JSCE Guidelines for Concrete
No.15

STANDARD SPECIFICATIONS FOR CONCRETE STRUCTURES – 2007
"Design"

Japan Society of Civil Engineers

JSCE Guidelines for Concrete
No.15

STANDARD SPECIFICATIONS FOR CONCRETE STRUCTURES – 2007
"Design"

Published by
Subcommittee on
English Version of Standard Specifications for Concrete Structures - 2007
Japan Society of Civil Engineers (JSCE)
Yotsuya 1-chome, Shinjuku-ku, Tokyo 160-0004, JAPAN
FAX +81-3-5379-2769 E-mail pub@jsce.or.jp
December 2010

Printed in Japan
英文版　2007年制定コンクリート標準示方書［設計編］ 1.0
ISBN 978-4-8106-0752-9

Preface

Concrete structures have supported our society as infrastructures. The society can only preserve itself in wholesome with tough, beautiful and durable concrete structures. Concrete Committee of Japan Society of Civil Engineers (JSCE), leading organization for investigation, research, technological promotion and education of concrete in Japan, considers the issuance and revision of Standard Specifications for Concrete Structures as its most important activity. Standard Specifications for Concrete Structures (JSCE-SSCS), which show the model for plan, design, execution, maintenance and repair of concrete structures, have been highly recognized in practice and contributed to the development of concrete technology in Japan since its first publication as “Standard Specifications for Reinforced Concrete – 1931.”

In order to cope with the development of concrete technology in Japan and the worldwide trend, Concrete Committee converted all Specifications in JSCE-SSCS namely ‘Structural Performance Verification,’ ‘Seismic Performance Verification,’ ‘Materials and Construction,’ ‘Maintenance,’ ‘Dam Concrete’ and ‘Pavement,’ from the “Prescriptive Code” to “Performance-based Code” and completed the work in 2002.

This revised edition adopts the technological development after 2002 and intends to enhance the performance-based nature in Standard Specifications. For practical efficiency, the three Specifications - ‘Design,’ ‘Materials and Construction’ and ‘Dam Concrete’ present not only general provisions for verification of specified performance requirements but also standard methods as simplified methods to achieve the performance requirements under certain conditions. JSCE-SSCS is ready for practical use as it describes the role of each Specification during the plan, design, execution, maintenance and repair phases as well as the relationship among them. And for the first time, it also describes roles of engineers for construction works. This JSCE-SSCS is yet an ultimate one. There are still remaining tasks, such as inclusion of provisions for scenario of concrete structures during service life. What readers can find in this revised edition is rationality with “the performance-based concept” and the applicability for practice, showing the high level of technology in Japan.

This revised JSCE-SSCS consists of five Specifications; ‘Design,’ which combines the previous ‘Structural Performance Verification’ and ‘Seismic Performance Verification’ together, ‘Materials and Construction,’ ‘Maintenance,’ ‘Dam Concrete’ and ‘Test Methods and Specifications.’ Specification of ‘Test Methods and Specifications’ was issued separately in May 2007. Specification of ‘Pavement’ was published as “Standard Specifications for Pavements – 2007” by the Committee on Pavement Engineering of JSCE, which has taken over the issuance and revision works from JSCE-SSCS.

Finally I would like to show my most sincere gratitude to Prof Taketo UOMOTO and Dr Tadayoshi ISHIBASHI, Chairman and Secretary General of Sub-committee on Revision of Standard Specifications for Concrete Structures as well as its Secretaries, Conveners and Members who devoted themselves continuously despite the tight drafting schedule. My gratitude also goes to Advisors, Secretaries, Executive Members and Members of Concrete Committee, who reviewed the draft.

December 2007

Toyo. Miyagawa

Toyoaki MIYAGAWA, Chairman
Concrete Committee of Japan Society of Civil Engineers

Preface to the English Version

The Japan Society of Civil Engineers' (JSCE) Concrete Committee has been publishing the Standard Specifications for Concrete Structures in Japanese since 1931. The English versions were published twice in 1987 and 2005 when the limit state design and the performance-based concept were introduced in the 1986 and 2002 editions of Standard Specifications for Concrete Structures (JSCE-SSCS) for the first time, respectively.

Since 2004 the Concrete Committee has put efforts to enhance information dissemination overseas by presenting various English publications including the series of "JSCE Guidelines for Concrete." Concrete Committee has also decided to prepare the English version of every edition of JSCE-SSCS. This Sub-committee on English Version of Standard Specifications for Concrete Structures was established in 2008.

Our task is to prepare the English version of four Specifications: 'Design,' 'Materials and Construction,' 'Maintenance' and 'Dam Concrete' of the 2007 edition of JSCE-SSCS. Specification of 'Test Methods and Specifications' of JSCE-SSCS is not included in this English version. However, some of these standard test methods and specifications have been translated for publication in a series of "JSCE Guidelines for Concrete." Please visit the website of Concrete Committee at http://www.jsce.or.jp/committee/concrete/e/index.html for the information on the English publications.

This English version includes most of the contents in the original Japanese version. Utmost efforts have been made to ensure that the translation accurately convey the description in the original Japanese version. If there were any discrepancy between the Japanese and English versions, however, reference should be made to the original Japanese version.

Translation of technical work does not only require expertise but a lot of time and dedication. I am grateful to all the members for their tiring efforts. My heartfelt appreciations go to Prof YOKOTA Hiroshi (Secretary General), Dr SHIMOMURA Takumi (Head, WG for Design), Prof SUGIYAMA Takafumi (Head, WG for Materials and Construction), Dr MAEDA Toshiya (Head, WG for Maintenance), Prof AYANO Toshiki (Head, WG for Dam Concrete) and Dr ISHIZUKA Takayuki who proof-read all the translated Specifications. Without them this English version would not have been published.

December 2010

Tamon Ueda

UEDA Tamon, Chairman
Sub-committee on English Version of Standard Specifications for Concrete Structures

JSCE Committee on
Revision of Standard Specifications for Concrete Structures

Taketo UOMOTO, Chairman

Tadayoshi ISHIBASHI, Secretary

Members

Hiroyuki IKEDA	Tomoya IWASHITA	Tamon UEDA
Kimitaka UJI	Hidetaka UMEHARA	Nobuaki OTSUKI
Hiroshi ONUMA	Satoshi OKAZAWA	Tsutomu KANAZU
Yuichi KANEKO	Toru KAWAI	Hirotaka KAWANO
Etsuo SAKAI	Koji SAKAI	Tsutomu SATO
Ryoichi SATO	Motoyuki SUZUKI	Shigeyuki SOGO
Koji TAKEWAKA	Takao CHIKADA	Yukikazu TSUJI
Tomoaki TSUTSUMI	Rokuro TOMITA	Kazuyuki TORII
Junichiro NIWA	Yoshinobu NOBUTA	Chikanori HASHIMOTO
Makoto HISADA	Tsutomu FUKUTE	Koichi MAEKAWA
Yasunori MATSUOKA	Kyuichi MARUYAMA	Tetsuya MISHIMA
Toyoaki MIYAGAWA	Hiroshi MUTSUYOSHI	Hiroshi YOKOTA
Keitetsu ROKUGO		

Former Members

Takashi SASAKI	Yasuo INOKUMA

Working Group for 'Design'

Junichiro NIWA, Chairman

Hiroyuki IKEDA, Vice Chairman

Tsutomu SATO, Secretary

Members

Tadatomo WATANABE	Motoyuki SUZUKI	Takumi SHIMOMURA
Yoshiki ISHIKAWA	Katsuhiro ICHINAMI	Tamon UEDA
Isao UJIKE	Yuichi UCHIDA	Soji OHSHIRO
Akira OGAWA	Tsutomu KANAZU	Toshiro KAMADA
Kenji KAWAI	Takanobu KIMURA	Hiroshi SHIMA
Takafumi SUGIYAMA	Yukihiro TANIMURA	Tetsuo SANTO
Takeshi TSUYOSHI	Toru TERAYAMA	Hikaru NAKAMURA
Takashi HANASHIMA	Yuzuru HAMADA	Mitsuo HARADA
Koichi MAEKAWA	Tetsuya MISHIMA	Hiroshi MUTSUYOSHI
Shinichi YAMANOBE	Hiroshi YOKOA	Hiroshi WATANABE

Former Members

Yasuo INOKUMA	Shizuo TANAKA	Shinichi TAMAI
Yasuhiko NISHI		

Subcommittee on English Version of Standard Specifications for Concrete Structures

Tamon UEDA, Chairman

Hiroshi YOKOTA, Secretary General

Secretaries

Toshiki AYANO	Takayuki ISHIZUKA	Takumi SHIMOMURA
Takafumi SUGIYAMA	Toshiya MAEDA	

Members

Tetsuya ISHIDA	Hajime ITO	Mitsuyasu IWANAMI
Takao UEDA	Atsushi UENO	Masahiro OUCHI
Yoshinobu OSHIMA	Yoshitaka KATO	Minoru KUNIEDA
Yoshimori KUBO	Hirohisa KOGA	Koichi KOBAYASHI
Shigehiko SAITO	Goro SAKAI	Yasutaka SAGAWA
Toshiya TADOKORO	Hiroaki TSURUTA	Hideki NAITO
Kohei NAGAI	Kenichiro NAKARAI	Akira HOSODA
Tomohiro MIKI	Maki MIZUTA	Hiroshi MINAGAWA
Shinichi MIYAZATO	Toshinobu YAMAGUCHI	Ken WATANABE

Working Group for 'Design'

Takumi SHIMOMURA, Chairman

Members

Hajime ITO	Mitsuyasu IWANAMI	Shigehiko SAITO
Toshiya TADOKORO	Hideki NAITO	Kohei NAGAI
Kenichiro NAKARAI	Tomohiro MIKI	Ken WATANABE

JSCE Guideline for Concrete No. 15

Standard Specifications for Concrete Structures -2007 "Design"

CONTENTS

GENERAL REQUIREMENTS

STANDARD METHODS

Application of Standard Specifications for Concrete Structures

Editorial notes for the English Version:

(1) The Standard Specifications for Concrete Structures are a model code which is supposed to be applied in Japan and have no legal bindings.

(2) Therefore, some contents, such as Chapters 2 and 3 of “Application of Standard Specifications for Concrete Structures,” may not fit to practical scheme in construction industry in the other countries.

(3) However, it is hoped that the Standard Specifications for Concrete Structures could be a model code for the other countries after necessary modifications of the contents considering the local conditions.

1. Basic concept concerning the organization of the Standard Specifications for Concrete Structures

The Standard Specifications for Concrete Structures are regularly revised reflecting the state-of-the-art concrete technologies developed in Japan and other countries, and provide standards concerning the technical aspects of concrete structures in a series of phases from planning to design, construction and maintenance.

In this revised edition, the “Standard Specifications for Concrete Structures, Design” is composed of the “Standard Specifications for Concrete Structures, Structural Performance Verification” and “Standard Specifications for Concrete Structures, Seismic Performance Verification” to enhance the convenience of design practice. Durability check and initial cracking check, which should be discussed in the “Standard Specifications for Concrete Structures, Design” have been transferred from the “Standard Specifications for Concrete Structures, Materials and Construction” to “Standard Specifications for Concrete Structures, Design”. The preparation and revision of the “Standard Specifications for Concrete Structures, Pavement” has been handed over to the Committee on Pavement Engineering, JSCE (Japan Society of Civil Engineers). The results are now published separately under the title of the Standard Specifications for Pavement. Thus, the Standard Specifications for Concrete Structures include five components: Design, Materials and Construction, Maintenance, Dam Concrete, and Test Methods and Specifications.

The General Requirements for “Design”, “Materials and Construction” and “Dam Concrete” are described based on the concept of performance-based code. Performance requirements are specified for structures and the methods for checking the compliance with requirements are shown in the General Requirements. In the Standard Methods, standard methods for satisfying the General Requirements under certain conditions are given for more efficient and simpler design and construction. In cases where no conditions specified in the Standard Methods are met, performance verification should be conducted in accordance with the General Requirements. For establishing new standards fitting for structures or regions to which no Standard Methods are applicable, the Standard Methods may be referred to.

Concrete structures are generally constructed for providing services in the phases of planning, design, construction and maintenance in accordance with the respective components of the Standard Specifications. Each type of work is not independent of the others. Data are handed over from an upstream to a downstream phase that are required for carrying out the work downstream to meet the conditions specified upstream. Handing over the data is therefore important to proper implementation of work in respective phases. In the Standard Specifications for Concrete Structures, "Design", "Materials and Construction" and "Maintenance" are closely interrelated to one another. Then, the required data shown in each Specifications should be accurately handed over to the next phase without fail.

Performance requirements for durability, safety, serviceability and restorability that are specified in the "Standard Specifications for Concrete Structures, Design" are determined in the design phase. Construction and maintenance methods are roughly determined in the phase. The data that affect construction and maintenance should therefore be handed over to the next phase without fail in the form of design drawings.

Construction records that are specified in the "Standard Specifications for Concrete Structures, Materials and Construction" provide important data for assessment, deterioration prediction and implementation of remedial measures in the maintenance phase. Accurate construction records should therefore be handed over to maintenance engineers. Construction plans and various inspection reports should also be provided to engineers as required.

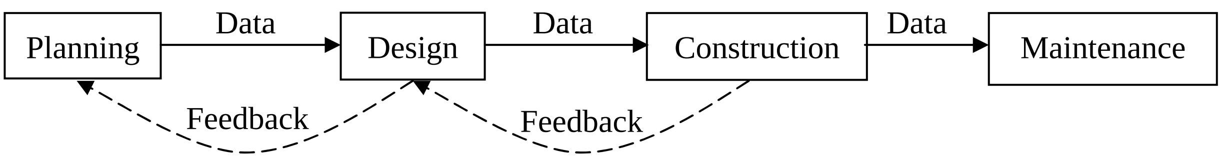


Fig. 1 Flow of work

Figure 1 shows a flow of work from the planning of a concrete structure to service commencement. Data collected in the maintenance phase are not transferred to engineering works in the planning, design or construction phase. It should, however, be taken into consideration that reflecting the data in the maintenance phase in the planning, design and construction of another structure for improvement is important to extend the service life of the structure. The interrelationship is basically the same for design, construction and maintenance described in the "Standard Specifications for Concrete Structures, Dam Concrete."

Each component of the Standard Specifications for Concrete Structures is described below.

The "Standard Specifications for Concrete Structures, Design" shows standard methods for performance verification of concrete structures such as reinforced concrete, prestressed concrete and steel-concrete composite structures, and stipulates the preconditions for checking and structural details. The revised Standard Specifications for Concrete Structures are not applicable to unreinforced concrete structures. Material design values or other applicable items may, however, be applied to unreinforced concrete structures.

The "Standard Specifications for Concrete Structures, Materials and Construction" provides basic general rules concerning the construction of concrete structures. In the construction phase, the construction method and the performance during the construction work are determined based on the design drawings and restrictions on construction. Then, materials are selected and concrete mix proportions are determined, where a concreting plan is developed so as to meet the requirements for water content, cement amount, cement type and other parameters. Whether the concreting plan meets the construction requirements or performance requirements of the structure or not is verified by an appropriate method. If the requirements are not satisfied, the concreting method is re-specified or the mix proportions are modified as long as the conditions handed over from the design phase are met.

The “Standard Specifications for Concrete Structures, Maintenance” provides general basic principles concerning the maintenance of concrete structures. In the maintenance phase, documents such as the design drawings and maintenance plans handed over from the design phase, and the construction plans, as-built drawings, construction records and inspection reports handed over from the construction phase should be fully used for efficient and effective maintenance work. In cases where the use or functions of the structure change due to social changes, performance verification should be made to verify whether the structure meets the resultant performance requirements or not. If designated performance requirements are not ensured, repair, strengthening or other remedial measures should be considered.

The “Standard Specifications for Concrete Structures, Dam Concrete” stipulates performance and quality requirements for dam concrete, and describes the methods for verifying the compliance with the requirements and the basic design and construction principles. The descriptions concerning design, construction and maintenance in the “Standard Specifications for Concrete Structures, Dam Concrete” are different from the contents of the Standard Specifications “Design”, “Materials and Construction” and “Maintenance” with many respects because of the factors unique to dam concrete such as the unreinforced nature and low or zero slump of dam concrete. The “Standard Specifications for Concrete Structures, Dam Concrete” therefore describes the matters concerning the design, construction and maintenance of dam concrete.

The “Standard Specifications for Concrete Structures, Test Methods and Specifications” lists the Japan Industrial Standards, provisions of JSCE and other standards for the methods mentioned in the other four components of the Standard Specifications. Figures 2 through 4 show work steps in respective phases described in the Standard Specifications for Concrete Structures “Design,” “Materials and Construction” and “Maintenance.”

2. Roles and deployment of responsible engineers

To produce and maintain a reliable structure that meets the performance requirements, the engineers involved should have a capacity to carry out the work and a high level of ethics.

In the planning, design, construction and maintenance of a concrete structure, the engineers should make appropriate decisions under varying work conditions. Therefore, the engineers with required technical skills should be deployed according to the level of difficulty of work. In the planning, design, construction and maintenance, therefore, responsible engineers not only with required technical expertise but also with responsibility and authority should be deployed in organizations of the owner, the consultant and the contractor.

