0.32	0.30	0.21	0.16	0.19	0.18	0.12

Notes: 1. This table is applicable for the concrete where ordinary Portland cement or rapid-hardening cement is used.

2. This table is applicable for the concrete exposed on enviroment with mean seasonal temperature variation of −20 ~ +40℃.

3. The values in this table are calculated according to the concrete with strength class of C40. For concrete with a strength grade of C50 or above, the values listed in the table shall be multiplied by $\sqrt{\frac{32.4}{f_{ck}}}$, where f_{ck} is the characteristic axial compressive strength of concrete (MPa).

4. In calculation, the annual mean relative humidity in the table is taken as $RH = 55\%$ when $40\% \leq RH < 70\%$, and taken as $RH = 80\%$ when $70\% \leq RH < 99\%$.

5. The theoretical thickness of the member in the table is $h = 2A/u$, where A is the sectional area of the member, u is the perimeter of the member in contact with the atmosphere. For members with a variable section, the mean value of A and u may be taken.

6. The values in the table are calculated over a 10-year period.

7. If the actual anchorage age, the loading age, or the theoretical thickness of the member are the median values listed in the table, the ultimate shrinkage strain may be calculated by linear interpolation.

C.2 Creep coefficient

C.2.1, C.2.2 Eq. (C.2.1-1) ~ Eq. (C.2.1-7) are compiled with reference to the provisions of

CEB-FIP Code.

The data listed in Table C. 2. 2 can be approximately applied to the concrete exposed on enviroment with seasonal temperature variations between $-20 \sim +40$℃. More accurately, all listed values are only applied to the concrete exposed on enviroment with a mean temperature between 10 ~ 20℃, otherwise, the effect of the deviation for the actual temperature ranged from 0℃ to +80℃ to the mean temperature 20℃ shall be corrected by the following method. The notional creep coefficient and creep development coefficient are corrected by the following equations:

(1) Notional creep coefficient

$$\phi_{RH,T} = \phi_T + (\phi_{RH} - 1) \cdot \phi_T^{1.2} \tag{C-4}$$

$$\phi_T = e^{0.015(T/T_0 - 20)} \tag{C-5}$$

where:

$\phi_{RH,T}$——coefficient depends on temperature, which is used to substitute ϕ_{RH} in Eq. (C. 2. 1-2);

ϕ_{RH}——coefficient calculated by Eq. (C. 2. 1-3);

T——actual temperature(℃).

$T_0 = 1$℃.

(2) Creep development coefficient

$$\beta_{H,T} = \beta_H \beta_T \tag{C-6}$$

$$\beta_T = e^{[1500/(273 + T/T_0 - 5.12)]} \tag{C-7}$$

where:

$\beta_{H,T}$——coefficient depends on temperature, which is used to substitute β_H in Eq. (C. 2. 1-6);

β_H——coefficient calculated by Eq. (C. 2. 1-7);

T——actual temperature(℃);

T_0——1℃.

In addition, the values listed in Table C. 2. 2 are calculated according to the concrete with strength class of C40. It can be found from the test results that the shrinkage and creep of high-strength concrete are less than those of normal concrete, especially for the creep, and they are inversely proportional to $\sqrt{f_{ck}}$. Therefore, for the shrinkage and creep coefficient of concrete with strength class of C50 and above, the values listed in Table shall be multiplied by $\sqrt{\frac{32.4}{f_{ck}}}$ for reduction, in which 32.4 is the characteristic axial compressive strength of C50 concrete, and f_{ck} is the characteristic axial compressive strength of concrete with strength grade of C50 and above.

C. 2. 3 In accordance with this clause, the ultimate creep coefficient of concrete is obtained as listed in Table C-1.

Table C-1 Ultimate creep coefficient of concrete $\phi(t_u, t_0)$

Loading age t_0(d)	40% ≤RH <70%				70% ≤RH <99%			
	Theoretical thicknessh(mm)				Theoretical thicknessh(mm)			
	100	200	300	≥600	100	200	300	≥600
3	3.78	3.36	3.14	2.79	2.73	2.52	2.39	2.20
7	3.23	2.88	2.68	2.39	2.32	2.15	2.05	1.88
14	2.83	2.51	2.35	2.09	2.04	1.89	1.79	1.65
28	2.48	2.20	2.06	1.83	1.79	1.65	1.58	1.44
60	2.14	1.91	1.78	1.58	1.55	1.43	1.36	1.25
90	1.99	1.76	1.65	1.46	1.44	1.32	1.26	1.15

Notes: 1. This table is applicable for the concrete where ordinary Portland cement or rapid-hardening cement is used.

2. This table is applicable for the concrete exposed on enviroment with mean seasonal temperature variation of −20 ~ +40℃.

3. The values in this table are calculated according to the concrete with a strength grade of C40. For concrete with a strength grade of C50 or above, the values listed in Table shall be multiplied by $\sqrt{\frac{32.4}{f_{ck}}}$, where f_{ck} is the characteristic axial compressive strength of concrete(MPa).

4. In calculation, theannual mean relative humidity in the table is taken as $RH = 55\%$ when $40\% \leq RH < 70\%$, and taken as $RH = 80\%$ when $70\% \leq RH < 99\%$.

5. The theoretical thickness of the member in the table is $h = 2A/u$, where A is the sectional area of the member, u is the perimeter of the member in contact with the atmosphere. For members with a variable section, the mean value of A and u may be taken.

6. The values in the table are calculated over a 10-year period.

7. If the actual anchorage age, the loading age, or the theoretical thickness ofthe member are the median values listed in the table, the ultimate creep coefficient may be calculated by linear interpolation.

C.2.4 The calculation for concrete shrinkage and creep in the former edition of the *Specifications* is mainly referred to the analysis model in *CEB-FIP Code*. Its application scope is: stress level $\sigma_c / f_c(t_0) < 0.4$, exposed environment with a mean temperature of 5 ~ 30℃ and a mean relative humidity of $RH = 40\% \sim 99\%$, and concrete made of a mixture with ordinary Portland cement. In this revision of the *Specifications*, the creep coefficient of fly ash concrete was added, considering the current phenomenon that mineral admixtures were mixed in concrete to improve its workability and durability in most of the prestressed concrete structures. The coefficient was specified referring to the creep characteristics of concrete mixing with fly ash, based on the research results from the Project of "Research on long-term evolution law and tracking observation technique for concrete performance of bridges(2006 318 223 02-08)" funded by Ministry of Transport in China.