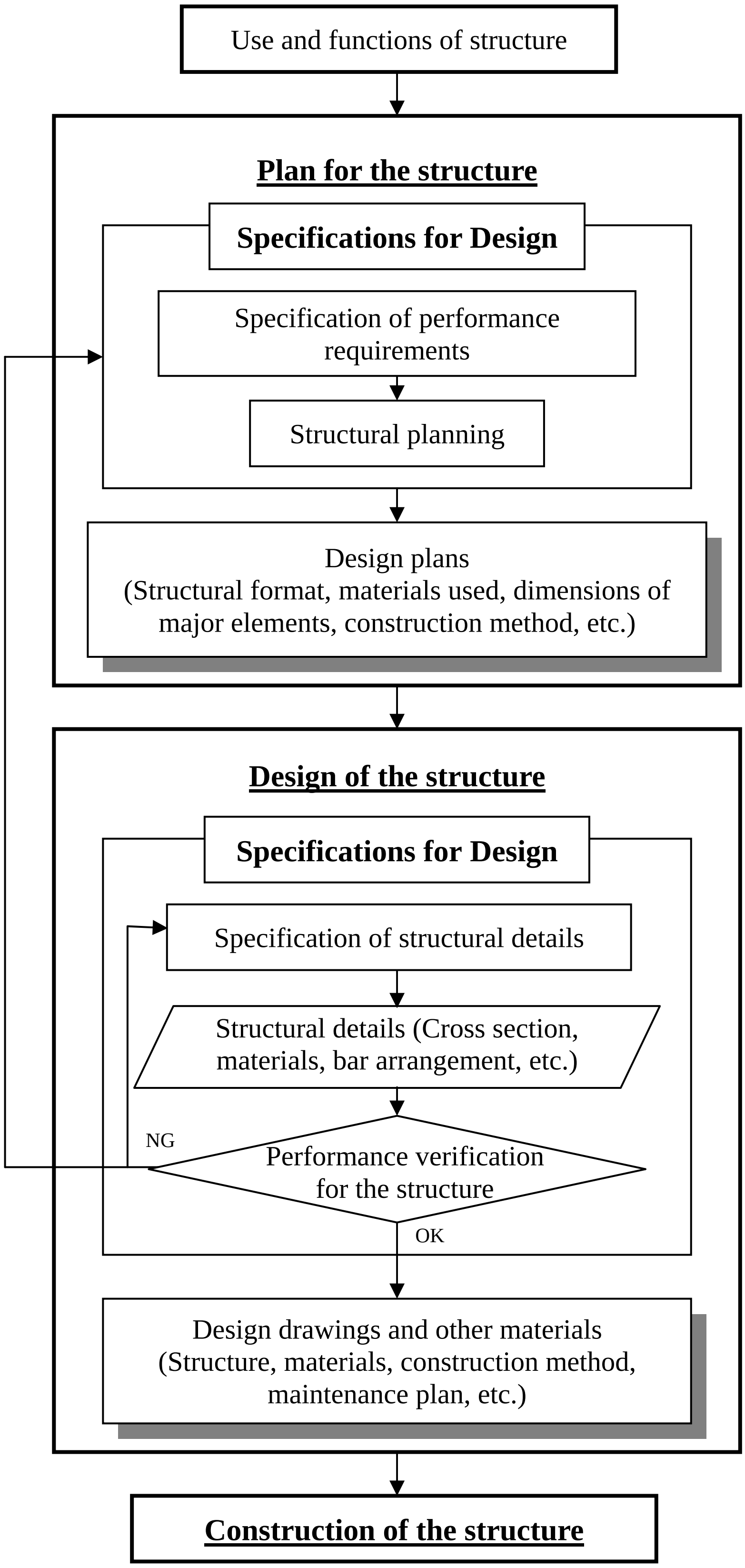


Fig.2 Work steps described in "Specifications for Design"

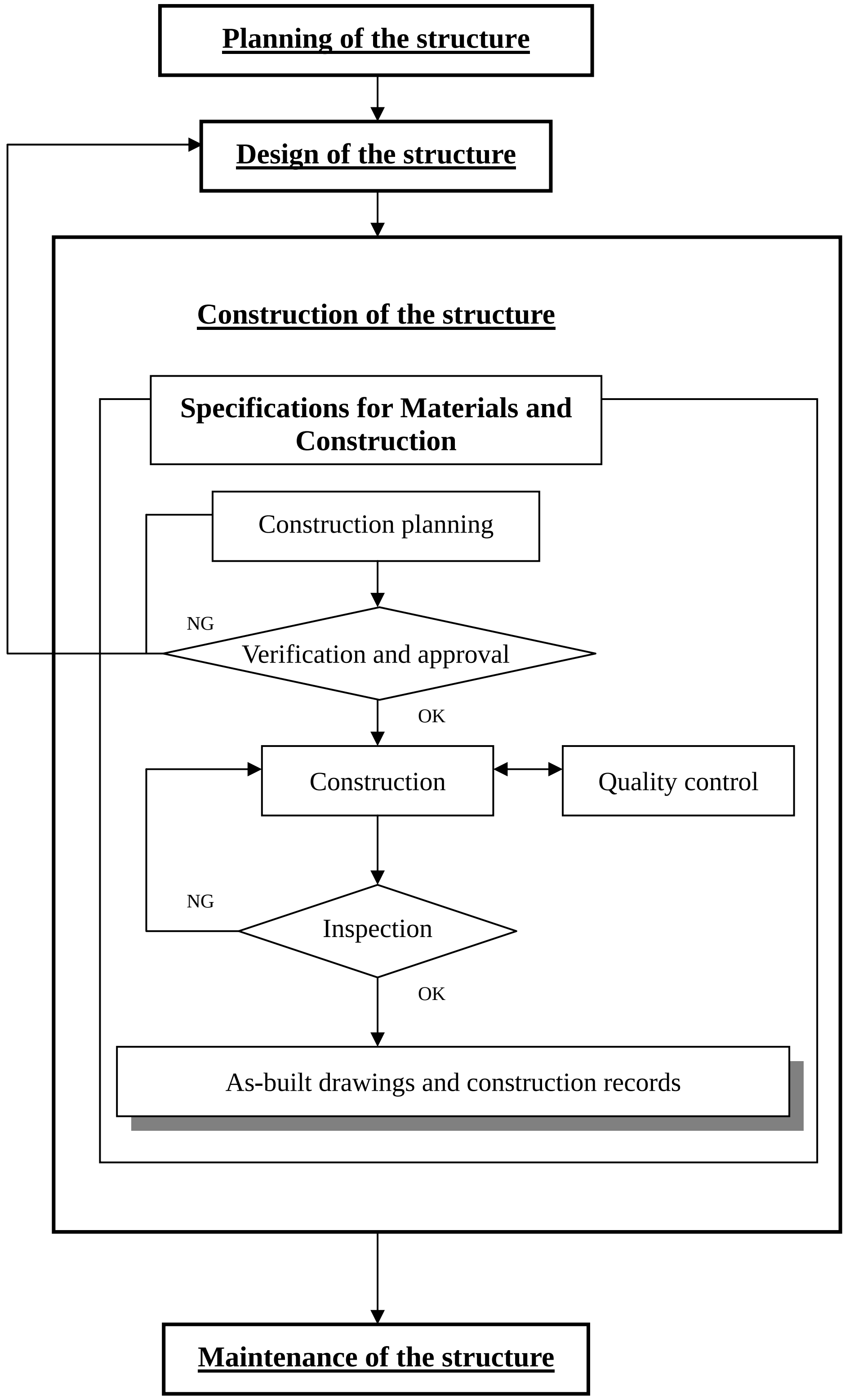


Fig. 3 Work steps described in "Specifications for Materials and Construction"

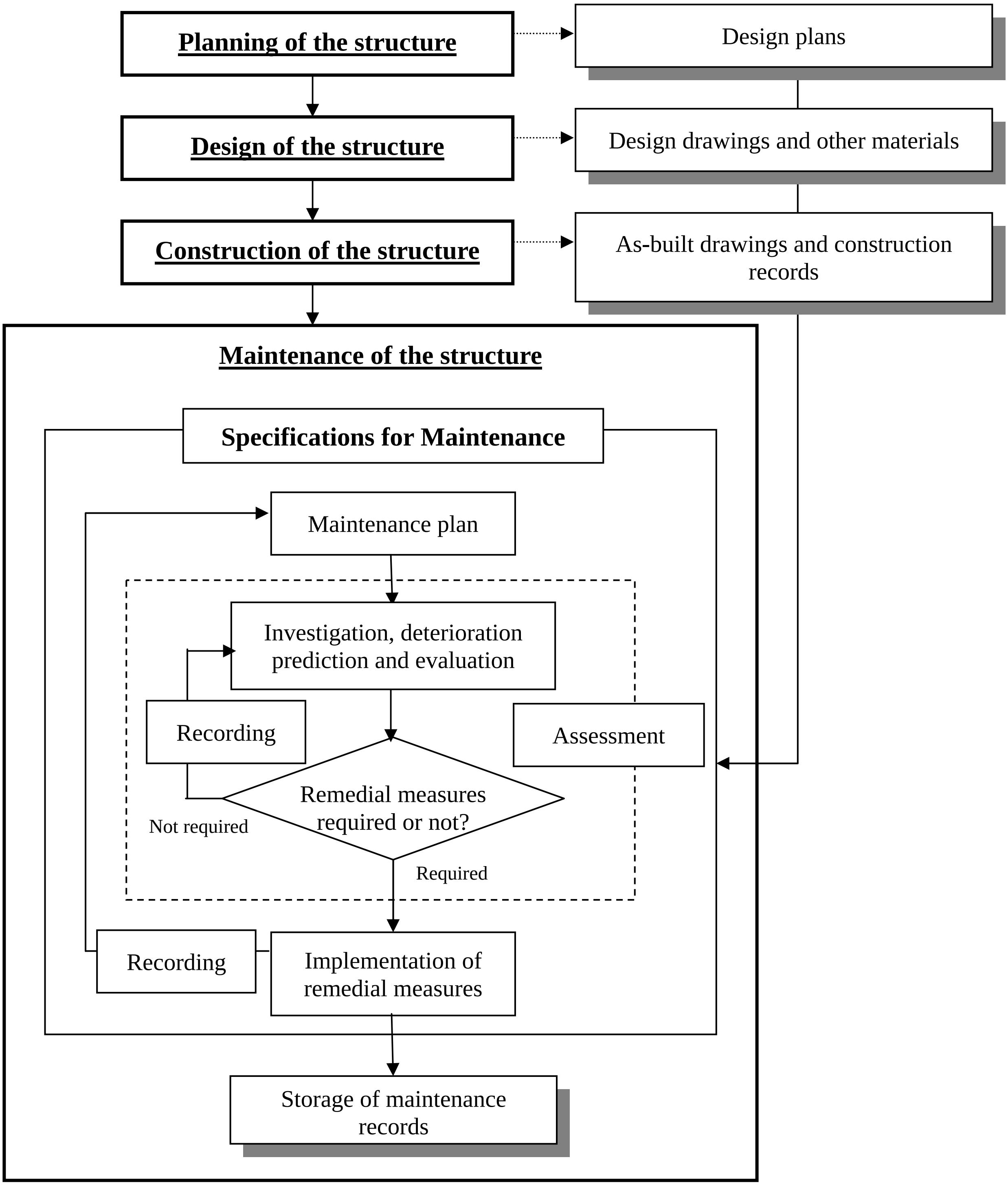


Fig. 4 Work steps described in “Specifications for Maintenance”

The technical skills required for responsible engineers should be defined according to a scale of the work, importance and difficulty of planning, design, construction or maintenance.

As capacity classifications of engineers, engineer qualifications authorized by JSCE are listed in Table 1. The qualifications for the special senior engineers and senior engineers are generally required for responsible engineers.

The JSCE technical qualifications cover several fields. Responsible engineers to be deployed in projects need to have qualifications only for major fields related to the specific project.

Table 1 Engineer qualifications authorized by JSCE

Qualification	Required skills
Executive Professional Civil Engineer	Japan's leading civil engineer with high-level knowledge and experience in his or her field of specialty, or with comprehensive civil engineering expertise
Senior Professional Civil Engineer	Engineer with high-level knowledge and experience in multiple fields or with comprehensive knowledge on civil engineering who can solve key problems as a leader
Professional Civil Engineer	Engineer with knowledge and experience at least in one special field who can carry out task at his or her discretion
Associate Professional Civil Engineer	Civil engineer with required basic knowledge who can carry out assigned task

3. System for ensuring reliability

Many groups are involved in the planning, design, construction and maintenance of a structure. In order to ensure the high reliability of the structure in each work phase, the organizations involved should play their role providing their know-how and assuming due responsibility.

To ensure reliability in the design (and planning) phase, two independent groups should basically check the design. After the design and check by the design company, engineers of the contractor should re-check the design, or request a third party to check the design. Then, fully skilled engineers should be selected for checking so that safety or other reliability parameters may be satisfied. The design drawings serving as a basis for contracting should carry the signatures of responsible engineers of the two groups that assume responsibility.

In the construction phase, reliability is ensured through the quality management by the contractor and the quality verification by the inspector independent of the contractor. Inspections have generally been conducted directly by the Owner and/or Consultant. Completed structures should be inspected wherever possible. If inspecting completed structures is impossible, inspections should be conducted while the structure is being constructed. If the Owner cannot directly inspect the structure under construction, the Owner may request an agent independent of the contractor. Adopting as many highly reliable methods as possible can reduce the labor required for quality management and inspection. In cases where adopting not so reliable methods is inevitable, the level of quality management should be raised or inspections should be conducted more frequently to improve reliability. Inspection items and decision criteria should be specifically presented at the time of contracting because they greatly affect the quality of the structure and the construction cost.

Ensuring safety requires regular investigations. When defects are detected, decision should be made as to remedial measures. For decision making concerning remedial measures for extremely difficult deformation, listening to the opinions of engineers with high skills who have experienced numerous cases is important.

In order for a system for ensuring reliability to work properly, the people or organization with technical skills fit for the specific work should be granted explicit responsibility and authority and assigned to the work. The compensation for the work and time required for the work should also be provided.

General Requirements

CHAPTER 1 GENERAL

1.1 Scope

This specification, "Design," (hereafter, the Specification) provides principles for structural design and verification of performances of all concrete structures including those made with reinforced concrete, prestressed concrete and steel-concrete composites, as well as the prerequisite for verification and the structural details.

[Commentary] The construction and management of concrete structures involves design, construction planning, fabrication and erection, construction in practice, and maintenance. At each stage, the work shall be carried out in a manner that all requirements specified in any upstream stage are satisfied. In case the verification at a previous stage is not satisfied, or in the case there is a change in the requirements at an upstream stage, the verification shall be repeated so that the condition above is met.

This specification provides the standard method of verification for durability, safety, serviceability and restorability of structures in the design stage of all concrete structures, including those made with reinforced concrete, prestressed concrete and steel-concrete composites, as well as the prerequisite of verification and the structural details. In certain special cases, it is possible that provisions in the Specification may not be sufficient, or in fact, the provisions in the specification may not be strictly applicable. In cases where adequate structural performance can be confirmed using prototype experiments assuming design loads, scaled model experiments, or numerical analyses whose accuracy and applicability have been ensured, the procedure for verification of structural performance as laid down in the present specification may not be followed based on the decision by a responsible engineer. However, even in such cases, the principles laid down in the present specification should be sufficiently respected. The provisions of the present specification may be taken to be generally applicable in cases where the characteristic compressive strength of concrete does not exceed 80 N/mm^2.

In principle, the limit state design method is adopted in the present specification for the verification of structural performance for durability, safety, serviceability and restorability, in view of the satisfactory performance of this methodology in the past.

Some structures may involve the use of special construction materials or special construction methods, which are not completely covered by the Specification. Reference should be made to other publications of the JSCE, which may provide guidelines or recommendations for the design and construction of such special structures, or use of special materials. Additional recommendations may be published by the JSCE from time to time, depending upon the requirement. The following is a list of JSCE publications that are considered to complement the Specification. Certain publications such as "Recommendations for Maintenance of Concrete Structures" or "Proposed Specifications of Durability Design for Concrete Structures", whose provisions have been essentially included in the present specification, have not been included in the list.

"Manual of Design and Construction for Lightweight Aggregate Concrete Structures," 1985.

"Recommendations for Design and Construction of Structures by Prestressed Concrete Panel Composite Slab Method," 1987.

"Recommendations for Design and Construction of Concrete Containing Ground Granulated Blast-furnace Slag as an Admixture," 1988.

“Recommendations for Design and Construction of Prestressed Concrete Structures,” 1991.

“Recommendations for Design and Construction of Anti-washout Underwater Concrete,” 1991.

“Recommendations for Design and Construction of Reinforced Concrete Structures Using D57 and D64 Large-diameter Threaded Reinforcing Bars,” 1992.

“Design Code for Steel-Concrete Sandwich Structures,” 1992.

“Recommended Practice for Expansive Concrete,” 1993.

“Recommendations for Design and Fabrication of Diffusion Bonded Joints in Reinforcing Bars Using Amorphous Metal Foil,” 1993.

“Recommendations for Design and Construction of Concrete Structures Using Silica Fume in Concrete,” 1995.

“Recommendations for Design and Construction of Concrete Structures Using Continuous Fiber Reinforcing Materials,” 1995.

“Recommendations for Design and Construction of Composite Structures,” 1997.

“Guidelines for Retrofit of Concrete Structures,” 1999.

“Recommendations for Design of Reinforced Concrete Columns with Steel Fiber,” 1999.

“Verifications of Seismic Performance of In-ground LNG Tank Structures,” 1999.

“Recommendations for Upgrading of Concrete Structures with Use of Continuous Fiber Sheets,” 2000.

“Recommendations for Design and Construction of Self-Compacting High Strength and High Durability Concrete Structures,” 2001.

“Recommendations for Design and Construction of Electrochemical Corrosion Control Method,” 2001.

“Recommendations for Design and Construction of Concrete Structures Using Epoxy-Coated Reinforcing Steel Bars [Revised Edition],” 2003.

“Recommendations for Concrete Repair and Surface Protection of Concrete Structures,” 2005.

“Recommendations for Design and Construction of Recycled Aggregate Concrete Using Demolished Concrete of Electric Power Plants,” 2005.

“Recommendations on Environmental Performance Verification of Concrete Structures (Draft),” 2005.

“Recommendations for Mix Design of Fresh Concrete and Construction Placement related Performance Evaluation,” 2007.

“Recommendations for Design, Fabrication and Evaluation of Anchorages and Joints in Reinforcing Bars [2007],” 2007.

1.2 Basic Rules of Design

(1) Design shall include the identification of performance requirements for a structure, structural planning and detailing to meet those performance requirements, and the verification to make sure the performance requirements are met throughout the design service life of the structure.

(2) When designing a structure, all performance requirements that need to be met in order to meet the intended purpose of the structure during construction and during the design service life of the structure shall be determined in accordance with Chapter 2.

(3) In structural planning, the type of structure and other requirements shall be determined in accordance with Chapter 3, taking into consideration such factors as structural characteristics, materials, construction method, maintenance method and economy, so that the performance requirements can be met.

(4) In structural detailing, structural details such as shapes, dimensions and reinforcement patterns shall be determined in accordance with the structural detailing requirements described in this Specification for the type of structure determined in structural planning.

(5) In the verification with respect to performance requirements, it shall be verified in accordance with Chapter 4 that the performance requirements for the structure related to durability, safety, serviceability, restorability, and the impact on the environment and landscapes are satisfied throughout the design service life.

[Commentary] When designing a structure, it is necessary to make effort to attain the required performance and construct a highly rational structure according to the purpose of the design of each structure, taking into consideration such factors as natural conditions, social conditions, constructibility, economy and environmental compatibility. The safety, serviceability and restorability of structures specified in this Specification are strongly affected by structural details such as shapes, dimensions and reinforcement patterns and the mechanical characteristics of materials. It may generally be said, therefore, that many of those details are determined by this Specification. Structural details, however, such as shapes, dimensions and reinforcement patterns are deeply related to constructibility, etc., indicated in the Construction section of this Specification. In order to develop a rational design, it is necessary to take those structural details into consideration in advance so that the designed structure does not fail to meet the constructibility requirement. Concrete structures may be difficult to repair or strengthen after they are constructed. It is therefore necessary to determine structural details by conducting a thorough study at the initial stage of design and predicting events that can occur during the service life of the structure with reasonable accuracy, taking maintainability into consideration. The basic rules of maintenance are described in the Maintenance section of this Specification.

(1) The design of a structure consists of a series of tasks, namely, the determination of performance requirements, structural planning, structural detailing, and the verification of performance requirements, and these tasks must be performed consistently (see Fig. C1.2.1).

(2) Performance requirements for a structure need to be determined appropriately in view of such factors as the intended purpose of the structure and the degree of importance of the structure. Performance requirements include those determined by the physical properties of a structure and those determined by other factors. Both types of performance requirements need to be determined for each structure. This Specification deals mainly with performance requirements determined by

physical properties. Other performance requirements, therefore, need to be determined appropriately on an as-needed basis.

(3) When determining the type of structure and other details in structural planning, it is necessary to take into consideration all factors such as structural characteristics, materials to be used, construction method, maintenance method and economy in order to meet the specified performance requirements. From the viewpoint of the rationality of design, it is important in structural planning to rely on performance requirement verification results wherever possible in order to prevent changes in details such as the type of structure.

It is also necessary at the stage of structural planning to take into consideration matters that cannot be verified concretely in accordance with this Specification.