The effects of fly ash on concrete and its creep mechanism are:

(1) Effect on the strength development of concrete: the jacking of prestressing steel in prestressed

concrete is generally completed within 7 d, and the fly ash has a significant influence on the early strength of concrete.

(2) Effect on the meso- and micro-structures of concrete: fly ash affects the hydration mechanism and meso- and micro-structure of cementitious materials in concrete, and thus will affect the creep effect of concrete materials.

Concrete of C40 and C50 were tested to study the creep characteristics of concrete mixed with fly ash in this project. The test results are shown in Figure C-1 ~ Figure C-4.

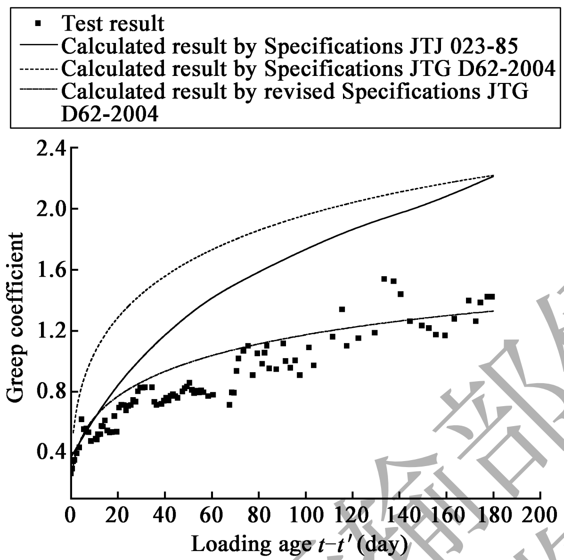


Figure C-1 Comparison of the tested curve and calculated curves by the specification models and revised model, for C40 concrete mixed with 15% fly ash

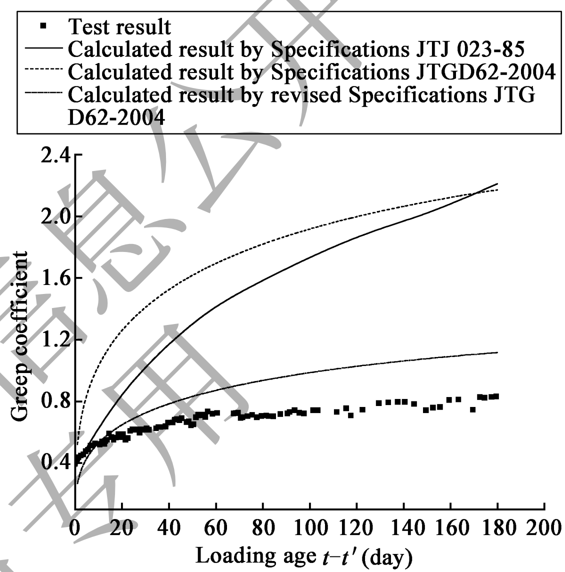


Figure C-2 Comparison of the tested curve and calculated curves by the specification models and revised model, for C40 concrete mixed with 30% fly ash

These figures show a large difference between the calculated results by specification models and the test results for concrete mixing with fly ash, and the deviation would be as high as 40%. Considering the effects of fly ash on the meso-and micro-structures related to creep characteristics and the effect of the early hydration for concrete materials, a correction method was proposed in this project for the notional creep coefficient of concrete, as shown in Eq. (C-8).

$$\phi(\alpha,\ t_0) = \beta(\alpha) \cdot \gamma(\alpha,\ t_0) \tag{C-8}$$

where $\gamma(\alpha,\ t_0)$ is the correction factor of concrete strength; $\beta(\alpha)$ is the correction factor of concrete material related to the content of fly ash. The correction factor can be obtained according to the test results and is expressed by Eq. (C-9).

$$\gamma(\alpha,\ t_0) = \frac{1}{[1.451 - 1.689 \times t_0^{-0.360} \times (1+\alpha)^{0.416}]^{0.5}} \tag{C-9}$$

$$\beta(\alpha) = 1 - 1.0273\alpha^{0.4218}$$

Values listed in Table C. 3. 1 are obtained in accordance with Eq. (C-9).

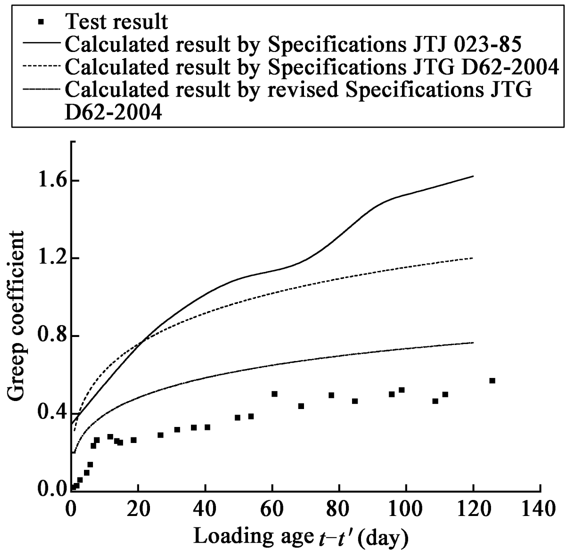


Figure C-3 Comparison of the tested curve and calculated curves by the specification models and revised model, for C50 concrete mixed with 10% fly ash

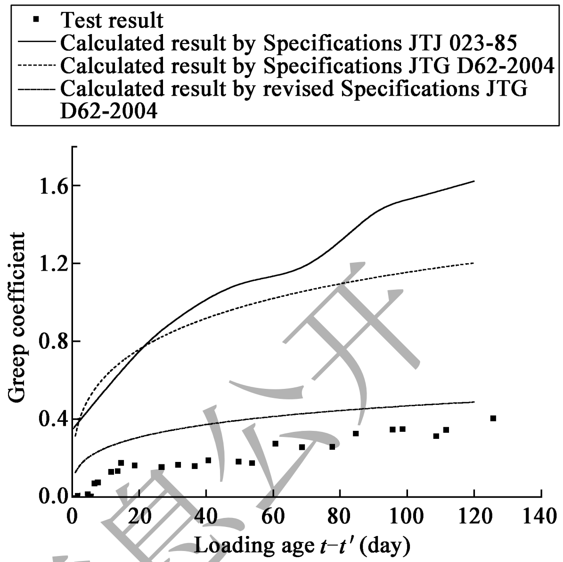


Figure C-4 Comparison of the tested curve and calculated curves by the specification models and revised model, for C50 concrete mixed with 30% fly ash

Appendix D
Calculation Formulas for Temperature Actions

The temperature gradient refers to the temperature difference nonlinearly along the beam section. Since the sectional deformation of the beam complies with the assumption of the plane sections remain plane, the temperature gradient-induced deformation of the beam section is restrained among the longitudinal fibers, and the self-balanced longitudinal restrained stress is generated on the section, which is called as self-balanced thermal stress. In Figure D-1, b) is the temperature gradient (the pattern of the unrestrained free strain is the same as that of the temperature gradient); c) is the deformation of the plane, which is the final strain; d) the shaded part is the difference between the free strain and the final strain, that is the strain of self-balanced thermal stress generated by the restraint among fibers.

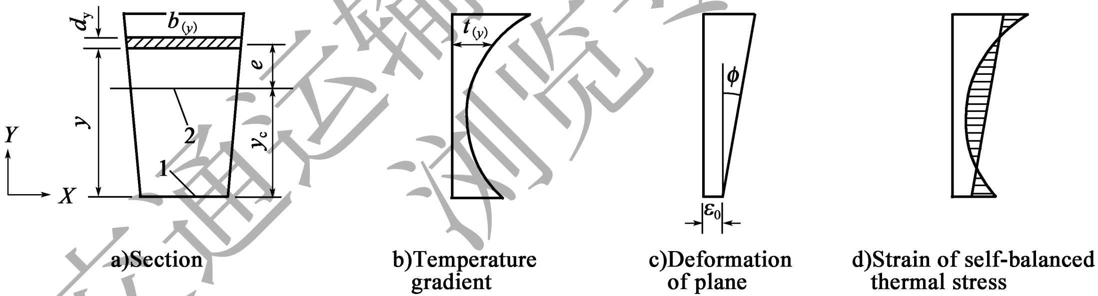


Figure D-1 Calculation pattern of temperature gradient

1-Datum axis; 2-Gravity axis

The free strain along the depth of the beam (when there is no restraint among the longitudinal fibers), $\varepsilon_{t(y)}$, is consistent with the temperature gradient, which is:

$$\varepsilon_{t(y)} = \alpha_c t_{(y)} \tag{D-1}$$

The sectional strain of the beam shall comply with the assumption of the plane sections remain plane after deformation because longitudinal fibers restrain each other. The final strain of the section shall be distributed linearly, which is:

$$\varepsilon_{f(y)} = \varepsilon_0 + \phi y \tag{D-2}$$

where:

ε_0——strain at datum axis $y = 0$;

ϕ——curvature of sectional deformation;

y——coordinates of the strain at any point above the datum axis;

α_c——coefficient of linear thermal expansion of concrete.

The difference between the free strain and the final strain, that is, the shadow in Figure D-1d), is generated because fibers restrain each other, which is:

$$\varepsilon_{\sigma(y)} = \varepsilon_{t(y)} - \varepsilon_{f(y)} = \alpha_c t_{(y)} - (\varepsilon_0 + \phi \cdot y) \tag{D-3}$$

The stress(self-stress) of the shadow is:

$$\sigma_{s(y)} = E_c \varepsilon_{\sigma(y)} = E_c [\alpha_c t_{(y)} - (\varepsilon_0 + \phi \cdot y)] \tag{D-4}$$

Axial force N and bending moment M on the whole cross-section is:

$$\begin{aligned} N &= E_c \int_h \varepsilon_{\sigma(y)} b_{(y)} \mathrm{d}y = E_c \int_h (\alpha_c t_{(y)} - \varepsilon_0 - \phi \cdot y) b_{(y)} \mathrm{d}y \\ &= E_c \left[\alpha_c \int_h t_{(y)} b_{(y)} \mathrm{d}y - \varepsilon_0 \int_h b_{(y)} \mathrm{d}y - \phi \int_h y b_{(y)} \mathrm{d}y\right] \end{aligned} \tag{D-5}$$

$$\begin{aligned} M &= E_c \int_h \varepsilon_{\sigma(y)} b_{(y)} (y - y_c) \mathrm{d}y = E_c \int_h (\alpha_c t_{(y)} - \varepsilon_0 - \phi_y) b_{(y)} (y - y_c) \mathrm{d}y \\ &= E_c \left[\alpha_c \int_h t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y - \varepsilon_0 \int_h b_{(y)} (y - y_c) \mathrm{d}y - \phi \int_h (y - y_c) y b_{(y)} \mathrm{d}y\right] \end{aligned} \tag{D-6}$$

where:

E_c——modulus of elasticity of concrete;

$b_{(y)}$——width of the beam at y.

For any section, $N = 0$, $M = 0$, that is, the sums of the internal forces are equal to zero.

Eq. (D-5) and Eq. (D-6) can be re-written as:

$$\varepsilon_0 \int_h b_{(y)} \mathrm{d}y + \phi \int_h y b_{(y)} \mathrm{d}y = \alpha_c \int_h t_{(y)} b_{(y)} \mathrm{d}y \tag{D-7}$$

$$\varepsilon_0 \int_h b_{(y)} (y - y_c) \mathrm{d}y + \phi \int_h b_{(y)} (y - y_c) \mathrm{d}y = \alpha_c \int_h t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y \tag{D-8}$$

Inside Eq. (D-7) and Eq. (D-8)

$$\int_h b_{(y)} \mathrm{d}y = A \tag{D-9}$$

$$\int_h y b_{(y)} \mathrm{d}y = A y_c \tag{D-10}$$

$$\int_h b_{(y)} (y - y_c) y \mathrm{d}y = \int_h b_{(y)} y^2 \mathrm{d}y - \int_h b_{(y)} y y_c \mathrm{d}y = I_b - \int_h b_{(y)} y y_c \mathrm{d}y = I_g \tag{D-11}$$

$\int_h b_{(y)} (y - y_c) \mathrm{d}y = 0$ (Statical moment of net area with respect to the gravity axis is zero)

where:

A——area of cross-section;

I_b——moment of inertia with respect to the datum axis;

I_g——moment of inertia with respect to the gravity axis.