(4) In structural detailing, information necessary for the verification of such details as the dimensions of structural members, reinforcement patterns and the performance of the materials to be used is specified for the type of structure specified at the structural planning stage. When doing this, it is necessary to appropriately take into consideration the structural details indicated in this Specification, design manuals, details of similar structures, and empirical information. It is also necessary, at this stage, to determine structural details giving consideration to maintenance facilities related to the maintainability of the structure, such as inspection openings and stairways.

(5) In the verification of performance requirements, verification for the specified performance requirements is made in accordance with this Specification by using the information on the specified structural details. The term "performance verification" refers to the task of ascertaining, by an appropriate method, that the type of structure, structural cross sections, materials to be used, structural specifications, etc., that have been determined taking into consideration such factors as natural conditions, social conditions, constructibility and economy meet the specified performance goals. An "appropriate method" refers to an experiment using a material, member or structure to be used or a model, a numerical analysis with known accuracy, etc. If it is difficult to use these methods, it is a basic rule in this Specification to make verification by the limit state design method. In such cases, it is essential that the preconditions for verification specified in this Specification be met.

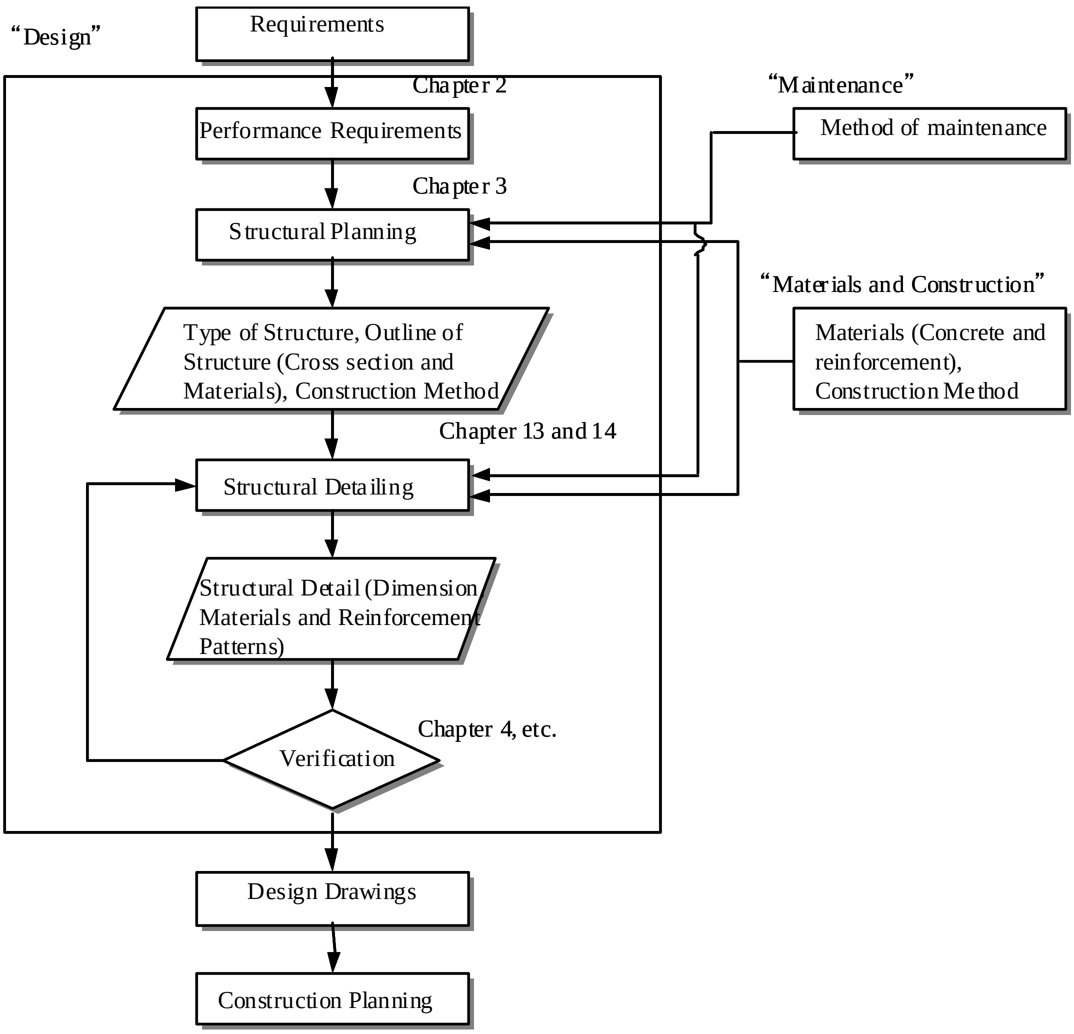


Fig. C1.2.1 Structural design flow

1.3 Definitions

The following terms are defined for general use in this Specification.

Design—An act consisting of the determination of the required performance of a structure, structural planning, structural detailing and performance verification

Required performance—Performance required of a structure according its purpose and function

Verification—The act of checking whether a structure has the required performance, for example by a verification experiment on full-scale specimens or an empirically and theoretically proven analysis method

Durability—The resistance of a structure to performance degradation over time due to the deterioration of materials in the structure caused by expected deteriorating agencies

Safety—The performance of a structure in preventing risks to users and other people in the vicinity

Serviceability—The performance of a structure that enables users to use the structure comfortably or prevents other people in the vicinity from feeling uncomfortable because of the structure, and the performance that ensure other functions required of the structure are performed appropriately

Restorability—Performance that enables functional restoration and continue use of a deteriorated structure

Earthquake resistance—Earthquake-related performance encompassing safety during an earthquake and post-quake serviceability and restorability

Service life—Period during which a structure is in service

Design life—Period specified in design during which the structure or the structural members shall maintain the service requirements.

Verification index—Physically quantifiable index for the required performance

Limit state—The state in which a structure begins to fail to meet the performance requirements

Load—Any action which causes variations in stresses or deformations in structures or members.

Permanent load—Load that acts continuously, or is such that the variations in the magnitude are rare or negligible.

Variable load—Load that varies frequently or continuously and is such that the variations in the magnitude cannot be neglected compared with the permanent load.

Accidental load—Load that occurs rarely during the design life, but has serious consequences when it occurs.

Design load—Value of load that is obtained by multiplying each characteristic value of load by a corresponding load factor.

Characteristic value of load—Value of load determined based on variations, limit states concerned, and combinations of loads, for all loads during construction period and throughout design life of the structure.

Specified value of load—Value of load specified by other design codes or specifications, apart from the characteristic value of load.

Nominal value of load—Value of load not specified in available relevant codes, but is commonly used in practice.

Characteristic value of material strength—Value of material strength which guarantees that the probability of tested strength being below this value is within the specified limit based on statistical interpretation of test results.

Specified value of material strength—Value of material strength that is specified by other design codes or specifications in a manner other than characteristic value of material strength.

Design basic strength—Basic strength for design. For concrete, the characteristic compressive strength may usually be taken as the design basic strength.

Design strength—Value obtained by dividing the characteristic value of material strength by a material factor.

Material factor—Safety factor to consider the unfavorable deviations of material strengths from the characteristic values, differences in material properties between test specimens and actual structures, effect of material properties on the specific limit states, and time dependent variations of material properties.

Load factor—Safety factor to consider the unfavorable deviations of loads from the characteristic values, uncertainty in evaluation of loads, effect of nature of loads on the limit states, and variations of environmental actions.

Structure factor—Safety factor to account for the relative importance of the structure, as determined by the social impact of the structure failing or reaching a particular limit state.

Structural analysis factor—Safety factor to consider the uncertainty of computational accuracy in determination of member forces through structural analysis.

Member factor—Safety factor to consider the uncertainties in computation of capacity

of the member, differences in the design and actual size of the member, and the importance of the member which reflects the influence on the overall structure when it reaches a certain limit state.

Load modification factor—Safety factor to transform specified or nominal values of loads into characteristic values of loads.

Material modification factor—Safety factor to transform specified values of material strength into characteristic values of material strength.

Design response value—A value obtained by multiplying a response value under the design load by a structural analysis factor

Design sectional force—A value obtained by multiplying the sectional force due to a combination of design loads by a structural analysis factor and a design response value obtained by using force as a verification index

Design limit value—A value obtained by dividing the performance of a member or structure calculated by using a material-specific design value by a member factor and a verification limit value set according to the required performance

Design sectional capacity — A value obtained by dividing the sectional capacity calculated by using the design strength of a material by a member factor and a design limit value obtained by using force as a verification index

Linear analysis—Structural analysis based on elastic primary theory assuming a linear stress-strain relationships for the materials used, and neglecting secondary effects of deformations.

Primary reinforcement—Reinforcement calculated and arranged to fulfill requirements of each limit state.

Positive moment reinforcement—Primary reinforcement arranged to resist the tensile force due to positive flexural moment.

Negative moment reinforcement—Primary reinforcement arranged to resist the tensile force due to negative flexural moment.

Distribution reinforcement—Reinforcement that is usually placed at right angles to the primary reinforcement for the purpose of appropriate distribution of stresses.

Shear reinforcement—Reinforcement arranged to resist the shear force.

Stirrup—Transverse reinforcement placed perpendicular or nearly perpendicular to the axis of member enclosing positive and/or negative moment reinforcement.

Bent bar—Reinforcement formed by bending up or down positive or negative moment reinforcement.

Tie—Transverse reinforcement enclosing longitudinal reinforcement at given intervals.

Hoop—Within the tie, the reinforcement enclosing longitudinal reinforcement circularly or elliptically.

Intermediate tie—Transverse reinforcement arranged to intersect the cross section.

Spiral reinforcement—Spirally wound continuous transverse reinforcement enclosing longitudinal reinforcement.

Additional reinforcement—Auxiliary reinforcement provided as a precaution against concentrated stresses caused by loads, or to guard against cracking due to variation of temperature or shrinkage.

Prestressing steel—High strength steel mainly used for prestressing.

Tendon—Single or a bundle of prestressing steel.

Sheath — Tube to form voids in concrete for accommodation of tendons in post-tensioned prestressed concrete members.

Anchorage—Device used to anchor tendons in concrete member.

Coupler—Device to connect one tendon with another.

Fretting fatigue—Accumulated fatigue in wire of tendon due to rubbing or pressing by loading in portions where they are in contact with one another.

Pretensioning system—Prestressing system in which tendons are tensioned before concrete is placed. Tensile stress in tendon is transferred to hardened concrete by bond between tendon and concrete.

Post-tensioning system—Prestressing system in which tendons are tensioned after concrete has hardened. Tensile stress in tendon is transferred to concrete by anchorage at ends.

Effective depth—Distance measured from extreme compression fiber to centroid of positive or negative moment reinforcement.

Tension reinforcement ratio—Ratio of cross-sectional area of primary tension reinforcement to effective cross-sectional area of concrete. The effective cross-sectional area of concrete is defined as the product of effective depth and flange width.

Compression reinforcement ratio—Ratio of cross-sectional area of primary compression reinforcement to effective cross-sectional area of concrete.

Balanced reinforcement ratio—Tension reinforcement ratio where both tensile stress of primary tension reinforcement and compressive strain of concrete at extreme fiber reach the design yield strength and the ultimate compressive strain simultaneously.

Development length of reinforcement—Length of embedded reinforcement required to develop the design tensile stress of reinforcement at the critical section.

Clear distance—Face to face distance between adjacent reinforcing bars, tendons or sheaths.

Concrete cover—Minimum thickness of concrete between concrete surface and surface of reinforcing bars, tendons or sheaths.

Clear span—Distance from face to face of supports for beams or slabs.

One-way slab—Rectangular slab supported by two opposite sides.

Two-way slab—Rectangular slab supported by four sides.

Deep beam—Beam of which depth is relatively greater than a span length.

Corbel—Cantilever attached to column with span to depth ratio being less than or equal to one.

Column—Member oriented vertical or nearly vertical, and the length of which is greater than three times of the least transverse dimension.

Beam element model—A model where a structure is represented as made up of a combination of one-dimensional beam elements

Finite element model—A model where a structure is represented as made up of a combination of two or three-dimensional finite elements

Nonlinear hysteretic model—A model that gives nonlinear hysteresis in the stress-strain relationship of constitutive materials, or in the load-displacement relationship of a member or a structure under reversed cyclic loading

1.4 Notation

In this Specification, the following notations have been used for design calculation for structures.

A : gross area of cross section

A_a : loaded area of bearing force

A_c : area of concrete section

A_m : effective area of concrete for resisting torsion

A_s : area of reinforcement, or area of reinforcing steel in tensile zone

A_{sc} : area of reinforcement required by calculation

A_{ss} : area of structural steel shape, plate or fabrication in tensile zone

A_{tl} : area of longitudinal reinforcement which is effective for resisting torsion

A_{tw} : area of transverse reinforcement which is effective for resisting torsion

A_w : area of shear reinforcement

a_v : distance from loading point to support face

b : width of member

b_e : effective width of member

b_0 : length of short leg of transverse reinforcement

b_w : web width of member

C'_d : design diagonal compressive force per unit width in concrete

c : concrete cover

Δc : construction error in cover

c_s : center-to-center distance of reinforcing steel

d : effective depth of member

d_0 : diameter of concrete cross section enclosed by transverse reinforcement for circular cross section, or length of long leg of transverse reinforcement for rectangular cross section

E_c : modulus of elasticity of concrete

E_p : modulus of elasticity of prestressing steel

E_s : modulus of elasticity of reinforcement or structural steel shape, plate or fabrication

F : load

F_k : characteristic value of load

F_n : specified value of load

F_p : permanent load

F_r : variable load

f : material strength

f'_a : bearing strength of concrete

f_b : flexural strength of concrete

f_{bo} : bond strength between concrete and reinforcing steel

f'_c : compressive strength of concrete

f_k : characteristic value of material strength

f'_{ck} : characteristic compressive strength of concrete, design basic strength

f_{lv} : yield strength of longitudinal torsion reinforcement

f_n : specified value of material strength

f_{pu} : tensile strength of prestressing steel

f_{py} : yield strength of prestressing steel

f_r	:	fatigue strength
f_t	:	tensile strength of concrete
f_u	:	tensile strength of steel
f_{vy}	:	yield strength of steel in shear
f_{wy}	:	yield strength of shear reinforcement or transverse torsion reinforcement
f_y	:	tensile yield strength of steel
f'_y	:	compressive yield strength of steel
h	:	total depth of member
I_e	:	moment of inertia of transformed cross section
I_g	:	moment of inertia of gross cross section
K_t	:	section modulus for torsion
k	:	coefficient to consider bond behavior of reinforcing steel
k_1	:	coefficient for size effect in tensile strength of concrete
k_2	:	coefficient to consider effect of frequency of variable load
k_c	:	coefficient to consider effects of concrete cover and transverse reinforcement on development length of reinforcement
l_0	:	development length of reinforcement
l_d	:	basic development length of reinforcement
l_s	:	additional embedment length at supports or a at point of inflection
M	:	moment
M_{cr}	:	cracking moment
M_t	:	torsional moment
M_{tc}	:	torsional moment capacity of member without torsion reinforcement
M_{tcu}	:	diagonal compressive capacity of web concrete for torsional moment
M_{tu}	:	torsional moment capacity of member
M_{ty}	:	torsional moment capacity of member determined by yield of torsion reinforcement
M_u	:	flexural capacity
N	:	fatigue life, or equivalent number of cycles of fatigue load
N'	:	axial compressive force
N_1, N_2	:	main in-plane forces acting on planar members, N_1 is in tension, not less than N_2
P_e	:	effective prestress force in tendon
p	:	ratio of tension reinforcement
p'	:	ratio of compression reinforcement
p_w	:	ratio of shear reinforcement
R	:	limited value or capacity of cross-section
R_r	:	fatigue capacity of cross-section
r	:	inside radius of bend
S	:	member force
S_p	:	member force due to permanent loads
S_r	:	member force due to variable loads
s	:	spacing of shear reinforcement, torsional reinforcement, or transverse reinforcement
T_c	:	total tensile force in concrete
T_x	:	tensile force in reinforcement in x direction, per unit width
T_y	:	tensile force in reinforcement in y direction, per unit width
u	:	perimeter of bar, or loaded area

u_p : **effective perimeter for resisting punching shears in slab, that is the sum of perimeter of bearing area of concentrated load or reaction and πd. (where, d is effective depth)**
V : **shear force**
V_c : **shear capacity of member without shear reinforcement**
V_{cw} : **shear transfer capacity of member at shear plane**
V_h : **additional vertical force due to variation of effective depth in the direction of member axis**
V_p : **shear force due to permanent loads**
V_{pc} : **punching shear capacity of slab**
V_{pe} : **additional vertical force due to inclination of tendon**
V_r : **shear force due to variable loads**
V_{rp} : **fatigue punching shear capacity of slab without shear reinforcement**
V_{rv} : **shear capacity provided by reinforced concrete section in steel reinforced concrete (SRC) member of cumulative type**
V_s : **shear capacity provided by shear reinforcement**
V_{srv} : **shear capacity of SRC member of cumulative type**
V_{sv} : **shear capacity provided by structural steel shapes, plates or fabrication in SRC member of cumulative type**
V_{wc} : **ultimate diagonal compressive capacity of web concrete in shear**
V_v : **shear capacity**
w : **crack width**
w_a : **limit value of crack width**
z : **distance between resultant compressive force and centroid of tension reinforcement**
α : **coefficient for development of reinforcement to consider effect of concrete cover and existence of transverse reinforcement**
α_c : **angle between compression fiber and member axis**
α_p : **angle between tendon and member axis**
α_s : **angle between shear reinforcement and member axis**
α_t : **angle between tension reinforcement and member axis**
β_d : **coefficient to consider effect of effective depth on shear capacity**
β_n : **coefficient to consider effect of axial force on shear capacity**
β_{nt} : **coefficient to consider effect of axial force on torsional moment capacity**
β_p : **coefficient to consider effect of longitudinal reinforcement on shear capacity**
γ : **rate of apparent relaxation of prestressing steel**
γ_a : **structural analysis factor**
γ_b : **member factor**
γ_c : **material factor for concrete**
γ_f : **load factor**
γ_i : **structure factor**
γ_m : **material factor**
γ_s : **material factor for steel**
δ : **coefficient of variation of test results**
δ_y : **yield displacement**
ε'_c : **compressive strain of concrete**
ε'_{cc} : **compressive creep strain of concrete**

ε'_{cu}	:	**ultimate compressive strain of concrete**
ε'_{cs}	:	**shrinkage strain of concrete**
ε'_{csd}	:	**value for evaluation of increment of crack width due to shrinkage and creep in concrete**
ρ_f	:	**load modification factor**
ρ_m	:	**material modification factor**
σ	:	**standard deviation**
σ'_{cp}	:	**compressive stress in concrete due to permanent loads**
σ'_n	:	**average compressive stress in concrete due to axial force**
σ_{pe}	:	**stress increment in prestressing steel for checking crack width**
σ_{pp}	:	**stress increment in prestressing steel due to permanent loads**
σ_{pw}	:	**tensile stress in prestressing steel as shear reinforcement at yield of other shear reinforcement**
σ_r	:	**variable stress**
σ_{se}	:	**stress increment in steel for checking crack width**
σ_{sp}	:	**stress increment in steel due to permanent loads**
σ_w	:	**stress in shear reinforcement**
σ_{wr}	:	**stress in shear reinforcement due to variable loads**
σ_{wpe}	:	**effective tensile stress in prestressing steel as shear reinforcement**
σ_x	:	**normal stress**
σ_y	:	**stress in direction orthogonal to that of σ_x**
σ_I	:	**diagonal tensile stress**
τ	:	**shear stress produced by shear force and/or torsional moment**
φ	:	**creep factor for concrete**
ϕ	:	**diameter of reinforcement or duct, nominal diameter of reinforcement**

[Commentary] It is preferable to use different symbols for different meanings. In principle, the following definitions have been adhered to:

A	:	area
b	:	width
c	:	concrete cover
d	:	effective depth
E	:	modulus of elasticity
F	:	load
f	:	material strength
I	:	moment of inertia
l	:	span length, development length
M	:	moment
N	:	cycle, axial force
P	:	prestress force in tendon
p	:	reinforcement ratio
R	:	capacity of cross section
S	:	member force
s	:	spacing

u : peripheral length
V : shear
w : crack width
x : distance from support
α : inclination angle with respect to the axis of member
β : coefficient for shear capacity
γ : safety factor, rate of relaxation
δ : coefficient of variation, displacement
ε : strain
ρ : modification factor
σ : stress
φ : creep factor
ϕ : diameter

There may be some cases, when the symbol defined above is used with a different meaning in text, or when new symbols are used. In such cases, the symbols are appropriately explained in the respective sections.