Substitute Eq. (D-9) ~ Eq. (D-11) into Eq. (D-7) and Eq. (D-8)

$$\varepsilon_0 A + \phi A y_c = \alpha_c \int_h t_{(y)} b_{(y)} \mathrm{d}y \tag{D-12}$$

$$\phi I_g = \alpha_c \int_h t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y \tag{D-13}$$

According to Eq. (D-12) and Eq. (D-13), we can obtain

$$\varepsilon_0 = \frac{\alpha_c}{A} \int_h t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y - \phi y_c \tag{D-14}$$

$$\phi = \frac{\alpha_c}{I_g} \int_h t_{(y)} b_{(y)} (y - y_c) dy \tag{D-15}$$

Set the temperature gradient of a tiny element within the section at the coordinatey is t_y, in which the element area and thickness are A_y and i, respectively. Take t_y as a constant value and substitute it into Eq. (D-14) and Eq. (D-15), and it is noted that the analyzed value only exists in the thickness range of i among the integral section, hence: $\int_h b_{(y)} \mathrm{d}y = \phi \int_h b_{(y)} \mathrm{d}y = A_y$, $t_{(y)} = t_y$, $y - y_c = e_y$ (the eccentricity of element area A_y to the center of gravity of the whole area).

$$\phi = \frac{\alpha_c}{I_g} \int_h t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y = \frac{\alpha_c}{I_g} \int_i t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y = \frac{\alpha_c t_y A_y e_y}{I_g} \tag{D-16}$$

$$\begin{aligned} \varepsilon_0 &= \frac{\alpha_c}{A} \int_h t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y - \phi y_c = \frac{\alpha_c}{A} \int_i t_{(y)} b_{(y)} (y - y_c) \mathrm{d}y - \phi y_c \\ &= \frac{\alpha_c t_y A_y}{A} - \frac{\alpha_c t_y A_y e_y y_c}{I_g} \end{aligned} \tag{D-17}$$

The stress at arbitrary point $\sigma_{s(y)}$ can be obtained from Eq. (D-4):

$$\begin{aligned} \sigma_{s(y)} &= E_c [\alpha_c t_{(y)} - (\varepsilon_0 + \phi \cdot y)] \\ &= E_c \alpha_c t_{(y)} - \frac{E_c \alpha_c t_y A_y}{A} - \frac{E_c \alpha_c t_y A_y e_y y_c}{I_g} - \frac{E_c \alpha_c t_y A_y e_y y}{I_g} \end{aligned} \tag{D-18}$$

If let:

$$N_{ti} = E_c A_y t_y,\ M_{ti} = -N_{ti} e_y = -E_c A_y t_y e_y$$

Then,

$$\sigma_{s(y)} = -\frac{N_{ti}}{A} + \frac{M_{ti}}{I_g}(y - y_c) + t_y \alpha_c E_c \tag{D-19}$$

This equation denotes the stress at any point of the cross-section induced by the temperature action of an element withan area of A_y. If there are various t_y on many elements in the section, the piecewise summation method shall be applied for the analysis of the sectional internal forces, resulting in the equations in Appendix D of the *Specifications*. In Appendix D of the *Specifications*, N_t corresponds to the sum of N_{ti} in this commentary; M_t^0 corresponds to the sum of M_{ti}; y corresponds to $y - y_c$, that is, the coordinates in Appendix D are with reference to the gravity axis of the cross-section.

Eq. (D-19) is applicable to the positive temperature gradient; a minus sign is added to the equation for the negative temperature gradient.

For cracked sections, such as reinforced concrete members or Type B prestressed concrete members permitted to crack, the temperature gradient below the neutral axis on the cracked section may not be taken into account in calculating the effect of temperature gradient in accordance with the equations of this Appendix. For the gravity axis, transformed sectional area and inertia moment, cracked section are adopted to calculate the stress induced by temperature gradient.

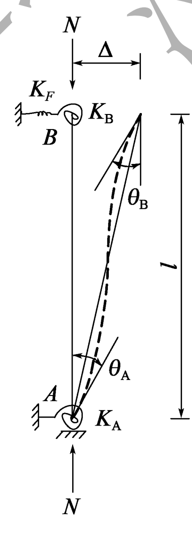
Appendix E
Simplified Calculation Formula for Effective Length of Compression Member

An idealized eccentrically loaded compression member with rotational and translational restraints at its ends provided by the supports is shown in Figure E-1a). These restraints are idealized into rotational and translational springs, whose stiffnesses are represented by K_A, K_B, and K_F, respectively, as shown in Figure E-1b). The relationships among the bending moment, angle of rotation, lateral displacement, and stiffnesses K_A, K_B, and K_F of the member are as follows:

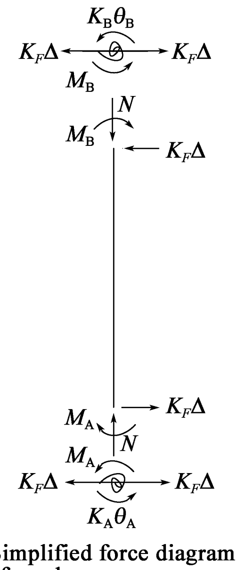
$$K_A = \frac{M_A}{\theta_A} \tag{E-1}$$

$$K_B = \frac{M_B}{\theta_B} \tag{E-2}$$

$$K_F = \frac{M_A + M_B + N\Delta}{\Delta \cdot l} \tag{E-3}$$



a)Force diagram of an elastic column



b)Simplified force diagram of a column

Figure E-1 Elastic-restrained column

In the above equations, N is the axial force of the member; Δ is the relative displacements between two ends of the member; l is the physical length of the member; $M_{\rm A}$ and $\rm M_B$ are the bending moment at two ends of the member, respectively, and their equations of rotation angle are:

$$M_{\rm A}=\frac{EI}{l}\left[s_{ii}\theta_{\rm A}+s_{ij}\theta_{\rm B}-(s_{ii}+s_{ij})\frac{\Delta}{l}\right] \tag{E-4}$$

$$M_{\rm B}=\frac{EI}{l}\left[s_{ji}\theta_{\rm A}+s_{jj}\theta_{\rm B}-(s_{ji}+s_{jj})\frac{\Delta}{l}\right] \tag{E-5}$$

where s_{ii}, s_{ij}, s_{ji}, and s_{jj} are functions for stability, which are calculated by the following equations:

$$s_{ii}=s_{jj}=\frac{\lambda l\sin(\lambda l)-(\lambda l)^2\cos(\lambda l)}{2-2\cos(\lambda l)-\lambda l\sin(\lambda l)} \tag{E-6}$$

$$s_{ij}=s_{ji}=\frac{(\lambda l)^2-\lambda l\sin\lambda l}{2-2\cos\lambda l-\lambda l\sin\lambda l} \tag{E-7}$$

where $\lambda=\sqrt{\frac{N}{EI}}$.