Subscripts used in this specification have the following meanings unless otherwise explained elsewhere:

a : bearing, structural analysis
b : member, balance, bending
bo : bond
c : concrete, compression, creep
cr : crack
d : design value
e : effective, transformed
f : load
g : gross area
k : characteristic value
l : longitudinal
m : material, mean
n : specified, standard, axial
p : prestress, prestressing steel, permanent, punching
r : variable
s : steel, reinforcement
t : tension, torsion, transverse
u : ultimate
v : shear
w : web

y : yield

The subscript, k, represents the characteristic value of load or material strength. The subscript, d, represents the design value of member force or the capacity of member cross-section. However, these subscripts may be omitted when the meaning is obvious.

The Specification follows the sign convention that stresses and strains are regarded positive in tension, and negative in compression. As an exception, the symbol with prime (') stands for compression as positive.

CHAPTER 2 PERFORMANCE REQUIREMENTS

2.1 General

(1) The design life of a structure shall be determined in consideration of required service period of the structure, maintenance methods, environmental conditions, and economy.

(2) For every structure, all performance requirements to meet the intended purpose of the structure during construction and during the design life of the structure shall be specified. As a general rule, performance requirements related to durability, safety, serviceability, restorability, environmental compatibility, landscape compatibility, etc., shall be specified.

[Commentary] (1) For design of a structure, it is necessary to indicate the design life with wide consideration to the purpose of the structure, economically determined period over which the structure is required to be in service, environmental conditions in the neighborhood of the structure. Generally, longer design life is specified, higher durability and fatigue resistance performance will be required.

(2) All of the specified performance requirements must be met throughout the design life of the structure. As a general rule, performance requirements must be specified with respect to durability, safety, serviceability, restorability, environmental compatibility, landscape compatibility, etc. Detailed items related to these performance requirements need to be identified. In order for a structure to meet safety, serviceability, restorability and other performance requirements throughout the design life of the structure, the structure must be free from material deterioration or other types of deformation that could affect those performance requirements due to environmental actions.

In this Specification, the resistance of a structure to deterioration over time is identified as one of the durability-related performance requirements of a structure. It does not include, however, the resistance to time-dependent changes in performance due to repetition of external forces such as variable loads (e.g., fatigue). Instead, such resistance is deemed to be part of safety-related performance against external forces. The required performance of a structure here is the performance of a structure as a whole. In general, the performance of a structure is closely related to the performance of the structural elements constituting the structure. In order to meet the performance requirements for a structure, therefore, it is necessary to give careful consideration to the relationship with the performance of each structural element so that the performance requirements for the entire structure can be met.

2.2 Durability

In this Specification, durability shall mean the resistance of a structure to performance degradation over time resulting from the materials deterioration in the structure under expected deterioration actions.

[Commentary] The durability of a structure should be specified for the purpose of maintaining the required performance such as safety, serviceability and restorability throughout the design life of the structure. Durability, therefore, is not independent of those performance requirements and should be deemed to be the resistance to changes in those performance over time. It is still difficult at present, however, to evaluate performance such as safety, serviceability and restorability as a function of time, and trying to do so is not necessarily economical, either. For this reason, it is common practice to specify the required durability of a structure so that problems due to environmentally induced deterioration of the materials in the structure do not occur during the design life of the structure, and to perform performance verification with respect to safety, serviceability and restorability. This indirectly ensures that the structure meets various performance requirements throughout the design life of the structure.

2.3 Safety

In this Specification, safety shall mean the performance of a structure to prevent risks to users and other people in the vicinity under all expected conditions. Safety includes the structural safety and functional safety of a structure, and performance requirements for both types of safety must be specified.

[Commentary] The safety of a structure can be broadly classified into performance that can be determined by the mechanics of the structure such as failure or collapse due to variable loads or accidental loads such as seismic loads and performance determined by the intended purpose of use or the loss of functionality. In this Specification, as a general rule, performance requirements for both types of safety must be specified.

2.4 Serviceability

In this Specification, serviceability of a structure shall mean the performance that enables users or other people in the vicinity to use the structure comfortably and functional performance required of the structure.

[Commentary] The serviceability of a structure is defined as the performance that enables comfortable use of the structure and functional performance under normal conditions.

Generally, it is good practice to specify such requirements as trafficability, walkability, appearance, noise and vibration with respect to the comfortability of use. With respect to functional performance, it is generally good practice to specify shielding performance and permeability such as watertightness, water permeability, sound-shielding performance, cold insulation and heat insulation, and the performance in preventing serviceability-impairing damage due to such actions as variable loads, environmental factors and accidental loads.

2.5 Restorability

In this Specification, restorability shall mean the performance in restoring the performance of a structure that has degraded because of accidental loads such as seismic loads, and making continued use of the structure possible. Restorability shall be specified in view of the degree of difficulty in repairing the structure and all factors affected by performance degradation.

[Commentary] Restorability is a performance requirement that indicates the degree of difficulty in restoring degraded performance of a structure due to accidental loads such as seismic loads. Usually, civil engineering structures are highly public in nature, and successful implementation and maintenance of their functions greatly affects the daily activities of individuals and social and production activities. This is why restorability is required as one of performance requirements of a structure. Restorability is greatly affected by not only the degree of difficulty in repairing a damaged structure (reparability) but also structural (tangible) measures such as preparing materials for restoring a damaged structure and improving restoration technology and nonstructural (intangible) measures such as establishing an organizational system for restoration efforts. In this Specification, mechanical performance requirements related to the reparability of concrete structures are specified as restorability on conditions that non-reparability-related factors are separately taken into consideration. Accidental actions that could damage structures include earthquakes, wind, fires, etc., but this Specification requires as a general rule that only the influence of earthquakes be taken into consideration. When considering the reparability of concrete structures, it is good practice to determine the performance level according to the magnitude of accidental loads involved, keeping in mind, for example, the state of a structure that can be used without doing repairs and the state of a structure that can be functionally restored in a short period of time.

2.6 Other Performance Requirements

Performance requirements such as environmental compatibility and landscape compatibility shall be specified on an as-needed basis.

[Commentary] In this Specification, performance requirements are concretely specified with respect to durability, safety, serviceability and restorability. When specifying requirements with respect to other performance requirements such as environmental compatibility and landscape compatibility, it is necessary to specify performance requirements, making sure that appropriate verification can be made. Environmental compatibility encompasses compatibility with natural and other environments such as the global environment, the regional environment and the working environment and compatibility with social environments such as landscapes. Environmental compatibility is to be regarded as required performance on an as-needed basis, keeping in mind the goal of mitigating environmental impact. Since, however, it is still difficult at present to verify these performance requirements at the same level as the safety, serviceability and restorability described in this Specification, it is recommended that performance requirements other than these be considered carefully at the stage of structural planning, and the verification be made if necessary.

CHAPTER 3　STRUCTURAL PLANNING

3.1　General

(1) In structural planning for a structure, the type of structure, materials to be used and main dimensions shall be specified so that the performance required of the structure can be met in the most rational way.

(2) Structural planning for a structure shall involve a comprehensive study that takes into consideration such factors as the construction methods, maintenance methods, environmental and landscape impacts, and economy so that the performance requirements specified in this Specification can be met.

(3) In order to draw up a rational structural plan, necessary studies and surveys shall be conducted according to the state of the planned construction site, the size of the structure, etc.

[Commentary]　This Specification requires a method in which the type of structure, materials and main dimensions are determined at the structural planning stage, after performance requirements by which to ensure the use and function of the structure are specified, and then performance verification is made on the basis of the specifications thus determined in order to ascertain that the structure meets the performance requirements. "Structural planning" mentioned in this chapter, therefore, is defined as a stage at which the type of structure, materials and main dimensions are determined after the performance requirements for a structure are specified.

The use and function required for a structure should be preconditions for the structural planning. The structural planning, therefore, must be consistent with the function requirements. The functions of structures are often regulated in accordance with laws, regulations and standards established under those laws and regulations. In the case of road bridges, for example, requirements related to width and number of lanes are stipulated in the Road Structure Ordinance. Because functions like this are obligatory requirements for the structures concerned, it is necessary to apply such requirements appropriately after carefully studying the content and interpretations of the applicable laws, regulations and standards.

Functions of a structure include not only those corresponding to the purpose and requirements for the structure but also those that are additionally required. In the case of bridge structures that forms a grade-separated intersection with a road or railroad, for example, infrastructural facilities such as waterworks, sewers and gas piping may also be installed. A structural plan, therefore, must be developed through thorough consultation with the organizations involved.

In Construction Part of this Specification describes 13 types of concrete that are special in terms of materials used, performance, construction methods, construction environments, etc. If the required performance of a structure cannot be achieved by a conventional method, it may be necessary to use these special types of concrete and new technologies. Special concretes include expansive concrete, lightweight aggregate concrete, continuous fiber reinforced concrete, short fiber reinforced concrete, high-strength concrete, high-fluidity concrete, shotcrete, prepacked concrete, antiwashout underwater concrete, marine concrete, prestressed concrete, steel-concrete composites, and factory products.

(1) and (2) The type of structure, materials and main dimensions must be determined through a comprehensive study in view of such factors as construction methods, maintenance methods,

environmental and landscape impacts, and economy so that the performance requirements (durability, safety, serviceability, and restorability) stipulated in this Specification can be met. It is not an exaggeration to say that structural planning roughly determines construction costs and practically determines future maintenance costs. It is therefore necessary to conduct careful studies, taking into consideration the future maintenance strategy of the structure.

If a sufficient amount of information is available concerning structures of similar types and sizes, the type of structure and main dimensions can be determined on the basis of past project data. Since, however, it is very difficult at the stage of performance verification to change the type of structure and main dimensions determined at the stage of structural planning, it must be ascertained in advance that there are no substantial differences between the preconditions for the past project and those for the structure concerned, and that the application of the past project data does not pose any problem.

If little or no data is available on structures of similar types or sizes, it is desirable that a detailed study be conducted so as to prevent changes in the type of structure or main dimensions at the stage of performance verification.

(3) The planning, design and construction of a structure requires various studies and surveys, and it is necessary to conduct various studies and surveys at the stage of structural planning according to the state of the planned construction site and the type and size of the structure to be constructed. If these studies and surveys are inadequate, the type of structure or other details may prove ill-suited to the site conditions, necessitating major changes in the structural plan. This requires careful attention.

3.2 Studies on Performance Requirements

In structural planning for a structure, consideration shall be given so that the required durability, safety, serviceability and restorability of the structure can be attained and maintained throughout the design life of the structure.

[Commentary] (i) Durability: In order to attain and maintain the required performance of a structure throughout the design life of the structure, it is common practice either to try to prevent material deterioration and deformation in the structure due to environmental actions or to design the structure so as to keep material deterioration, if any, at a minor level so that structural performance degradation does not result. In order to do this, it is important that the concrete used in the structure have the required level of strength, durability and quality.

Performance verification as to whether a structure is durable enough to remain free from trouble due to material deterioration throughout the design life of the structure is made in accordance with Chapter 8. Passing this verification is necessary in order to maintain the required level of safety, serviceability and restorability throughout the design life of the structure.

In Chapter 8, performance verification of durability is made using durability-related characteristics of concrete, stress in steel bars, thickness of concrete cover, etc. In cases where a structure is to be constructed in an environment that is highly corrosive to steel or in a highly chemically corrosive environment, careful studies on durability needs to be conducted if attaining the required resistance by the resistance of concrete itself is likely to be difficult and necessary countermeasures are likely to be very expensive.

Recommended countermeasures that can be taken in such cases include the use of rust-proofed steel bars and concrete surface protection. If a structure is constructed in a highly erosive environment such as hot-spring areas or in the vicinity of an acidic river where concrete deterioration cannot be controlled completely, it may be necessary to plan exposure tests using concrete specimens in a real exposure environment in order to verify the durability of concrete.

(ii) Safety: Chapter 9 requires as a general rule that the safety of a structure be verified by ascertaining, for example, that no structural member reaches the limit state of cross-sectional failure, and the structure does not reach the limit state of stability under the design loads. Because these verifications are made by using detailed information such as dimensions of structural members and steel bar arrangements, the safety of the structure is examined in detail at the stage of performance verification.

If a sufficient amount of information is available concerning structures of similar type and size, studies on safety of the structure at the stage of structural planning can be made by referring to past project data. In this case, if the size of the structure, topographical, geological and other conditions at the site of existing structures differs considerably from those of the planned structure, it may be necessary to change the type of structure or the shape and dimensions of the structure determined at the stage of structural planning, causing a major change in the structural plan. It is therefore necessary to carefully study difference in the preconditions between existing structures and the planed structure.

If little or no information is available concerning a structure of the same type and size as the planned structure, it is desirable that general studies on safety of the structure be conducted to make sure that no changes in the type of structure or main dimensions occur at the stage of performance verification.

In order to achieve a higher level of safety of a structure, it is desirable that the structure be designed so as to prevent the entire structure from collapsing even if some of the structural members reach the limit state of cross-sectional failure. This characteristic is called "redundancy" or "structural robustness." For example, a statically indeterminate structure is more redundant than a statically determinate structure.

(iii) Serviceability: Chapter 10 requires as a general rule that serviceability of a structure be verified by ascertaining that neither the structural members nor the structure reaches the limit state of serviceability under the design loads. Commonly used verification indices for comfortability and functionality include crack width, displacement, deformation, stress, acceleration and vibration. These indices are determined at the stage of design of the structure because they are calculated by using such values as dimensions of structural members and steel bar arrangements.

A study on serviceability needs to be conducted at the stage of structural planning if application of waterproofing or other countermeasures becomes obviously necessary from the initial stage of construction and such control measures are likely to be very expensive in case when a structure is to be constructed in an environment that is highly corrosive to steel or when a high level of water-tightness is required.

(iv) Restorability: Chapter 2 defines the restorability of a structure as the performance related to the degree of difficulty in restoring a function degraded by accidental loads such as seismic loads.

Restorability is a performance requirement of civil engineering structures and is greatly affected by not only the degree of difficulty in repairing a damaged structure (reparability) but also the state of implementation of structural measures such as preparation of materials for restoring a damaged structure and developing restoration technologies and nonstructural measures such as establishing an organizational system for restoration efforts.

Chapter 11 requires that performance verification be made by specifying a mechanical limit state for concrete structures. This performance verification is supposed to deal mainly with the reparability of concrete structures. The determination of limit states of this type is predicated on the implementation of structural and nonstructural measures for a damaged structure. In structural planning, therefore, it is necessary to take into consideration both the structural and nonstructural aspects on the restoration of civil engineering structures.

The reparability of a structure is affected by the type of structure. It is necessary, therefore, to select a type of structure that makes repair and functional restoration as easy as possible in the event of structural damage, taking into account such factors as the environment for restoration activities at the construction site. For example, in the case of a structure that is required to remain usable even during an earthquake or even immediately after an earthquake, it is good practice to think of using such methods of controlling structural damage as seismic base isolation or vibration mitigation system. It is also necessary to see to it that the center of stiffness of the structure coincides with the center of action of the loads as much as possible so that horizontal torsion of the structure is minimized.

Regardless of the type of structure, if a structure is damaged, the time and costs required for repair vary considerably depending on not only the degree of damage but also the position of damage in the structure. It is therefore desirable that possible damage areas be located in easy-to-access areas where inspection and repair can be performed easily.

For details on seismic structural planning, refer to Section 11.1.

3.3 Studies on Construction

In structural planning, restraints related to construction shall be taken into consideration.

[Commentary] If a structure is to fulfill its functions and retain the required performance, the structure needs to be constructed so that the conditions indicated on the design drawings and other documents can be met. In order to do this, it is necessary to develop a structural plan in due consideration of restraints associated with construction.

Since the Hyogoken Nanbu Earthquake of 1995, there has been a tendency toward the complication of reinforcement arrangements in cross sections of structural members. Mainly by reason of economy, there have also been cases, such as pylons of concrete cable-stayed bridges, in which the cross sections of structural members were made smaller by using high-strength materials such as high-strength concrete. In such cases, careful consideration needs to be given to concreting, and it is necessary to think about ways to ensure successful concreting such as using high-fluidity (self-compacting) concrete. Depending on the configuration of the structure to be constructed, there are cases where the complication of the fabrication and erection of formwork and reinforcement results in lower construction accuracy so as to make it difficult to attain the required quality of concrete and the required performance of the structure or where the date of completion be delayed because of the needs to attain the required construction accuracy. In order to secure the required accuracy of structures and rationalize the construction process, therefore, it is necessary to think of employment such measures as using large steel formwork or prefabricated reinforcement cages at the stage of structural planning.

There are also cases where the type of structure is determined by the construction methods. For example, if supporting works are difficult because of topography at the construction site of a bridge at a ravine in a mountainous region, the adoption of a precast concrete box girder structure to be constructed by the cantilever erection method may be considered. In the case of a bridge to be constructed on flat land, the adoption of a precast concrete slab girder structure to be constructed by movable supporting methods may be considered.

Thus, the type of structure may be determined according to such restraints as topography at the construction site, and there are also cases where a particular type of structure is adopted and, for example, the use of special concrete is required or formwork and reinforcing work methods are specified. The structural plan, therefore, needs to be drawn up taking into consideration such construction-related restraints.

3.4 Studies on Maintenance

Structural planning shall take into consideration such factors as the degree of importance of the structure to be constructed, the design life of the structure, service conditions, environmental conditions and the degree of difficulty in maintenance so as to facilitate in-service maintenance.

[Commentary] As indicated in the Maintenance Part of this Specification, many inspections need to be conducted for every in-service structure including the initial inspection conducted when maintenance begins, daily inspections and periodic inspections conducted on a regular basis, detailed inspections conducted to evaluate various states in details, and special inspections conducted when the structure has been subjected to accidental external forces due to natural disasters or other causes. Depending on the method of inspection used, inspection may be highly costly and labor-consuming. Also, there may be cases where different repair, strengthening and reconstruction measures are taken for a structure according to the degree of performance degradation of the structure. In such cases, the costs of countermeasures might be huge depending on the degree of performance degradation. In structural planning, therefore, it is desirable that the type of structure and materials to be used be determined so that maintenance activities during services can be performed efficiently, and the cost of countermeasures can be minimized.

Because maintenance-related problems associated with bridges often occur at expansion joint devices or bearings, it is desirable, for example, that the adoption of a continuous rigid-frame structure, which is relatively free from those devices, be considered.

If the degradation of a structure in a harsh environment and huge costs of countermeasures are expected as when a structure is to be constructed at an offshore site, it is desirable, besides the selection of an appropriate type of structure, that studies be conducted on durability enhancement measures such as surface treatment of concrete, rust-proofing treatment of steel bars and cathodic protection to consider maintenance options in advance in order to reduce maintenance costs. If it is thought certain that such maintenance measures need to be taken while the structure is in service, it is good practice to take some kinds of measures in advance at the stage of structural planning in order to facilitate maintenance measures during services. Such measures are effective in reducing maintenance costs.

Clearly indicating a maintenance strategy for the planned structure at the stage of structural planning will contribute to rationalization of maintenance because the information thus identified can be reflected into the maintenance plan drawn up before the commencement of the maintenance stage. It is desirable, therefore, that a prototype of the maintenance plan be developed at the stage of structural planning.

3.5 Studies on Environmental and Landscape Compatibility

Structural planning shall take into consideration the influence of concrete structures on the natural, social and other environments and landscapes.

[Commentary] A concrete structure affects the natural, social and other environments and landscapes at various stages such as the manufacture of materials for concrete structure and the construction, the service and maintenance of the structure. It is necessary to consider the influence on the environment and landscapes at each of those stages, paying attention to factors causing such influence.

When evaluating the influence on natural environments, it is necessary to take into consideration impacts on discharge of greenhouse gases, air pollutants and water and soil pollutants, the consumption of resources and energy, and discharge of waste. For example, in order to evaluate the influence associated with water quality when an underwater concrete structure is to be constructed under strict conditions imposed for the purpose of preventing water contamination, it is necessary to consider the use of antiwashout underwater concrete.

The influence of the structure on natural landscape also needs to be taken into consideration. Civil engineering structures are part of the infrastructure that exists over a long period of time after they are constructed, and those structures themselves can affect landscapes. Since the appearance is greatly dependent on the type of structure, thorough studies needs to be conducted at the stage of structural planning so that there is no adverse effect on natural landscape including the structure itself. To do this, it is necessary to evaluate, in advance, the influence of the construction of the planned structure on the natural landscape at and around the construction site and to draw up a plan so that the completed structure will harmonize with the natural landscape.

From the viewpoint of effective use of resources, efforts are underway to promote reuse of waste generated by concreting work such as waste wood, metal scraps and concrete fragments under the Act on the Promotion of Recycling of Resources (Recycling Act). Waste wood and concrete fragments are among the resources covered by the Recycling Act. It is therefore necessary reuse these resources for producing recycled aggregate concrete or for other applications in order to contribute to environmental conservation. For further information on this, refer to the Recommendations on Design and Construction of Recycled Aggregate Concrete Structures Using Concrete from Demolished Power Plant Structures.

It is also necessary to take into account the influence of noise and vibration during the construction of a concrete structure on local environments around the construction site and the working conditions for construction workers. There is a need for the management of the construction site in accordance with the Noise Regulation Act and the Vibration Regulation Act and the use of low-noise and low-vibration vehicles and equipment.

In the area of environmental conservation, laws such as the Basic Environment Act supplemented by environmental laws including Cabinet orders and ordinances and the Industrial Safety and Health Act have been enacted in order to contribute to healthy and cultured living of the people and the welfare of humankind. Naturally, environmental studies at each stage of activities related to concrete structures must be conducted in accordance with those laws and regulations. Within the framework of comprehensive evaluation of environmental impact, however, it is also necessary to take into consideration all factors affecting the environments. This makes it necessary not only to conduct studies in accordance with those laws and regulations but also to evaluate the influence on global environments, regional environments and working conditions at each stage of the design life of the structure.

For information on environmental performance verification, refer to the Recommendations for Verification of Environmental Performance of Concrete Structures.

3.6 Studies on Economical Aspects

As a general rule, structural planning for a structure shall take into consideration economy from the viewpoint of life cycle costs of the structure.

[Commentary] In structural planning, it is very important to select a type of structure, dimensions of structural members, materials to be used, etc., that are economical. Careful studies are required with regard to economical aspects because total costs of the project will be dominantly determined at the stage of structural planning.

The conventional approach to the evaluation of economy of a structure focused mainly on initial the construction costs. In view of growing stock of infrastructure and increase in maintenance costs, however, it is necessary to evaluate economy from the viewpoint of not only the initial construction costs but also the life cycle costs taking maintenance and reconstruction into account.

Structures need to be repaired periodically and reinforced or reconstructed if they have become unable to meet the performance requirements. Maintenance and other costs required after the completion of construction, therefore, are substantial. When a structure is reconstructed, it is not uncommon that the costs of reconstruction become greater than the initial construction costs, though depending on construction-related restraints. At the stage of structural planning, therefore, it is important not only to evaluate the initial construction costs but also to estimate the future maintenance costs and reconstruction costs and to make evaluation taking those costs into account.

Because structural details such as dimensions of structural members have not yet been determined at the stage of structural planning, construction, maintenance and other costs cannot be evaluated accurately. It is also necessary, therefore, to refer to past project data. Since, however, the estimated costs may differ considerably from the actual costs if the preconditions for a past project and those for the planned structure greatly differ, the preconditions for the past project need to be examined carefully.

CHAPTER 4 RULES FOR PERFORMANCE VERIFICATION

4.1 General

(1) As a general rule, the performance verification of a structure shall verify that a structure or its member having the structural details, such as configuration and dimensions, assumed at the design stage does not reach the limit state specified to meet the required performance during construction and during the design life of the structure.

(2) Generally, the limit state shall be specified for durability, safety, serviceability and earthquake resistance. The limit state of earthquake resistance shall be comprehensively specified in view of the safety of the structure during an earthquake and post-quake serviceability and restorability.

(3) In performance verification of a structure, the influence of changes in performance over time due to environmental actions may be ignored if the requirements in Chapter 8 and Chapter 12 are met.

(4) As a general rule, performance verification of a structure shall be performed by identifying an appropriate index for verification and comparing its limit value with the response value.

[Commentary] (1) As a principle of this Specification, required performance of a structure should be declared clearly at first. Thereafter, an equivalent limit state corresponding to each performance requirement should be specified. When a structure, or a part of it, reaches a certain limit state, its serviceability may suddenly lose, or it may even reach failure in some cases. Then, the structure cannot perform its function nor meet the performance requirements because of various defects. To prevent those situations, performance verification of the structure shall be done by examining the limit state. In setting a limit state, an index representing the state of a structure, member or material should be selected, and then its limit value set in accordance with the required performance. Performance verification is done by examining if its calculated response value under given actions exceeds the limit values or not. The limit values should be set taking into account reliability of analysis methods and models employed in calculating the response value.

In the case that a total structural system is not simple, consisting of several structures, a process with a possibility that the structural system does not meet the requirements is selected. Required performance is set for each constituent structure, and thereafter, the limit state may be set for each element in the structural system. In this case, the procedure to determine the limit state should be shown in a hierarchy way. Recommendations listed in Chapter 1, which show a series of procedures from embodiment of required performance to determination of the limit state of member, can be referred to.

(2) Generally, the limit state is to be determined for durability, safety, serviceability and earthquake resistance. Earthquake resistance is defined as a performance requirement of a structure against seismic actions, and the limit state is comprehensively determined in view of the safety of the structure during an earthquake and its post-quake serviceability and restorability.

It is good practice to define the limit state for safety on the basis of mechanical properties as follows. Other types of limit states than the mechanical safety need to be determined separately according to the functions of the structure. These include performance requirements from the viewpoint of the users of the structure such as trafficability and walkability and performance requirements associated with disasters involving third parties resulting from structural faults such as falling of cover concrete.

1) Cross-sectional failure:

Defined as performance of a structure in retaining its load-carrying capacity under any loads that can occur during the design life of the structure.

The performance of a structure as a whole in resisting failure is closely related to the state of the members constituting the structure. If a structure consists of two or more members, what needs to be done in performance verification for safety is to ascertain that the structure as a whole does not fail even after one or more of the structural members fail. In this Specification, in order to be conservative in verifying the safety of a structure, Chapter 9 describes a concrete verification method to be used when the failure of a structure is defined as the failure of one of the structural members.

2) Fatigue failure:

Defined as performance of a structure in retaining its load-carrying capacity under repetition of any variable loads that can occur during the design life of the structure.

3) Stability of a structure:

Defined as performance of a structure in retaining stability in spite of deformation, displacement, mechanisms of deformation or displacement of foundation structures, etc., under any actions that can occur during the design life of the structure.

With respect to serviceability, it is generally recommended that the limit states described below be set as functional limit states determined by comfortability of use and other functions of a structure. Limit values for these limit states are to be determined according to the purpose and functions of the structure.

1) Appearance:

Defined as performance that ensures that cracks in concrete, soiled concrete surfaces, etc., neither make people uneasy or uncomfortable nor prevent the use of the structure.

2) Noise and vibration:

Defined as performance that ensures that noise and vibration generated from the structure neither have adverse effects on the environments nor prevent the use of the structure.

3) Trafficability and walkability:

Defined as performance that ensures that vehicles and pedestrians can move and walk comfortably

4) Watertightness:

Defined as performance of a watertight concrete structure that ensures that it does not lose its functionality due to water or moisture permeation

5) Resistance against damage:

Defined as performance that ensures that a structure is not damaged by such causes as variable loads and environmental actions so that the structure is not rendered unusable

Table C4.1.1 shows examples of the relationship among limit states, verification indices and loads to be considered for each performance requirement specified in this Specification. Specific settings for earthquake resistance are indicated in Chapter 11.

Table C4.1.1 Examples of relationship among limit state, verification index and load for each performance requirement

Required performance	Limit state	Verification index	Design load to be considered
Safety	Cross-sectional failure	Force	All loads (maximum value)
	Fatigue failure	Stress, force	Cyclic load
	Displacement, deformation, mechanisms	Deformation, deformation of foundation structure	All loads (maximum value), accidental loads
Serviceability	Appearance	Crack width, stress	Load of a magnitude whose frequency of occurrence is relatively high
	Vibration	Noise, vibration level	Load of a magnitude whose frequency of occurrence is relatively high
	Comfortability of vehicle ride, etc.	Displacement, deformation	Load of a magnitude whose frequency of occurrence is relatively high
	Watertightness	Water permeation through the structure Crack width	Load of a magnitude whose frequency of occurrence is relatively high
	Resistance against damage (maintenance of functionality)	Force, deformation, etc.	Variable load, etc.

(3) The basic rule when verifying the required performance of a structure is to take into consideration changes in performance over time during the design life of the structure. In this Specification, however, the safety, serviceability and earthquake resistance described in this Specification may be verified without taking into account deterioration of the constituent materials during the design life if the requirements for the verification of durability described in Chapter 8 and the requirements for the verification concerning initial cracks described in Chapter 12 are met. It is necessary to estimate the degree of safety by assuming structural member details, the accuracy of reinforcement locations, the variability of the mechanical properties of materials used, etc.

(4) In order to rationally verify the performance of a structure, the basic rule is to compare limit values with response values by using verification indices that can directly express the performance to be considered. On the other hand, it is a prerequisite that performance requirements such as environmental compatibility and landscape compatibility are verified by an appropriate method, if possible. Landscape compatibility, for example, needs to be verified by using structure models, perspective drawings, etc., having structural details satisfying the verification criteria for durability, safety, serviceability and earthquake resistance.

4.2 Prerequisite for Verification

The performance verification of a structure in accordance with this Specification assumes, as a prerequisite, that the structural detail requirements for performance verification specified here, other structural detail requirements, the standard construction method requirements and standard placeability requirements in accordance with Construction Part of this Specification, and the maintenance procedure requirements specified in Maintenance Part of this Specification are met.

[Commentary] The methods of performance verification described in this specification are based on structural mechanics of structures and materials, which generally have some assumptions regarding deformational consistency of concrete and reinforcement and local stress states. Thus, general requirements for structural details are specified to realize these assumptions. Otherwise, accuracy of the verification decreases and the range of application is restricted. If verification methods would be more applicable in future, fewer requirements on structural details would be enough.

In this Specification, safety margin is considered on the basis of the standard construction method and concrete placeability indicated in Construction Part of this Specification. Actual construction methods and concrete placeability (expressed in terms of workability, such as filling ability, pumpability and setting characteristics, and strength during construction) are determined on the assumptions that the construction plan is drawn up and the materials design is made in accordance with Construction Part of this Specification so that the quality of concrete is equivalent to or better than that specified in this Specification. It is also assumed that the maintenance procedure requirements specified in Maintenance Part of this Specification are met so that the structural performance specified in Design Part of this Specification can be kept.

4.3 Verification method

(1) The verification of a limit state shall be made by the methods specified in Chapters 8, 9, 10, 11 and 12, after calculating response values by the methods specified in Chapter 7 by use of characteristic values of material strengths and loads, and the safety factors specified in Section 4.5.

(2) In general, verification shall be made by using Eq.(4.3.1):

$$\gamma_i \cdot S_d \,/\, R_d \leqq 1.0 \qquad (4.3.1)$$

where S_d : design response value

R_d : design limit value

γ_i : structure factor as per Section 4.5

[Commentary] (2) In general, when verifying a limit state, it is necessary to consider the state at the end of the design life in view of the influence of changes in performance over time and use Eq.(4.3.1). Eq.(4.3.1) shows the case where the limit value of performance, R_d, is regarded as the lower limit value. If R_d represents the upper limit value, the direction of the inequality sign and the treatment of safety factors differ from those in Eq.(4.3.1).

4.4 Calculation of Response Values and Limit Values

(1) In principle, the function to calculate a response value shall earn the mean value of it, when loads, material characteristics and stiffness of members are actual values.

(2) In principle, the function to calculate a limit value of performance shall earn the mean value of it when material characteristics and stiffness of member are actual values.

[Commentary] Here given is a principle for functions to calculate response values and limit values. When a new method for calculating limit values of performance is proposed by new findings, the equation shall conform to this principle, that is, to relate most expected mean values. In addition, it is required to offer the appropriate member factor associated with accuracy of the equation and reliability of the analytical values.

4.5 Safety Factors

(1) Safety factors are material factor, γ_m, load factor, γ_f, structural analysis factor, γ_a, member factor, γ_b, and structure factor, γ_i .

(2) Material factors, γ_m, shall be determined considering the unfavorable deviations of material strengths from the characteristic values, the difference in material properties between test specimens and actual structures, the effects of material properties on the specific limit states, the time dependent variations of material properties, etc.

(3) Load factors, γ_f, shall be determined considering the unfavorable deviations of loads from the characteristic values, the uncertainties in evaluation of loads, the change in loads during the service life, the effect of loads on the limit states, etc.

(4) Structural analysis factors, γ_a, shall be determined considering the uncertainties of computational accuracy in determination of response values through structural analysis, etc. The value of structural factor, γ_a, may be taken as 1.0.

(5) Member factors, γ_b, shall be determined considering the uncertainties in computation of limit values of performance of members, the effect of dimensional errors of members, the importance of members among the entire structure when it reaches a certain limit state, etc. The value of member factor, γ_b, is determined corresponding to each equation for limit value of performance.

(6) Structure factors, γ_i, shall be determined considering the importance of the structure, as determined by the social impact when the structure would reach the limit state. The value of structure factor, γ_i, may be in between 1.0 and 1.2.

(7) If a nonlinear analysis method is used for performance verification, safety factors shall be determined appropriately according to the verification indices used in the analysis in view of the meaning of the safety factors.

(8) Safety factors used for verification of earthquake resistance described in Chapter 11 shall be determined appropriately according to the verification method in view of the meaning of the safety factors.

[Commentary] (1) Verification of safety based on the limit state of cross-sectional failure employs five safety factors - two of them, γ_f and γ_a, are related to calculation of response values, another two, γ_m and γ_b, are associated with calculation of limit values of performance, and the other, γ_i, is used when comparing the response value with the limit value of performance. The concept of these safety factors can be applied to other limit states. Fig. C4.5.1 shows a process for performance verification using the safety factors described in this Specification.

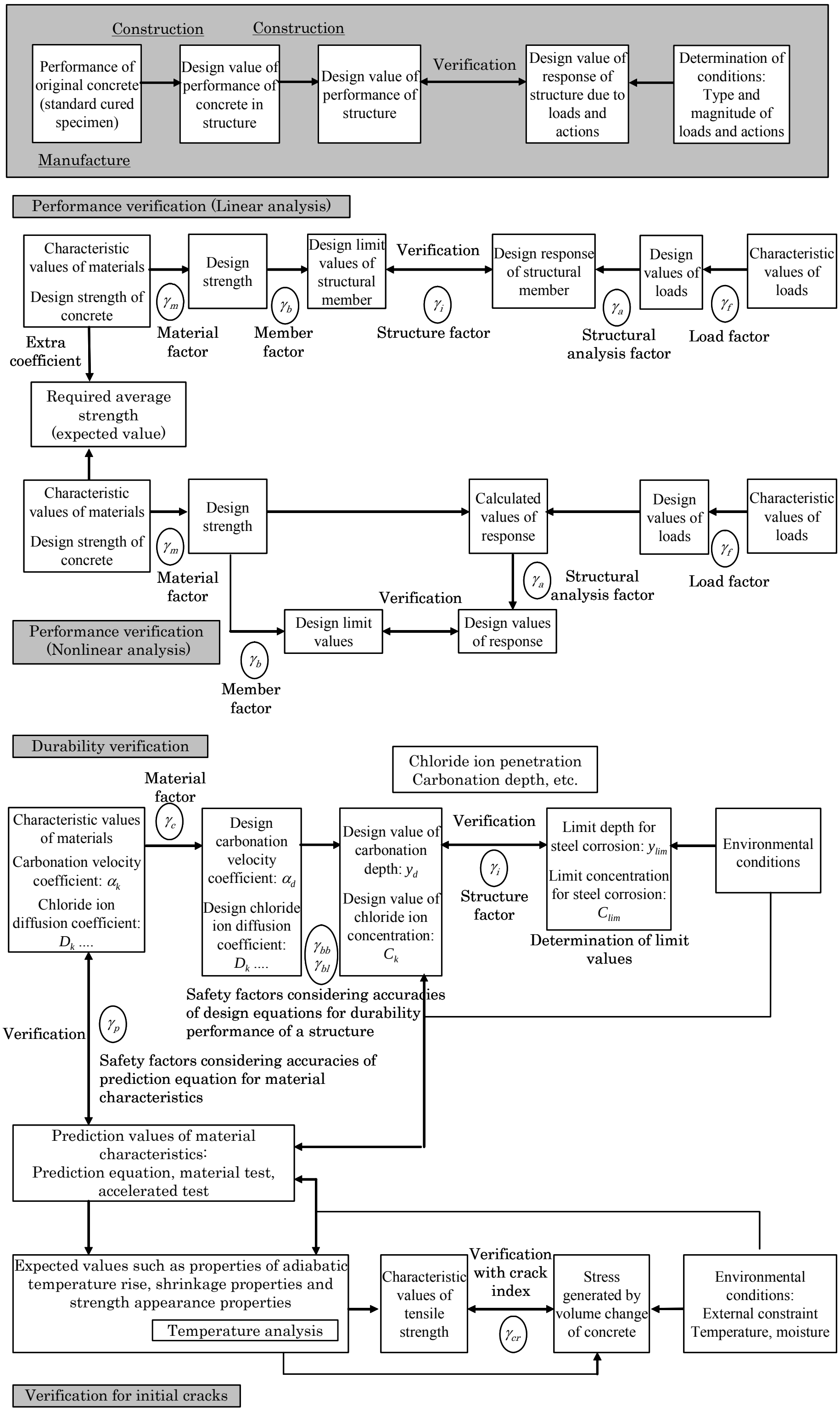


Fig. C4.5.1   Safety factors used for performance verification

(3) The value of load factor, γ_f, may depend upon the type of load, the limit state concerned and the effect of the load on calculated response values in the target cross-section. It should be noted that the maximum load does not always create the critical condition, but in some structures, the minimum applied load may become dominant in design.