Eq. (E-4) and Eq. (E-5) are substituted into Eq. (E-1) ~ Eq. (E-3) and simplified to obtain:

$$\begin{bmatrix} s_{ij}+k_A & s_{ij} & -(s_{ii}+s_{ij}) \\ s_{ij} & s_{ii}+k_B & -(s_{ii}+s_{ij}) \\ -(s_{ii}+s_{ij}) & -(s_{ii}+s_{ij}) & 2(s_{ii}+s_{ij})-(kl)^2+k_F \end{bmatrix}\begin{bmatrix}\theta_A \\ \theta_B \\ \frac{\Delta}{l}\end{bmatrix}=\begin{bmatrix}0\\0\\0\end{bmatrix} \tag{E-8}$$

where,

$$k_A=\frac{K_A l}{EI},\ k_B=\frac{K_B l}{EI},\ k_F=\frac{K_F l^3}{EI}$$

Eq. (E-8) can be expressed in the matrix form:

$$KD=0 \tag{E-9}$$

where K is the matrix of stiffness, and D is the conversion matrix. In order to obtain an effective solution, take:

$$\det|K|=0 \tag{E-10}$$

that is,

$$\begin{vmatrix} s_{ij}+k_A & s_{ij} & -(s_{ii}+s_{ij}) \\ s_{ij} & s_{ii}+k_B & -(s_{ii}+s_{ij}) \\ -(s_{ii}+s_{ij}) & -(s_{ii}+s_{ij}) & 2(s_{ii}+s_{ij})-(kl)^2+k_F \end{vmatrix}=0 \tag{E-11}$$

Eq. (E-11) is derived to obtain:

$$\begin{aligned}&[k_A+k_B+k_F-(\lambda l)^2](s_{ii}^2-s_{ij}^2)+\{(k_A+k_B)[k_F-(\lambda l)^2]+2k_Ak_B\}s_{ii}+\\ &2k_Ak_Bs_{ij}+k_Ak_B[k_F-(\lambda l)^2]=0\end{aligned} \tag{E-12}$$

that is,

$$\begin{aligned}&\left[1+\frac{k_F-(\lambda l)^2}{k_A+k_{AA}}\right](s_{ii}^2-s_{ij}^2)+\left[k_F-(\lambda l)^2+\frac{2k_Ak_A}{k_A+k_A}\right]s_{ii}+\\ &\frac{2k_Ak_B}{k_A+k_B}s_{ij}+\frac{k_Ak_B}{k_A+k_B}[k_F-(\lambda l)^2]=0\end{aligned} \tag{E-13}$$

Substitute Eq. (E-6) and Eq. (E-7) into Eq. (E-13):

$$\begin{gathered}\left[1+\frac{k_F-\left(\frac{\pi}{k}\right)^2}{k_A+k_B}\right]\left(\frac{\pi}{k}\right)^2+\left[k_F-\left(\frac{\pi}{k}\right)^2+\frac{2k_Ak_B}{k_A+k_B}\right]\left(1-\frac{\pi/k}{\tan(\pi/k)}\right)+\\ \frac{2k_Ak_B}{k_A+k_B}\left[\frac{\pi/k}{\sin(\pi/k)}-1\right]+\frac{k_Ak_B}{k_A+k_B}\left[k_F-\left(\frac{\pi}{k}\right)^2\right]\left[\frac{2\tan(\pi/2k)}{\pi/k}-1\right]=0\end{gathered} \tag{E-14}$$

where $\lambda l=\sqrt{\frac{N}{EI}}l=\pi\sqrt{\frac{N}{N_e}}=\frac{\pi}{k}$, k is the coefficient of effective length, $N_e=\frac{\pi^2EI}{l^2}$, $N=\frac{\pi^2EI}{(kl)^2}$.

For a member with a fixed bottom end and a top end rotation free while translation elastic-restrained, namely $k_A=\infty$, Eq. (E-14) can be simplified as:

$$\begin{gathered}\left(\frac{\pi}{k}\right)^2+\left[k_F-\left(\frac{\pi}{k}\right)^2+2k_B\right]\left(1-\frac{\pi/k}{\tan(\pi/k)}\right)+2k_B\left[\frac{\pi/k}{\sin(\pi/k)}-1\right]+\\ k_B\left[k_F-\left(\frac{\pi}{k}\right)^2\right]\left[\frac{2\tan(\pi/2k)}{\pi/k}-1\right]=0\end{gathered} \tag{E-15}$$

k can be obtained by solving Eq. (E-15) as follows:

$$k=0.5\exp\left[\frac{0.35}{1+0.6k_B}+\frac{0.7}{1+0.01k_F^2}+\frac{0.35}{(1+0.75k_B)(1+1.15k_F)}\right] \tag{E-16}$$

For a member fixed at one end and only horizontally elastically restrained at the other end, $k_B=0$ is taken and substituted into Eq. (E-15) to obtain:

$$\tan\left(\frac{\pi}{k}\right)=\frac{\pi}{k}-\frac{1}{k_F}\left(\frac{\pi}{k}\right)^3 \tag{E-17}$$

Eq. (E-17) is solved directly through numerical calculation to obtain:

$$k=2-\frac{1.3k_F^{1.5}}{9.5+k_F^{1.5}} \tag{E-18}$$

Appendix F
Calculation for Compressive Resistance of an Eccentrically Loaded Reinforced Concrete Compression Member of Circular Cross-section with Longitudinal Reinforcement Evenly Distributed around the Perimeter

Eq. (5.3.8-1) and Eq. (5.3.8-2) are expressed in the form of ultimate load-carrying capacity:

$$N_u = \alpha f_{\mathrm{cd}} A\left(1 - \frac{\sin 2\pi\alpha}{2\pi\alpha}\right) + (\alpha - \alpha_t) f_{\mathrm{sd}} A_s \tag{F-1}$$

$$N_u \eta e_0 = \frac{2}{3} f_{\mathrm{cd}} A r \frac{\sin^3 \pi\alpha}{\pi} + f_{\mathrm{sd}} A_s r_s \frac{\sin\pi\alpha + \sin\pi\alpha_t}{\pi} \tag{F-2}$$

Eq. (F-2) is divided by Eq. (F-1) to obtain:

$$\eta \frac{e_0}{r} = \frac{\dfrac{2}{3}\dfrac{\sin^3\pi\alpha}{\pi} + \rho \dfrac{f_{\mathrm{sd}}}{f_{\mathrm{cd}}} \dfrac{r_s}{r} \dfrac{\sin\pi\alpha + \sin\pi\alpha_t}{\pi}}{\alpha\left(1 - \dfrac{\sin 2\pi\alpha}{2\pi\alpha}\right) + (\alpha - \alpha_t)\rho \dfrac{f_{\mathrm{sd}}}{f_{\mathrm{cd}}}} \tag{F-3}$$

From Eq. (F-1), we can obtain:

$$n_u = \alpha\left(1 - \frac{\sin 2\pi\alpha}{2\pi\alpha}\right) + (\alpha - \alpha_t)\rho \frac{f_{\mathrm{sd}}}{f_{\mathrm{cd}}} \tag{F-4}$$

where $n_u = \dfrac{N_u}{A f_{\mathrm{cd}}}$.

In general, the ratio of the periphery radius where the reinforcement is located to the sectional radius of the member, $\dfrac{r_s}{r} = 0.85 \sim 0.95$, hence $\dfrac{r_s}{r} = 0.9$ is adopted. Given the value of $\eta \dfrac{e_0}{r}$ and $\rho \dfrac{f_{\mathrm{sd}}}{f_{\mathrm{cd}}}$, the half of pressure-angle α may be obtained from Eq. (F-3), and then n_u can be obtained by substituting α into Eq. (F-4).

For concrete with a strength class between C30 and C50, f_{cd} = 13.8 ~ 22.4 MPa. The minimum design yield strength of reinforcing steel (HRB400, HRBF400, RRB400) used as the longitudinal reinforcement in the practical projects is 330 MPa, and the maximum one is 400 MPa (HRB500). The longitudinal reinforcement ratio is considered as 0.5% ~4%, then the minimum of $\rho\frac{f_{sd}}{f_{cd}}$ is $0.005\times\frac{330}{22.4}=0.074$, and the maximum one is $0.04\times\frac{400}{13.8}=1.159$. Take $\rho\frac{f_{sd}}{f_{cd}}=0.06\sim1.20$ and $\eta\frac{e_0}{r}=0.05\sim10$, then values of n_u in Table F-1 are obtained by the above method.

Appendix G
Simplified Calculation for Loss of Prestress in Curved Tendons due to Anchorage Set, Reinforcement Retraction and Joint Compression after Considering Reverse Friction

G. 0. 2 The stress distribution line of prestressing steel after deducting the loss due to the positive friction of the duct is assumed as one straight line *caa'* (Figure G-1). The calculated result shows that the error of this assumption is not large at the instant before anchoring. During anchoring, a retraction of $\sum \Delta l$ will be generated by jacking the prestressing steel. The loss due to retraction after considering reverse friction is the maximum at the jacking end, and gradually decreases away from the jacking end, and becomes zero at the end of the influence length of reverse friction, l_f. Beyond l_f, the stress of the prestressing steel remains unchanged as that before anchoring, that is, it is not affected by retraction. Since the friction coefficients of duct for forward friction and reverse friction are assumed to be equal, the slope of *Line ca* and *Line ea* representing the instantaneous stress change of prestressing steel before and after anchoring, are also the same, but the direction of the friction is opposite. Thus, the stress distribution line of the prestressing steel after anchoring can be represented by the broken line *eaa'* (see Figure G-1).

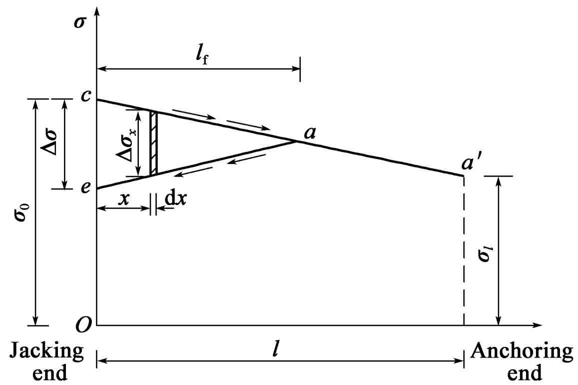


Figure G-1 Stress changes of prestressing steel before and after anchoring

It can be found from Figure G-1 that *Line ca* and *Line ea* are symmetric. The loss of prestress at the jacking end can be obtained by Eq. (G-1):

$$\Delta\sigma = 2\Delta\sigma_d l_f \tag{G-1}$$

where:

$\Delta\sigma_d$——loss of prestress per unit length due to duct friction, which is taken as $(\sigma_0 - \sigma_t)/l$;

l_f——influence length of prestressing steel due to retraction.

Influence length due to retraction (reverse friction), l_f, can be calculated by integral method (that is, calculating the area of cae) according to the retraction $\sum\Delta l$:

$$\sum\Delta l = \int_0 \Delta\varepsilon \mathrm{d}x = \int_0^{l_f} \frac{\Delta\sigma_x}{E_p} dx = \int_0^{l_f} \frac{2\Delta\sigma_d x}{E_p} \mathrm{d}x = \frac{\Delta\sigma_d}{E_p} l_f^2$$

After rearranging terms, we can obtain

$$l_f = \sqrt{\frac{\sum\Delta l \cdot E_p}{\Delta\sigma_d}} \tag{G-2}$$

Eq. (G-2) is only applicable to a member jacked at one end when l_f does not exceed the total length of the member. If the forward friction loss is small and the stress decrease curve is flat, or the retraction is large, l_f may exceed the total length of the member. Herein, it can only be used within the range of l, in which the reinforcement retraction is consistent with the anchorage set, and the loss of prestress is evaluated by the trial calculation.

Appendix J
Calculation for Depth of Compression Zone in Type B Prestressed Concrete Flexural Members Permitted to Crack

Figure 7.1.4 in the *Specifications* shows the cracked section and stress block in Type B prestressed concrete flexural members, in which the internal force is transformed to an eccentric compressive force N_{p0}. Assume the neutral axis of the cracked section is located inside the web, the moments induced by internal and external forces at the point of application of the eccentric compressive force N_{p0} equal to zero, namely $\sum M_{Np0}=0$, we may obtain:

$$\begin{aligned}&\frac{\sigma_{cc}x}{2}\cdot b_{\mathrm{f}}'\left(e_{0N}-C+\frac{x}{3}\right)+\frac{1}{2}\left(\frac{x-h_{\mathrm{f}}'}{x}\right)\sigma_{\mathrm{cc}}(x-h_{\mathrm{f}}')(b_{\mathrm{f}}'-b)\left(e_{0N}-C+h_{\mathrm{f}}'+\frac{x-h_{\mathrm{f}}'}{3}\right)+\\&A_p'\sigma_p'(e_{0N}-C+a_p')+A_s'\sigma_s'(e_{0N}-C+a_s')-A_p\sigma_p(e_{0N}-C+h_p)-\\&A_s\sigma_s(e_{0N}-C+h_s)=0\end{aligned}\tag{J-1}$$

According to Figure 7.1.4 in the *Specifications*, equations can be obtained as follows:

$$\begin{aligned}\sigma_p=\alpha_{EP}\sigma_{cc}\frac{h_p-x}{x},\ \sigma_s=\alpha_{ES}\sigma_{cc}\frac{h_s-x}{x}\\\sigma_p'=\alpha_{EP}\sigma_{cc}\frac{x-\alpha_p'}{x},\ \sigma_s'=\alpha_{ES}\sigma_{cc}\frac{x-\alpha_s'}{x}\end{aligned}\tag{J-2}$$

Let

$$e_{0N}-C=e_N,\ b_f'-b=b_0$$
$$e_{0N}-C+h_p=g_p,\ e_{0N}-C+h_s=g_s$$
$$e_{0N}-C+\alpha_p'=g_p',\ e_{0N}-C+\alpha_s'=g_s'$$

By substituting Eq. (J-2) and the above data into Eq. (J-1), expanding and merging it in the power of x, Eq. (J.0.1-2) ~ Eq. (J.0.1-5) in Appendix J of the *Specifications* can be obtained.