(4) The function to calculate the response values should earn the mean value of it in principle according to Section 4.4. The deviation of the response values calculated by the function should be taken into account by the structural analysis factor, γ_a.

(5) The importance of the member should be judged from its role in the entire structure, in such a manner as primary members are more important than secondary ones. The member factor, γ_b, may be used to control the safety levels against flexural and shear failure modes separately, and to specify the member to be failed in a structure. The function to calculate the limit values of performance should earn the mean value of it in principle according to Section 4.4. The deviation of the limit values calculated by the function should be taken into account by the member factor, γ_b.

(6) The structure factor, γ_i, which is associated with the importance of the structure, should account for the social impact of violation of the limit state. This impact could be in view of disaster mitigation and the cost of reconstruction or repair.

Table C4.5.1 shows the contents considered by each safety factor for verification of safety against cross-sectional failure.

Table C4.5.1 Details covered by safety factors

Contents being considered	Value / Safety factor
Limit values of performance	
1. Variation of material strength	
(1) Where can be evaluated by records of material tests	Characteristic value, f_k
(2) Where can not be evaluated by records of material tests (Consider such conditions as lack or inadequacy of the records, degree of quality control, differences in material strength between test specimen and actual structure, time-dependent variation, etc.)	Material factor, γ_m
2. Degree of influence on limit state	
3. Uncertainty in calculation, dimensional errors, importance of member, and failure mode	Member factor, γ_b
Response values	
1. Variation of load	
(1) Where can be evaluated by statistical records of loads	Characteristic value, F_k
(2) Where can not be evaluated by statistical records of loads (Consider such conditions as lack or inadequacy of the records, variation of load during service life, uncertainty in evaluation of load, etc.)	Load factor, γ_f
2. Degree of influence on limit state	
3. Uncertainty in structural analysis	Structural analysis factor, γ_a
Importance of structure, influence on society when the structure reaches a limit state	Structure factor, γ_i

Table C4.5.2 Standard values of safety factors

Safety factor / Required performance (Limit state)		Material factor, γ_m		Member factor, γ_m	Structural analysis factor, γ_m	Load factor, γ_m	Structure factor, γ_m
		for concrete γ_m	for steel γ_m				
Safety (Cross-sectional failure)*1		1.3	1.0 or 1.05	1.1 - 1.3	1.0	1.0 - 1.2	1.0 - 1.2
Safety (Cross-sectional failure, collapse)*2 Earthquake resistance (II, III)*2	Response value	1.0	1.0	-	1.0 - 1.2	1.0 - 1.2	1.0 - 1.2
	Limit value	1.3	1.0 or 1.05	1.0, 1.1 - 1.3	-	-	
Safety (Fatigue)*1		1.3	1.05	1.0 - 1.1	1.0	1.0	1.0 - 1.1
Serviceability*1 Earthquake resistance (I)*1		1.0	1.0	1.0	1.0	1.0	1.0

Remark) *1: Calculated by linear analysis *2: Calculated by non-linear analysis

The value of safety factors should be given for each limit state, and they are not necessarily the same for different limit states. Although each safety factor covers the uncertainty of individual event separately, the influence of each safety factor may be considered collectively.

The standard values of safety factors that may be used when performance verification is made according to this Specification together with inspection system in accordance with Construction Part of this Specification are shown in Table C4.5.2.

(7) Fig. C4.5.2 shows typical safety factors for verification against limit state of cross-sectional failure when linear analysis is used. When nonlinear analysis is used to determine the limit state of cross-sectional failure of a structure, indices other than sectional forces may be used for performance verification. Examples are an average in-plane strain in a shell structure in shear and an average curvature of a member in bending. Here, S is a function of both the design values of material property and the design values of loads. In this case, it is recommended that the safety factors be set for the indices and limit values related to failure as shown in Fig. C4.5.3. To determine design response values, uncertain factors considered by both the structural analysis factor and the member factor in Table C4.5.1 are taken into consideration. Examples are accuracy and reliability of analysis including setting of boundary conditions, and variability and dimensional errors of members and structures. When determining limit values, R_d, it is good practice to determine it conservatively, taking into consideration the uncertain factors considered by the member factor indicated in Table C4.5.1.

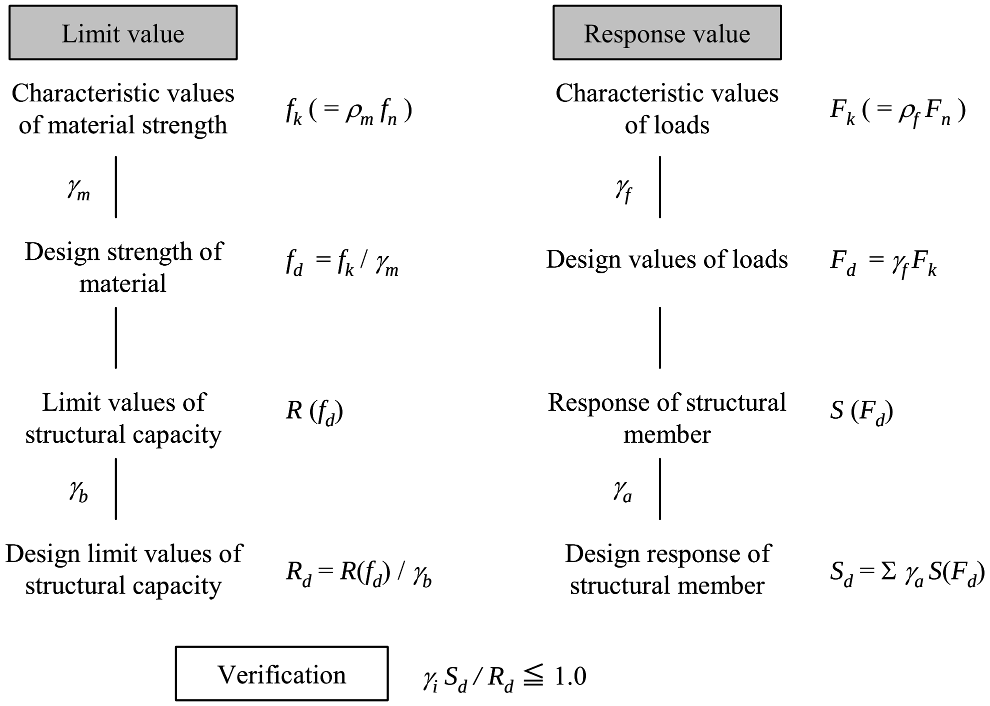


Fig. C4.5.2 Safety factors in the case where linear analysis is used

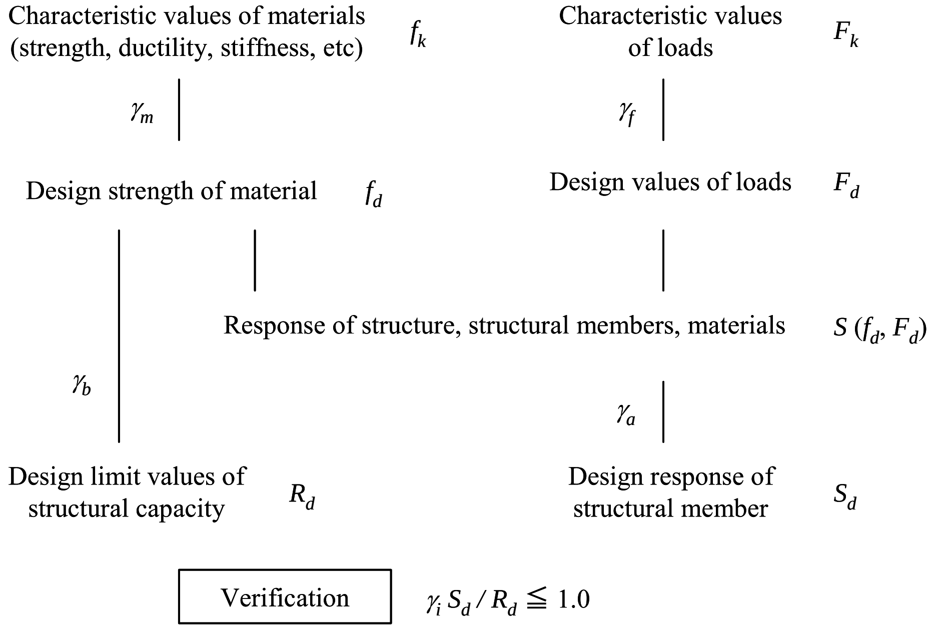


Fig. C4.5.3 Safety factors in the case where nonlinear analysis is used

(8) Safety factors used for verification of earthquake resistance are to be determined in accordance with Chapter 11.

4.6 Modification Factors

(1) The Specification defines two modification factors namely material modification factor, ρ_m, and load modification factor, ρ_f.

(2) Material modification factor, ρ_m, shall be determined considering the difference in material strength between characteristic value and specified value.

(3) Load modification factor, ρ_f, shall be determined for each limit state considering the difference in load between characteristic value and specified or nominal value.

[Commentary] When specified or nominal values are defined on material strength and load instead of characteristic values, the specified or nominal values can be converted into those characteristic values by the modification factors. The value of load modification factor, ρ_f, can be given depending on each limit state. The modification factors are specified as a tentative measure until specified or nominal values are to be indicated according to the definition of characteristic values in the Specification.

4.7 Documentation of Design Calculations

(1) The design calculations clearly showing required performance and associated limit states of members or structures for verifying durability, safety, serviceability, earthquake resistance, etc. shall be preserved with calculation processes of their examination.

(2) The calculated results shall be precise in two significant digits at final stages in principle.

[Commentary] (2) The calculated results at final stages above refer to the value obtained from Eq.(4.3.1). In order to report these results with two significant digits, computations for response values, limit values, stress and strength are generally required to be carried out with three significant digits.

4.8 Design Drawing

(1) Design drawings shall indicate not only structural and reinforcement steel details but also basic information on design calculation, construction and maintenance conditions, etc., listed below. The information in Items 16 to 20 should be indicated for reference purposes.

1) Design life and environmental conditions

2) Characteristic values of loads and combinations of design loads

3) Safety factors

4) Required performance and verification results

- Design response values

- Design limit values

5) Characteristic values of materials used (concrete, steel)

6) Type and quality of steel

7) Concrete cover over steel in all members and construction accuracy

8) Type of reinforcing bar joints and joint locations or regions in which joints may be provided

9) Tensioning force at tension end, elongation and tensioning procedure

10) Information necessary for construction and maintenance

11) Name of structure and place of use

12) Signature of responsible engineer

13) Date of design

14) Scales, dimensions and units of measurement

15) Name of applied standard

16) Type of cement

17) Maximum size of coarse aggregate

18) Cement content

19) Slump or slump flow of concrete

20) Water/cementitious material ratio

21) Air content

(2) Design drawings shall be stored along with construction records throughout the period in which the structure remains functional and in service.

[Commentary] Design drawings may be considered to be the only means through which the designer communicates his idea(s) to the agency responsible for construction and maintenance of the structure. This Specification requires as a general rule that mixture proportion of concrete shall be determined so that the requirements of characteristic values of material properties of concrete indicated in the design drawings are met during construction. To this end, the design drawings must indicate all characteristic values of concrete determined at the design stage, and these drawings must be turned over to the construction agency. The characteristic values of concrete mentioned in Item (1) 5) include material strength, shrinkage strain measured by the JIS test method, shrinkage of members, creep, carbonation rate coefficient, chloride diffusion coefficient and relative dynamic modulus of elasticity. In view of the cases, however, where field-proven mix proportion of concrete and simplicity of mix design during construction, it is also required that other information including type of cement, maximum size of coarse aggregate, cement content, slump or slump flow and water/cementitious material ratio be also indicated in the design drawings for reference purposes. It is also important to clearly indicate the requirements and other information that may be useful during construction such as the timing of formwork and supporting removal, the compressive strength of post-tensioned prestressed concrete at which prestress may be given, and the thermal cracking index.

For durability enhancement of concrete structures, factors such as the cover to reinforcement and the quality of concrete cover, cracks assumed at the design stage, are of importance. Details pertaining to these are also needed for inspection at the end of construction when the structure is handed over to the management agency for actual operation and also during in-service maintenance. In order to convey the relevant considerations in detail made at the design stage to the responsible agency for construction and maintenance of the structure, it is necessary to describe clearly in design drawings the information on concrete cover over steel, construction accuracy of steel bars arrangement, design crack widths in all parts. It is further desirable that comprehensive drawings for the complicated parts be prepared showing the details for arrangement of reinforcement, sheaths, anchor-bolts, and so on. Such information should be utilized when checking whether the concrete placed in these parts can be adequately compacted. It is also requested that all other useful information for construction and maintenance such as calculation details, member force diagrams, agendas of meetings, data and relevant records selected from design documents to determine the structural details, are also appropriately preserved as well as the design drawings. It may be useful to record the following items other than those indicated above, if necessary.

1) List of materials used

2) Detail diagrams of concrete cover part

3) Detail diagrams where steel bars, sheaths and anchor bolts are complicatedly arranged

4) Order of concrete placement

5) Position of construction joints

6) Supporting

7) Ground conditions and bearing capacity of supporting piles

8) Geological profile and N-value

CHAPTER 5 DESIGN VALUES FOR MATERIALS

5.1 General

(1) Quality of concrete or steel is represented by not only compressive or tensile strength but also other material properties such as other strengths, modulus of elasticity or deformation characteristics, thermal characteristics, durability and water tightness. The effect of loading rate on strength and deformation characteristics should be considered if necessary.

(2) The characteristic value for material strength f_k shall be determined, taking into account the variation in tested values, such that most of the tested values exceed this characteristic value.

(3) The design strength of material f_d shall be obtained by dividing the characteristic value for material strength f_k by a material factor γ_m.

(4) When a specified value for material strength f_n is determined independently from its characteristic value, the value of f_k shall be obtained by multiplying the specified value by a material modification factor ρ_m.

[Commentary] (1) Various types of concrete are used in concrete structures ranging in compressive strength from 18 to 100 N/mm^2 even for concrete using ordinary Portland cement. Concrete could be classified according to its properties – for example, air-entrained concrete, super-plasticized concrete, self-compacting concrete, expansive concrete, high strength concrete, ultra high-early-strength concrete, ultra-rapid-hardening concrete, anti-washout concrete and steel fiber reinforced concrete. When considering the methods of construction, there are varieties of concrete such as that cast in bentonite, tremie concrete, prepacked concrete, shotcrete and precast concrete, in addition to the concrete cast in the conventional manner.

Appropriate type and quality of the concrete are necessary to be applied for concrete used in structures or members, in consideration of its purpose for use, environmental condition, design life, construction condition and so on.

Reinforcing bars, prestressing steel and rolled sections of structural steel used in composite steel-concrete construction are the different steels used in concrete structures. Besides these, there are other steels for the anchorage or connection. Most of these steels are standardized by JIS (Japanese Industrial Standard).

In accordance with needs in structural performance verification, quality of concrete is represented by not only compressive strength but also quantities for various material properties. Material properties can be classified into mechanical properties, such as strength and deformation characteristics, physical properties, chemical properties and so on.

Strength characteristics are expressed by strengths under static and fatigue loadings in compression, tension, bond, etc. Deformation characteristics are expressed by time-independent quantities such as modulus of elasticity and Poisson's ratio, and time-dependent quantities such as creep coefficient and shrinkage strain. There are also mechanical properties expressed using a relationship between two mechanical variables, such as a stress-strain relationship. Fracture energy is used to express the crack resistance and toughness in some cases.

Physical properties include thermal characteristics such as coefficient of linear expansion and specific heat, density, water tightness, air tightness and so on. Among these, the quantitative treatment of density and thermal characteristics are generalized at present.

Resistance against acid corrosion, decomposition induced by sulfate and so on is considered as chemical properties.

Durability of concrete is considered to be its resistance to time-dependent deterioration resulting from various actions, such as weather, intrusion of chemicals and erosion by chemicals. For the durability of reinforced concrete, resistance to corrosion of reinforcing steel over time is an additional concern. In regard to steel corrosion, durability performance verification has been carried out using the resistance to carbonation and chloride ion intrusion of concrete as indices.

Quality of concrete is influenced by not only conditions of material used and mix proportion, but also construction and environmental conditions at the site. Since these conditions are various, general values for material properties used in usual design, which are given here, are applicable to concrete which is made mainly from Portland cement and natural aggregate or artificial lightweight aggregate, cast in atmosphere with normal temperature, and existing under normal environmental conditions. These values are just standard values, some of which vary little under changes in conditions but others vary greatly. Therefore, if a reliable value can be obtained under actual conditions of material used, mix proportion, construction, environment, it is advisable to use it instead of a value given here.

It is the same for steel as concrete, where the values for material properties other than tensile strength are used in accordance with needs in structural performance verification.

Loads that act on civil structures are classified into static loads, dynamic loads, and impact loads. The static load acts throughout the service life such as dead loads. The dynamic loads include effects of traffic, waves, and earthquakes. The impact loads include explosions and collision of flying object. The rate of action of these loads is normally expressed as a stress or strain rate. Loading rates are known to be significantly affecting the failure mode of concrete structures. Also, the widening variety of structures increases the necessity for investigation into measures against high loading rates. The mechanical properties of materials that make up the concrete structures, i.e., concrete and steel, should therefore preferably be determined according to the stress and strain rates.

The material property values given in this chapter may be used in the examination of limit states under normal static and dynamic loads. Where special consideration for strain rate is required, such as in cases of impact loads, values obtained from highly reliable experiments shall be used.

Where the effects of strain rate on the yield stress, tensile strength, and strain at rupture of steel need to be considered, these values must be determined through reliable experiments.

(2) The characteristic value for material strength f_k may generally be obtained using Eq.(C5.1.1)

$$f_k = f_m - k\sigma = f_m(1 - k\delta) \qquad \text{(C5.1.1)}$$

where, f_m : mean value of tested values

σ : standard deviation of tested values

δ : coefficient of variation of tested values

k : coefficient

Coefficient k is determined by probability to have tested values less than a characteristic value and distribution shape of tested values. When the probability of the tested values is less than 5 % and the normal distribution is assumed, the coefficient k becomes 1.64 (See Fig.C5.1.1).

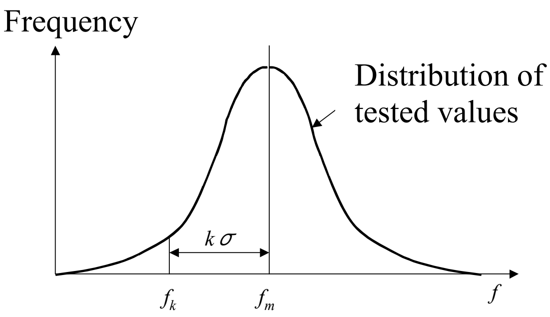


Fig. C5.1.1 Characteristic values for materials strength

5.2 Concrete

5.2.1 Strength

(1) Characteristic values for concrete strengths shall, in general, be based on 28-day tests. They may be based on tests of other appropriate ages depending upon factors such as, the proposed function of the structure, time of main loading and construction schedule.