Technical Terms in Chinese and English

序号	中文术语	英文术语
	A	
1	安全等级	safety level
	B	
2	板的厚度	depth of slab
3	板式伸缩装置	mat expansion joint
4	板式橡胶支座	laminated elastomeric bearing
5	绑扎接头	lap splice
6	闭合式箍筋	closed stirrup
7	边缘拉力	edge tension force
8	边界条件	boundary condition
9	边跨	end span
10	边支点/边支座	end support
11	变角度空间桁架模型	variable angle space truss model
12	变截面	variable cross-section
13	剥裂力	spalling force
14	波纹管	corrugated duct
15	部分预应力混凝土	partially prestressed concrete
	C	
16	材料强度标准值	characteristic value of material strength
17	材料强度设计值	design value of material strength
18	长细比	slenderness
19	承台	pile cap
20	承载力	resistance
21	承载能力极限状态	ultimate limit state

续上表(**continued**)

序号	中文术语	英文术语
22	持久状况	persistent situation
23	齿块锚固	anchorage blister
24	冲击系数	impact factor
25	冲切	punching shear
26	冲切破坏	punching shear failure
27	冲切破坏锥体	truncated pyramid for punching shear failure
28	传递长度	transfer length
29	次弯矩	secondary moment
	D	
30	带肋钢筋	deformed bar
31	单室箱梁	single-cell box girder
32	单肢箍筋	single-leg stirrup
33	等截面	constant cross-section
34	底面积	base area
35	吊环	lifting hook
36	吊筋	hanger reinforcement
37	动力系数	dynamic factor
38	短暂状况	transient situation
39	墩帽	pier cap
	F	
40	法向应力	normal stress
41	反拱值	camber
42	放大系数	amplification factor
43	(先张法预应力筋)放松	release
44	分布钢筋	distribution reinforcement
45	分布宽度	distribution width
46	分项系数	partial factor
47	负弯矩区	negative moment region
48	腹筋	web reinforcement
49	复杂桥梁	complex bridge
	G	
50	盖梁	cap beam
51	刚架拱	rigid frame arch
52	钢绞线	(steel) strand
53	钢筋	(steel) bar

续上表（**continued**）

序号	中文术语	英文术语
54	钢筋混凝土	reinforced concrete
55	钢筋混凝土板桥	reinforced concrete slab bridge
56	钢筋网	wire fabric
57	钢丝	(steel) wire
58	拱圈	arch ring
59	公称直径	nominal diameter
60	拱上建筑	arch spandrel structure
61	拱轴系数	arch axis coefficient
62	拱轴线	arch axis
63	箍筋	stirrup
64	光圆钢筋	plain bar
	H	
65	焊接接头	welded joint
66	桁架拱	truss arch
67	荷载组合系数	load factor
68	横隔梁	diaphragm
69	横坡	cross slope
70	横向钢筋	transverse reinforcement
71	后张法	post-tensioning
72	后张法构件	post-tensioned member
73	后张预应力锚固区	post-tensioned anchorage zone
74	滑移	sliding
75	换算截面	transformed section
76	换算截面面积	area of transformed section
77	混凝土保护层	concrete cover
78	混凝土保护层厚度	thickness of concrete cover
79	混凝土弹性压缩	elastic shortening of concrete
80	混凝土龄期	age of concrete
81	混凝土强度等级	concrete strength class
	J	
82	计算跨径	effective span
83	计算宽度	effective width
84	计算长度	effective length
85	计算长度换算系数	effective length factor
86	几何参数标准值	nominal value of geometric parameter

续上表(**continued**)

序号	中文术语	英文术语
87	极限状态	limit state
88	架立钢筋	auxiliary reinforcement
89	间接钢筋	confinement reinforcement
90	剪跨比	shear span-to-depth ratio
91	剪扭构件	member subjected to combined shear and torsion
92	(空心板的)铰缝	joint
93	节段预制拼装结构	precast segmental structure
94	结构重要性系数	importance factor of structure
95	截面受压/拉区边缘	extreme compression/tension fiber of section
96	截面受压区高度	depth of compression zone
97	净截面面积	net cross-sectional area
98	精细化分析	refined analysis
99	局部承压	local bearing
100	聚四氟乙烯滑板式橡胶支座	Polytetrafluoroethylene(PTFE)slide bearing
	K	
101	开裂截面	cracked section
102	开裂弯矩	cracking moment
103	抗冲切承载力	punching shear resistance
104	抗冲切钢筋	punching shear reinforcement
105	抗剪承载力	shear resistance
106	抗剪弹性模量	shear modulus of elasticity
107	抗剪钢筋	shear reinforcement
108	抗拉承载力	tensile resistance
109	抗裂	resistance to cracking
110	抗扭承载力	torsional resistance
111	抗弯承载力	flexural resistance
112	抗弯刚度	flexural stiffness
113	抗压承载力	compressive resistance
114	抗压弹性模量	compressive modulus of elasticity
115	可变作用	variable action
116	空间网格模型	spatial grid model
117	空心板桥	voided slab bridge
118	扩大基础	spread footing
	L	
119	拉筋	tie reinforcement

续上表(**continued**)