Tests for compressive strength of concrete shall be carried out in accordance with JIS A1108 "Method of Test for Compressive Strength of Concrete".

Tests for tensile strength of concrete shall be carried out in accordance with JIS A1113 "Method of Test for Splitting Tensile Strength of Concrete"

(2) When ready-mixed concrete in accordance with JIS A5308 is used, the nominal strength specified by the purchaser may be used as the characteristic compressive strength of concrete f'_{ck}.

(3) Characteristic values for bond and bearing strengths of concrete shall be determined from strengths obtained using appropriate tests.

(4) Characteristic values for tensile, bond and bearing strengths of concrete may be obtained using Eq.(5.2.1) to Eq.(5.2.3) indicated below, based on the characteristic compressive strength f'_{ck} (basic strength for design).

In the case of lightweight aggregate concrete, where all the aggregate is lightweight, if actual strength data is not available, the design strengths may be taken to be 70% of the design strengths for normal concrete. The unit of strength is N/mm^2.

Tensile strength $f_{tk} = 0.23 f'^{2/3}_{ck}$ **(5.2.l)**

Bond strength (For deformed bars satisfying the requirements given in JIS G3112),

$f_{bok} = 0.28 f'^{2/3}_{ck}$ **(5.2.2)**

where $f_{bok} \leq 4.2$ **N/mm^2**

For plain bars, the design bond strength shall be 40% of that of deformed bars, provided that semicircular hooks shall be provided at the ends of plain bars.

Bearing strength

$$f'_{ak} = \eta \cdot f'_{ck} \tag{5.2.3}$$

where, $\eta = \sqrt{A/A_a} \le 2$

A **: concrete area for distributing bearing stress**

A_a **: bearing area**

(5) Flexural cracking strength may be obtained using Eq.(5.2.4)

$$f_{bck} = k_{0b} k_{1b} f_{tk} \tag{5.2.4}$$

where, $k_{0b} = 1 + \dfrac{1}{0.85 + 4.5(h/l_{ch})}$ **(5.2.5)**

$k_{1b} = \dfrac{0.55}{\sqrt[4]{h}}$ **(≥0.4)** **(5.2.6)**

k_{0b} **: coefficient representing the relation between tensile strength and flexural cracking strength on account of tension softening characteristics of concrete**

k_{1b} **: coefficient representing reduction in strength on account of other reasons such as drying and heat of hydration**

h **: height of member (m) (>0.2)**

l_{ch} **: characteristic length (m)**

= $G_F E_c / f_{tk}^2$ **, where,** E_c **:Young's modulus,** G_F **: Fracture energy,** f_{tk} **:tensile strength**

Fracture energy and Young's modulus may be obtained as outlined in Sections 5.2.4 and 5.2.5

(6) The material factor of concrete, γ_c **, may be taken as 1.3 (**$f'_{ck} \le 80$ **N/mm²) and 1.0, when carrying out the examination for the ultimate limit state and the serviceability limit state, respectively.**

[Commentary] (1) When concrete is cured properly, its compressive strength increases with age, and in normal structures it can be expected that the strength will become greater than the 28-day compressive strength of cylinders with standard curing. Considering this, the characteristic strength of concrete for normal structures may be determined using results of 28-day strength of standard cylinders.

However, there are the cases in which it is not appropriate for practical application to determine the characteristic strength of concrete from strength at early age. For example, many of massive concrete structure, such as underground LNG storage tanks and foundation mats, are firstly experienced with design load after a very long period has passed after placing concrete, and cement with low heat of hydration and fly ash are used for massive concrete structures. For these cases, characteristic values may be based on strengths determined at 91 days.

In the case of concrete used in products made in factories, initial strength development is rapid though the rate of increase of strength after 14 days is likely to be less than that for normal structures because of use of high-early-strength Portland cement and low water-cement ratio in concrete and use of accelerated curing such as steam and autoclave curing, etc. In some cases factory product is subject to earlier service after the required strength has been obtained. Thus it is desirable that the characteristic strengths for concrete used in products made in factories is based on strengths determined at 14days or less.

The age at which concrete can be prestressed in prestressed concrete structures depends on the type of structure and the purpose of construction. A characteristic compressive strength at prestressing may be determined from the strength tested at age when the required strength can be obtained.

As mentioned above, for some types and purposes of structures, it is not reasonable that the characteristic value is based on 28-day tests. In such cases, the characteristic value may be based on tests carried out at any other appropriate age rather than 28days.

Concrete strength depends on factors such as, age at test, curing condition before test, shape and size of specimen, loading rate, and loading conditions: eccentricity, inclination and roughness of loading surfaces, etc. In the Specification, the strengths shall be determined by the tests specified by the Japanese Industrial Standard. Shape and dimension of specimens shall be in accordance with JIS A 1132 “Method of Making and Curing Concrete Specimens”.

(4) Equation (5.2.1) was obtained for normal concrete having a strength (f'_{ck}) between 20 and 50 N/mm^2, but Eq.(5.2.1) has also been found to be applicable for concretes with f'_{ck} less than or equal to 80 N/mm^2. Since mechanical behavior of resin concrete, fiber reinforced concrete, ultra high strength concrete having strength greater than 100 N/mm^2 is different from that of normal concrete, appropriate characteristic values should be determined through experiments.

Though it has been reported that the bond and bearing strengths of high strength concrete increase as the compressive strength increases, the data is insufficient and the required characteristic values for high strength concrete should be determined through appropriate experiments.

Equation (5.2.3) is given for design bearing strength in the case where concrete area for distributing bearing stress A is greater than bearing area A_a, referring to previous experiments or requirements reported elsewhere. In Eq.(5.2.3), A is the area whose centroid coincides with that of A_a and is the center of symmetry of A (See Fig.C5.2.1). When there are many of A_a, each A shall not be overlapped with each other. Equation (5.2.3) is applicable to concretes having a strength (f'_{ck}) between 20 and 50 N/mm^2. Bearing strength for the same ratio A/A_a becomes smaller as concrete strength becomes higher than that range.

In cases when the area immediately under the bearing surface is reinforced using wire mesh, steel pipe, prestressing steel, etc. the value of η may be increased if the safety is confirmed using experiments, past experience, etc.

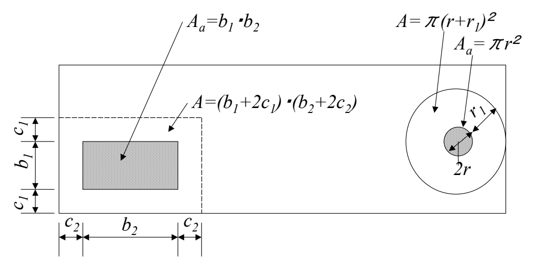


Fig. C5.2.1 Definition of bearing area

Properties of lightweight aggregate concrete depend on the proportion of lightweight aggregate content in the total, i.e., fine and coarse, aggregates. Now, the specifications given here are applicable for the case when all the fine and coarse aggregates are lightweight. For this case, for the same compressive strength the tensile, bond and bearing strengths of lightweight aggregate concrete are known to be smaller than those of normal concrete. Thus, the strengths of lightweight aggregate concrete may be taken to be 70% of the values for normal concrete.

(5) An accurate estimate of flexural cracking strength is difficult at present, as it depends on the quality of concrete, environmental conditions, and size of member. For this reason, an equation based on test data (see Fig. C5.2.2) according to which the flexural cracking strength is inversely proportional to the cube-root of the height of the member ($f_{bck} = 0.6/\sqrt[3]{h} \cdot f_{tk}$) has been used so far. On the other hand, analysis incorporating tension softening of concrete has revealed that the flexural strength decreases as the specimen size increases, converging at the tensile strength. Further, it has been qualitatively shown that the apparent flexural cracking strength can be even lower than the tensile strength if the concrete is dry. However, it is difficult at present to accurately predict residual stresses induced by drying and heat of hydration in structures. Accordingly, flexural cracking strength has been calculated taking into account the size effect, as it can be incorporated analytically by considering the softening properties. The effect of drying and heat of hydration have been treated separately by quantifying them on the basis of past test data. These can be incorporated into the equation in future, when tools for quantitative evaluation of the drying and heat of hydration become available.

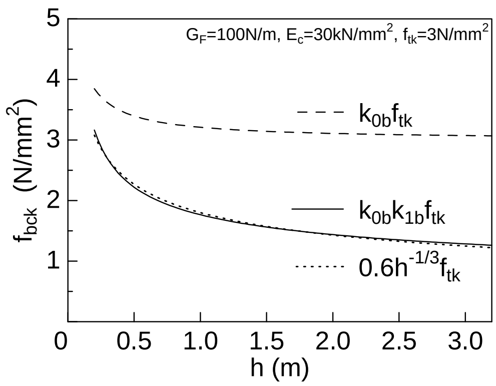


Fig. C5.2.2 Size effect of flexural cracking strength

(6) When the response of high-strength concrete ($f'_{ck} \geq 60$ N/mm^2) such as the limit state of cross-sectional failure in a wide range including the nonlinear range needs to be considered, conventional practice is to use a greater value of the material factor γ_c than the value used for normal-strength concrete. However, when the concrete construction is carried out in accordance with the provisions of the Standard Specifications for Concrete Structures “Materials and Construction”, the same value for the material factor, γ_c, may be used for all concretes in the range of $f'_{ck} \leq 80$ N/mm^2.

5.2.2 Design fatigue strength

(1) The characteristic value for fatigue strength of concrete shall be determined from fatigue strength obtained by experiments carried out considering the type of concrete, exposure condition of the structure, and others.

(2) Material factor of concrete, γ_c, used in the examination of the limit state of fatigue shall be 1.3.

(3) Design fatigue strengths f_{rd} for concrete in compression, flexural compression, tension and flexural tension, as a function of fatigue life N and stresses due to permanent loads σ_p, may be obtained using Eq.(5.2.7).

$$f_{rd} = k_{1f} f_d \left(1-\sigma_p / f_d\right)\left(1-\frac{\log N}{K}\right) \quad \textbf{(5.2.7)}$$

where, $N \le 2\times10^6$

f_d **: design strength of concrete, where material factor of concrete, γ_c, is 1.3.**

The upper limit of f_d shall be the design strength for f'_{ck} =50 N/mm^2

(i) K shall be taken as 10 for normal concrete continuously or often saturated with water and lightweight aggregate concrete, and shall be taken as 17 for other general cases.

(ii) In general, k_{1f} may be determined as follows:

For compression and flexural compression, k_{1f} =0.85

For tension and flexural tension, k_{1f} =l.0

(iii) Although σ_p is the stress due to permanent loads, it shall be taken as zero in the case of reversed cyclic loading.

[Commentary] (2) The value of γ_c may be decreased if fatigue failure will not cause the ultimate state of the structure.

(3) On the basis of previously obtained experimental data on compressive fatigue strength of normal concrete, an approximate relationship can be expressed by Eq.(C5.2.1), which consists of the ratio of minimum stress to static strength ($S_{\min}$), the ratio of maximum stress to static strength ($S_{\max}$), ratio of stress range to static strength $S_r (S_r = S_{\max} - S_{\min})$, and the fatigue life N, for N not greater than 2×10^6.

$$\log N = 17\frac{1-S_{\max}}{1-S_{\min}} = 17\left(1-\frac{S_r}{1-S_{\min}}\right) \quad \text{(C5.2.1)}$$

If the characteristic value f'_{ck} is used instead of f_d, a modified form of the Eq.(5.2.7) can be obtained. The modified equation gives a conservative estimate for most of the experimental results where k_{1f} is unity and K is 17. However, k_{1f} in Eq.(5.2.7) is determined conservatively to be 0.85 considering reduction in strength due to permanent loads, etc. Due to insufficient data for f'_{ck} above 50 N/mm^2, Eq. (5.2.7) should be used only when f'_{ck} is 50 N/mm^2 or less. However for

f'_{ck} above 50 N/mm^2, the design fatigue strength at 50 N/mm^2 may be used.

Fatigue strength of concrete in flexural tension or tension has been reported to have a greater scatter compared with that in compression. However, a modified equation of Eq.(5.2.7) with a characteristic value instead of design static strength f_d can predict conservatively most of the data similarly to the case of compression.

It has been established that the compressive fatigue strength of concrete, immersed in water or otherwise continuously wet, is approximately two thirds of that of concrete dried in air. Thus, the design fatigue strength should be specified depending on exposure conditions. It is reported that the compressive fatigue strength of lightweight aggregate concrete does not differ from that of normal concrete. On the other hand, according to another report, the former is about 10~25 % less than the latter. The lightweight aggregate concrete may be considered to include more water due to the great absorption by the lightweight aggregate. In the Specification, the compressive fatigue strength of lightweight aggregate concrete has been assumed to be equal to that of continuously wet normal concrete.

The design fatigue strength of other special concretes shall be determined experimentally to ensure that the safety level is similar to that of normal concrete.

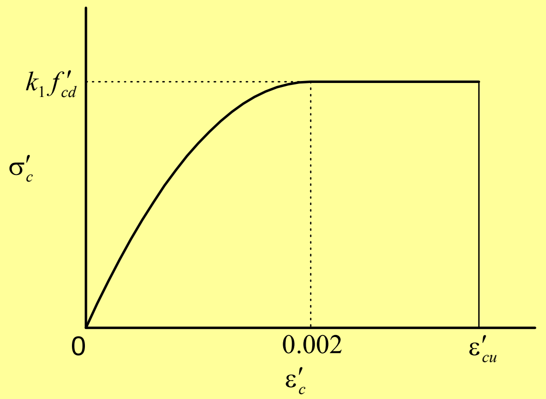
In spite of no report on fatigue strength in the case of reversed cyclic loading, compressive and tensile fatigue strengths are determined as stress range repeated in only compression and tension side, respectively.

In the case where the required fatigue life, N, exceeds 2×10^6 cycles, the design fatigue strength should be determined experimentally, because sufficient data is not available. However, Eq. (3.2.7) may still be used in the examination of the fatigue limit state, as the equation is considered to provide a conservative estimate for N exceeding 2×10^6.

5.2.3 Stress-strain curve

(1) An appropriate curve shall be assumed to express the stress-strain behavior of concrete depending upon the purpose of examination.

(2) An idealized stress-strain curve given in Fig.3.2.1 may generally be used for examination of the ultimate limit state of failure of members subjected to flexural moment or to flexure and axial forces. The stress-strain curve in Fig.3.2.1 may also be used for lightweight aggregate concrete, for the examination mentioned above.



$k_1 = 1 - 0.003 f'_{ck} \quad \le 0.85$

$\varepsilon'_{cu} = \dfrac{155 - f'_{ck}}{30000} \quad 0.0025 \le \varepsilon'_{cu} \le 0.0035$

The unit of f'_{ck} is N/mm^2.

Equation for the curved line is:

$$\sigma'_c = k_1 f'_{cd} \times \frac{\varepsilon'_c}{0.002} \times \left(2 - \frac{\varepsilon'_c}{0.002}\right)$$

Fig.5.2.1 Stress-strain curve of concrete

(3) The stress-strain curve for concrete may be assumed to be linear for examination of the serviceability limit states. In this case, the Young's modulus shall be determined in accordance with Section 3.2.5.

(4) Since the stress-strain curve under biaxial or tri-axial stress differs greatly from that in Fig.3.2.1, the effect of such stresses shall be taken into consideration for examination of ultimate limit state. For examination of serviceability limit states, the concrete may be assumed as elastic. Values of the modulus of elasticity and Poisson's ratio may be taken from those given in Sections 5.2.5 and 5.2.6.

(5) Stress strain envelop curve in compression shall include the softening branch after peak stress. Stress hysteresis shall consider both residual plastic strain and stiffness degradation on loading and reloading path.

(6) In the analysis of linear members in (5), stress-strain relationship in tension may be neglected. In cases when the compressive strength is less than 50N/mm^2, the stress-strain relationship given in Fig 5.2.2 may generally be used.

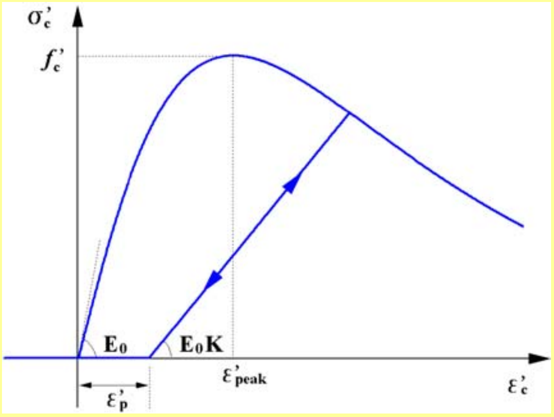


Fig.5.2.2 Simplified hysteresis model of concrete

$$\sigma'_c = E_0 K \left(\varepsilon'_c - \varepsilon'_p\right) \geq 0 \quad \textbf{(4.1.1)}$$

$$E_0 = \frac{2 \cdot f'_{cd}}{\varepsilon'_{peak}}$$

$$K = \exp\left\{-0.73 \frac{\varepsilon'_{\max}}{\varepsilon'_{peak}}\left(1 - \exp\left(-1.25 \frac{\varepsilon'_{\max}}{\varepsilon'_{peak}}\right)\right)\right\} \quad \textbf{(4.1.2)}$$

$$\varepsilon'_p = \varepsilon'_{\max} - 2.86 \cdot \varepsilon'_{peak} \{1 - \exp(-0.35 \frac{\varepsilon'_{\max}}{\varepsilon'_{peak}})\}$$

(4.1.3)

Where

ε'_{peak} : strain corresponding to compressive strength(in general, a value of 0.002 may be used)

ε'_{max} : maximum value of experienced compressive strain

ε'_p : plastic strain

K : residual rate of elastic stiffness

(7) When multi-directional cracks are considered for planar member in (5), it may be assumed that only the crack which is almost perpendicular to the direction of principle tensile stress contribute to deformation. Stress strain relationship in Fig 5.2.2 may be used assuming that the compressive zone is parallel to the direction of crack. In the tensile zone perpendicular to direction of crack, tension stiffening effect due to bond on average stress strain relationship shall be considered after cracking. If concrete fails in compression prior to the yielding of re-bar, such as the case of large reinforcement ratio, degradation of compressive strength and stiffness due to the deformation in the direction perpendicular to the direction of crack propagation shall be accounted for.

(8) When the reinforcement ratio in orthogonal directions is same for planar member in (5), shear stress transfer on crack surface may be neglected. Relationship between shear stress and shear strain on crack surface shall be considered, when reinforcement ratios in orthogonal directions differ by more than 2 times, or the loading direction change greatly during an earthquake.

[Commentary] (1), (2), (3) and (4) Even for ordinary concrete, the stress-strain curve varies greatly depending upon the type and age of concrete, applied stress condition, loading rate and history, etc. There are, however, cases such as the ultimate strength of a linear member, which are not much affected by the shape of the stress-strain curve. In such cases, an appropriate stress-strain relationship such as that given in Fig 5.2.1, or a rectangle which has been used conventionally, may be assumed. However, considering the fact that the difference between the cylinder strength and the strength obtained from estimates based on the load-carrying capacity of members increases as the strength increases, k_1 has been taken to depend on f'_{ck} as shown in Fig. C5.2.3. Also, considering the tendency of brittle failure in high strength concrete, the ultimate strain has also been reduced. It should be noted that Fig. 5.2.1 should be used as a model only for calculating the ultimate load-carrying capacity under bending and axial forces. Since the stress-strain relationship has a significant effect on the deformation, the full stress-strain curve including the descending portion showed in Fig. 5.2.2 should be used when making a detailed assessment of the deformation at the ultimate state. In such cases, compressive strain of concrete is not uniformly distributed in the specimen but localized in the descending zone. It is therefore advisable to indicate the element size, length of specimen, and the size of strain localization, which determine the stress-strain curve.