序号	中文术语	英文术语
120	拉压杆模型	strut-and-tie model
121	拉应力限值	tensile stress limit
122	立方体抗压强度	cube compressive strength
123	立方体试件	test cube
124	螺旋筋	spiral(reinforcement)
125	螺旋肋钢丝	deformed wire
	M	
126	锚垫板	bearing plate
127	锚固力	anchor force
128	锚固区	anchorage zone
129	锚固长度	development length
130	毛截面面积	gross area of section
131	锚具/锚头	anchorage(device)
132	密集锚头	closely spaced anchorage
133	名义徐变系数	notional creep coefficient
134	名义收缩系数	notional shrinkage coefficient
135	模数式伸缩装置	modular joint
136	摩擦损失	friction loss
137	摩擦系数	coefficient of friction
	N	
138	挠度长期增长系数	factor for long-term deflection
139	牛腿	corbel
	O	
140	偶然作用	accidental action
	P	
141	排架式墩	bent pier
142	配筋率	reinforcement ratio
143	盆式橡胶支座	pot(rubber)bearing
144	劈裂力	bursting force
145	疲劳应力幅	fatigue stress range
146	偏心距	eccentricity
147	偏心受拉构件	eccentrically loaded tension member
148	偏心受力构件	eccentrically loaded member
149	偏心受压构件	eccentrically loaded compression member
150	普通钢筋	reinforcing steel, steel bar

续上表(**continued**)

序号	中文术语	英文术语
	Q	
151	汽车荷载	vehicular load
152	汽车制动力	vehicular braking force
153	墙式墩	wall pier
154	桥面铺装	deck overlay
155	倾覆	overturning
156	球形支座	spherical bearing
157	全预应力混凝土	fully prestressed concrete
	R	
158	软钢	mild steel
159	软化	softening
	S	
160	三铰拱	three-hinged arch
161	设计状况	design situation
162	伸缩缝	joint gap
163	伸缩量	movement range
164	伸缩装置	expansion joint
165	施工荷载	construction load
166	收缩	shrinkage
167	收缩应变	shrinkage strain
168	受拉钢筋	tension reinforcement
169	受压钢筋	compression reinforcement
170	受压翼缘	compression flange
171	梳齿板式伸缩装置	finger plate joint
172	束筋	bundled bar
173	塑性抵抗矩	plastic section modulus
174	塑性铰	plastic hinge
	T	
175	T 梁	T-girder
176	体积配筋率	volumetric ratio of reinforcement
177	体内预应力筋	internal tendon
178	体外预应力筋	external tendon
179	体外预应力桥梁	bridge with external tendons
	W	
180	(普通)弯起钢筋	bent bar

续上表（**continued**）

序号	中文术语	英文术语
181	温差	temperature difference
182	温度梯度	temperature gradient
183	无缝式伸缩装置	flexible plug expansion joint
184	无铰拱	hingeless arch
	X	
185	系杆拱	tied arch
186	先张法	pretensioning
187	先张法构件	pretensioned member
188	现场浇筑构件	cast-in-place member
189	线膨胀系数	coefficient of thermal expansion
190	相对湿度	relative humidity
191	箱型截面梁桥	box girder bridge
192	消除应力钢丝	stress-relieved wire
193	消压	decompression
194	斜板桥	skewed slab bridge
195	斜腹板	inclined web
196	斜交角	skew angle
197	斜拉破坏	diagonal tension failure
198	斜压破坏	diagonal compression failure
199	修正系数	correction factor
200	徐变	creep
201	徐变系数	creep coefficient
	Y	
202	应力扰动区	disturbed region
203	应力松弛	relaxation
204	应力图形	stress diagram
205	应力云图	stress contour
206	有效承压面积	effective bearing area
207	有效预应力	effective prestress
208	预拱度	pre-camber
209	预压应力	precompression stress
210	预应力	prestress / prestressing force
211	预应力钢绞线	prestressing strand
212	预应力钢筋	prestressing steel
213	预应力钢束	prestressing tendon

续上表(**continued**)

序号	中文术语	英文术语
214	预应力管道	tendon duct
215	预应力混凝土	prestressed concrete
216	预应力损失	loss of prestress
217	预应力弯起/曲线钢筋	curved tendon
218	预制构件	precast member/element
219	圆柱体抗压强度	cylinder compressive strength
220	圆桩	circular pile
221	约束扭转	warping torsion
	Z	
222	张拉力	jacking force
223	张拉控制力	maximum jacking force
224	张拉控制应力	maximum jacking stress
225	折减系数	reduction factor
226	整体现浇	cast-in-place
227	正常使用极限状态	serviceability limit state
228	正交板	orthotropic plate
229	正截面抗弯承载力	flexural resistance of cross-section
230	正弯矩区	positive moment region
231	直腹板	vertical web
232	中间跨	interior span
233	中间支点/支承/支座	interior support
234	中心距	center-to-center spacing
235	中性轴	neutral axis
236	轴心抗压强度	axial compressive strength
237	轴心受拉构件	axially loaded tension member
238	轴心受压构件	axially loaded compression member
239	主钢筋/受力主筋	primary reinforcement
240	主拉/压应力	principal tensile/compressive stress
241	转动刚度	rotational stiffness
242	桩基承台	pile cap
243	装配式	prefabricated
244	自由扭转	St. Venant torsion
245	纵向(受力)钢筋	longitudinal(primary)reinforcement
246	纵向钢筋配筋率	longitudinal reinforcement ratio
247	组合构件	composite member

续上表(**continued**)

序号	中文术语	英文术语
248	组合式受弯构件	composite flexural member
249	组合箱梁桥	composite box girder bridge
250	作用频遇组合	frequent combination of actions
251	作用准永久组合	quasi-permanent combination of actions

References

[1] AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020
[2] BS EN 1990:2002, Eurocode—Basis of structural design
[3] BS EN 1991-1-1: 2002, Eurocode 1: Actions on structures—Part 1-1: General actions—Densities, self-weight, imposed loads for buildings
[4] BS EN 1992-1-1: 2004, Eurocode 2: Design of concrete structures—Part 1-1: General rules and rules for buildings
[5] BS EN 1993-1-1: 2005, Eurocode 3: Design of steel structures—Part 1-1: General rules and rules for buildings
[6] BS EN 1994-1-1: 2004, Eurocode 4: Design of composite steel and concrete structures—Part 1-1: General rules and rules for buildings
[7] BS EN 1994-2: 2005, Eurocode 4—Design of composite steel and concrete structures—Part 2: General rules and rules for bridges
[8] BS EN 1998-1: 2004, Eurocode 8: Design of structures for earthquake resistance—Part 1: General rules, seismic actions and rules for buildings
[9] BS EN 12390-3: 2019, Testing hardened concrete Part 3: Compressive strength of test specimens
[10] CEB-FIP Model Code 1990: Design Code
[11] Design Manual for Roads and Bridges (DMRB) CD 357 Revision 1 (2020) Bridge expansion joints
[12] JTG/T D65-06-2015 (E), Specifications for Design of Highway Concrete-filled Steel Tubular Arch Bridges