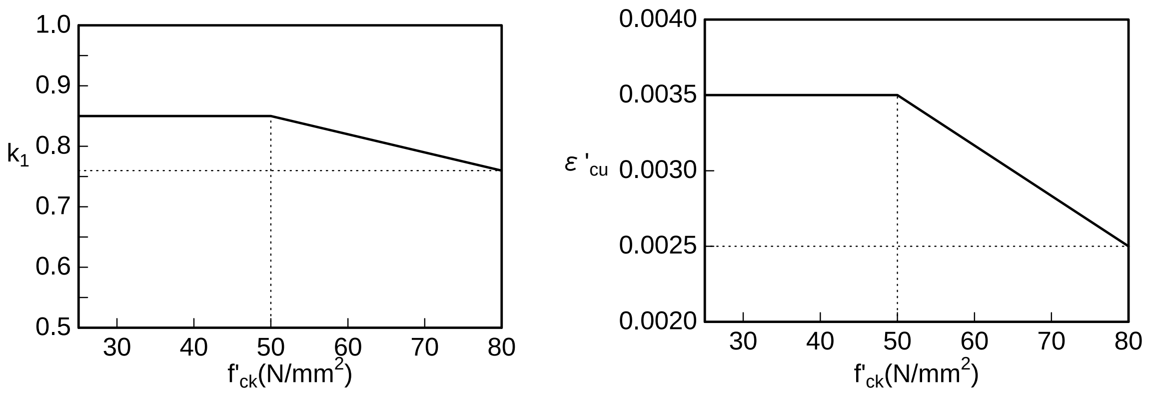


Fig. C5.2.3 Relations between k_1, ε'_{cu} and f'_{ck}

Though the present specification allows the use of the stress-strain curve in Fig.3.2.1 for lightweight aggregate concrete, the actual curve is slightly different from that of normal concrete, and therefore, an appropriate curve should be used in the examination for deformation and ductility in the case of lightweight aggregate concrete. In general, lightweight aggregate concrete is characterized by features such as, almost a linear stress-strain curve up to high levels of stress, a small initial tangential modulus of stiffness, a difference in secant modulus at one third point of compressive strength compared to normal concrete, and a sudden reduction in stress after peak stress.

The stress-strain curves given here are some commonly used examples. Concrete surrounded by hoops and spiral reinforcement is especially known to exhibit greater compressive strength and ultimate strain due to the confining effect of such reinforcement. When actual values of these parameters are available from appropriate experiments, etc. they may also be used.

(5)(6) Uni-axial material model of concrete can be distinguished with three parts: compression, tension and recontact after cracking. It is ideal that hysteresis energy absorption can be evaluated in response analysis considering the hysteretic behavior in all models.

In general, the effect of hysteresis loop of concrete in compression on response behavior is small for linear members in contrast with planar members. Therefore, the hysteretic damping at unloading and reloading in compression may be neglected. This assumption gives safety side evaluation because the response deformation is overestimated. Moreover, even if the tensile stress is neglected, the effect to the response of structures is small although concrete carries the tensile stress after cracking due to the bond with re-bar, because nonlinear region is only limited in a certain portion of linear member.

When no special examination is conducted, a simplified material model described in Eqs. (5.2.8)-(5.2.11) and in Fig. 5.2.2 may be applied in all cross section of linear member in which stress level under the constant axial force is less than about 10 % of uni-axial compressive strength of concrete. Applicability of the relation has been confirmed for the concrete with compressive strength less than about 50 N/mm^2.

For cracked concrete, the compressive stress is transmitted on closing of cracks under repeated cyclic loading. In real structures, crack contact gradually begins before the cracks are completely closed and stiffness gradually increases. On the other hand, a rapid change of stiffness is assumed in a simplified Eq. (5.2.8) and high frequency component of acceleration often generate to the axial direction of the member in dynamic response analysis. A simplified model can, however, be used, which does not consider the complex nonlinearity at recontact of crack, since the influence to the response values in the seismic performance verifications are negligibly small.

Strength to axial direction and deformation capacity of concrete increase due to confinement stress in the parallel and perpendicular direction of member axis from the re-bar arranged laterally and/or placed at outer portion in cross section. Therefore, it is desirable to apply the different stress strain relationship at each part in cross section considering the difference of confinement effect according to lateral re-bar arrangement and so on. Eqs. (5.2.8)-(5.2.11), however, show the stress strain relationship neglecting confinement effect, because it gives safety side evaluation for the general columns subjected to small axial force. Though uni-axial stress strain relationship changes with the increase in confinement stress of lateral re-bar, it has been confirmed that the residual rate of the elastic stiffness becomes large by higher confinement stress and the relationship between maximum value of experienced compressive strain and plastic strain are not influence. If the residual rate of elastic stiffness is appropriately defined according to the confinement stress, it is possible to represent the change of maximum stress, corresponding strain increment and the hysteresis under unloading and reloading.

When the response values of Seismic Performance 2 and 3 are computed for the members subjected to large axial force, it is recommended to consider the confinement effect because Eqs. (5.2.8)-(5.2.11) underestimate the member performance and it sometimes results in uneconomical design. Three dimensional analysis considering multi-axial stress field can give response value with higher accuracy, if material model including confinement effect is used. It is possible to estimate the response values using a simplified stress strain relationship of confined concrete based on the experimental results.

(7) For the planar members and shell structures, the strain hardly localizes in contrast with linear members and cracks distribute over wide area. The effect that stress transfer due to bond restraints deformation of re-bar affects to the structural response greatly. Therefore, in order to enhance the accuracy of estimated response values, accurate modeling of stress strain relationship in tension is required.

Fig (C5.2.4) is an example of uni-axial averaged stress strain relationship applied to tension field of reinforced concrete with sufficiently arranged re-bars. This example includes the bond effect with re-bar and is considered that concrete between cracks transmit tensile stress even after cracking. In this case, the effect of bond action should be simultaneously considered in the re-bar model in tension to form a counterpart with concrete model in tension. The yield stress of averaged stress strain relationship of re-bar in concrete is lower than that of bare bar due to the effect of bond action. In addition, it is desirable to consider the gradual increment of stiffness at recontact of cracks and absorption of hysteresis energy.

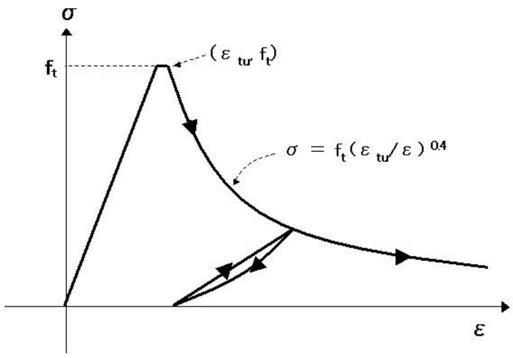


Figure C5.2.4 An example of stress strain relationship in tension of concrete

5.2.4 Tension softening properties

(1) Fracture energy, G_F, for normal concrete may be obtained using Eq.(5.2.12).

$$G_F = 10(d_{\max})^{1/3} \cdot f_{ck}'^{\,1/3} \quad \text{(N/m)} \tag{5.2.12}$$

where, $d_{\max}$ **: maximum size of aggregate (mm)**

f'_{ck} **: characteristic compressive strength (N/mm²)**

(2) An idealized curve given in Fig.5.2.3 may be used for the tension softening part.

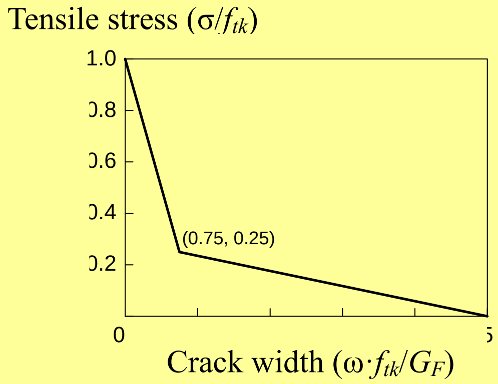


Fig. 5.2.3 Tension softening curve

[Commentary] Verification for safety of reinforced concrete may generally be carried out by assuming concrete to be a completely brittle material under tension. However, since performance verification for members, where the occurrence and propagation of cracking governs, cannot be carried out in a rational manner using conventional models, it may be necessary to take into consideration a fracture process zone at the crack tip where micro cracks accumulate. In a fracture process zone, which is located between the elastic zone having no cracks and the completely cracked portion, the transferred tensile stress decreases as the 'crack width', i.e., the total width of minute cracks in the zone, increases. This is the so-called tension softening. A tension softening curve expresses the relationship between the transferred stress and the crack width, and the area below the curve corresponds to the fracture energy, which is equal to the energy required to form a unit area of completely opened crack. It is reported that by incorporating the tension softening properties in analysis, the fracture phenomenon in concrete associated with propagation of cracks can be understood, and the size effect in the apparent strength of members can be rationally explained.

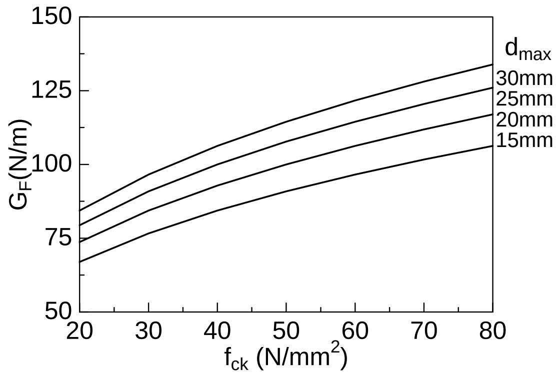


Fig. C5.2.5 Fracture energy

Equation (5.2.12) and Fig. 5.2.3 have been obtained on the basis of experimental data available in literature. Figure C5.2.5 shows the fracture energy determined using Eq. (5.2.12).

5.2.5 Young's modulus

(1) The Young's modulus of concrete shall be obtained in accordance with JIS A 1149 "Test method for static modulus of elasticity of concrete."

(2) In general, the value of the Young's modulus for concrete E_c may be taken to be equal to that shown in Table 5.2.1.

Table 5.2.1 Young's modulus of concrete

f'_{ck} (kN/mm^2)		18	24	30	40	50	60	70	80
E_c (kN/mm^2)	Normal concrete	22	25	28	31	33	35	37	38
	Lightweight aggregate concrete*	13	15	16	19	-	-	-	-

* All aggregates are lightweight

[Commentary] (2) The values given in Table 5.2.1 may be applied for computation of elastic deformation and statically indeterminate force(s) at serviceability limit states. In the case of repeated loading or when the applied stress level is low, the values in Table 5.2.1 may be preferably increased by 10 %, as the Young's modulus in such cases is closer to the initial modulus of elasticity. Each value in Table 5.2.1 is the average from a survey of test results from across the nation, and individual values vary considerably depending upon the type and quality of aggregates, and the region.

The relative effect of the Young's modulus of concrete on the safety of the structure is small compared to other characteristic values. However, in cases where the affect of the value of the Young's modulus on the design calculations is large, it is desirable that an investigation into the design conditions is undertaken and, if necessary, the measured value determined using the actual materials shall be used.

3.2.6 Poisson's ratio

Poisson's ratio of concrete may, in general, be taken to be 0.2 within elastic range.

However, it shall be taken to be zero when cracking due to tension is allowed.

3.2.7 Thermal characteristics

(l) The thermal characteristics of concrete shall be decided on the basis of experiments or previous available data.

(2) In general, the coefficient of thermal expansion for concrete may be taken as 10×10^{-6} /°C.

[Commentary] (1) In general, thermal characteristics of concrete are greatly influenced by characteristics of aggregate, which constitute most of the volume of the concrete. Even for concrete with the same mix proportion, the thermal characteristics vary considerably, depending on level of saturation and temperature.

For normal concrete, the values for the thermal characteristics may be taken to be equal to those shown in Table C5.2.2.

Thermal characteristics of lightweight aggregate concrete vary considerably depending on the above-mentioned factors. For reference, some values of heat conductivity, on the basis of previous experiments are given in Table C5.2.3. It is reported that the specific heat of lightweight aggregate is 1.6 ~1.8 kJ/kg°C, compared with 1.0~1.3 kJ/kg°C for normal concrete, and that the coefficient of heat diffusion of lightweight aggregate concrete is 0.0014 to 0.0020 m^2/h, compared with 0.0028 to 0.0040 m^2/h for normal concrete.

Equation (C5.2.2) shows the relation between thermal characteristics – the coefficient of heat conductivity, λ_c, coefficient of heat diffusion, h_c^2, and specific heat, C_c and the including density ρ. When one of the four quantities is unknown, it may be estimated using the other three. The quantities λ_c, h_c^2 and C_c are also affected by the level of saturation of concrete, therefore, saturation effect should be appropriately considered.

$$h_c^2 = \lambda_c / (C_c \cdot \rho) \qquad \text{(C5.2.2)}$$

Table C5.2.1 Thermal characteristics of normal concrete

Thermal conductivity	9.2 kJ/mh°C
Specific heat	1.05 kJ/kg°C
Thermal diffusivity	0.003 m^2/h

Table C5.2.2 Thermal conductivity of lightweight aggregate concrete (kJ/mh°C)

Unit weight (kg/m^3)	dry condition	wet condition
1500	2.1	-
1600	2.3	4.6
1700	2.5	4.8
1800	2.9	5.0
1900	3.3	5.9

5.2.8 Shrinkage

(1) Shrinkage of concrete shall be determined, considering the humidity around the structure, shape and dimension of members, mix proportion of concrete, etc.

(2) For computation of statically indeterminate forces using the elastic theory, the shrinkage strain of concrete may be taken to be equal to 150×10^{-6}. However, when this value is used, effect of creep shall not be added.

[Commentary] (1) Shrinkage of concrete, which includes drying shrinkage, autogenous shrinkage and carbonation shrinkage, is affected by properties of the aggregate and cement, compaction of concrete and curing conditions, as well as temperature and humidity around structures, shape and dimension of cross-sections of members and mix proportion of concrete. The properties of aggregate may vary regionally, and shrinkage may reach as great as 1,000 × 10^{-6}.

As a general rule, therefore, the shrinkage strain of concrete used for verification purposes must be determined on the basis of the values obtained from shrinkage strain tests of the concrete to be used or past performance data. If no such data are available, the value of shrinkage strain to be used to calculate the response of a structure may be determined by multiplying the value calculated from Eq. (C5.2.4), Eq. (C5.2.9) or Eq. (C5.2.14) by 1.5. The reason is as follows. It has been reported that in tested conducted in accordance with JIS A 1129 (10 × 10 × 40 cm specimens, seven days of water curing followed by six months of dry curing), few concrete specimens showed a shrinkage greater than $1{,}000 \times 10^{-6}$. To this shrinkage, the autogenous shrinkage occurring before the age of seven days and an estimated shrinkage that is thought to occur after the first six months (200 × 10−6) were added. Hence, as a final value, a shrinkage strain of about 1,200 × 10−6 is allowed for. Since the maximum final value of shrinkage strain obtained from an equation like Eq. E5.2.4 is about 800 × 10−6, the value is increased by multiplying by 1.5.

Environmental humidity and member size strongly affect the magnitude and rate of shrinkage strain, which may normally be estimated using Eq. (C5.2.3) for normal concrete with a compressive strength of up to 55 N/mm^2 (up to 70 N/mm^2 when W/C is reduced to increase the strength). Here, Eq. (C5.2.3) is for the concrete using normal aggregates which don't greatly increase the shrinkage of the concrete.

$$\varepsilon'_{cs}(t,t_0) = \left[1 - \exp\left\{-0.108(t-t_0)^{0.56}\right\}\right] \cdot \varepsilon'_{sh} \qquad \text{(C 5.2.3)}$$

where,

$$\varepsilon'_{sh} = -50 + 78\left[1 - \exp(RH/100)\right] + 38\log_e W - 5\left[\log_e(V/S/10)\right]^2 \qquad \text{(C 5.2.4)}$$

ε'_{sh} : final value of shrinkage strain ($\times 10^{-5}$)

$\varepsilon'_{cs}(t,t_0)$: shrinkage strain of concrete from age of t_0 to t ($\times 10^{-5}$)

RH : relative humidity (%) (45%≤ RH ≤80%)

W : unit water content (kg/m^3) (130 kg/m^3≤ W ≤230 kg/m^3)

V : volume (mm^3)

S : surface area in contact with outside air (mm^2)

V/S : volume-surface ratio (mm) (100 mm≤ V/S ≤300 mm)

t_0 and t : temperature adjusted age (days) of concrete at the beginning of drying and during drying, values corrected by Eq.(C5.2.5) should be used.

$$t_0 \text{ and } t = \sum_{i=1}^{n} \Delta t_i \cdot \exp\left[13.65 - \frac{4000}{273 + T(\Delta t_i)/T_0}\right] \qquad \text{(C5.2.5)}$$

Δt_i : number of days when the temperature is T (°C)

T_0 : 1°C

The range indicated above for the variables is the range over which the variables were varied in the experiments used in deriving Eq.(C5.2.3). The equation (Eq.(C5.2.3)) for predicting shrinkage is derived for concrete made using ordinary Portland cement. It is known that the shrinkage of concrete is not significantly affected by the cement type, while it varies depending on the mix-proportion of the concrete. Equation (C5.2.3) has also been verified to be applicable to concrete using high-early-strength cement.

Equation (C5.2.3) has been driven using experiments carried out at 20°C. When concrete is constantly subjected to high or low temperatures, the shrinkage strain shall be determined based on experimental results, because the shrinkage behavior may be different from that observed in the normal range of temperature (0~40°C).

Equation (C5.2.3) may be used for the cases where drying begins between the age of 3 to 90 days. It is applicable to concretes having a W/C between 40 to 65%, and having a compressive strength of up to 55 N/mm^2 (up to 70 N/mm^2 when W/C is reduced to increase the strength).

It has been pointed out that, in high strength concrete, autogenous shrinkage resulting from hydration of cement may not be ignored. In the case of high strength concrete with a compressive strength exceeding 55 N/mm^2, i.e., the range where Eq. (C5.2.3) is not applicable, Eq. (C5.2.6) may be used to determine the magnitude and rate of development of shrinkage strain, incorporating the effects of the environmental humidity and member size. These parameters considerably affect the strain and strain rate. Though Eq. (C5.2.6) has been derived using data up to 120 N/mm^2, it may be used for concrete having a compressive strength up to 80 N/mm^2, which is covered by the present specification. For cases when both Equations (C5.2.3) and (C5.2.6) are applicable, the one that yields a more conservative estimate should be used.

$$\varepsilon'_{cs}(t,t_0) = \varepsilon'_{ds}(t,t_0) + \varepsilon'_{as}(t,t_0) \tag{C5.2.6}$$

where, $\varepsilon'_{cs}(t,t_o)$: shrinkage strain of concrete from age of t_0 to t ($\times 10^{-6}$)

$\varepsilon'_{cs}(t,t_o)$: dry shrinkage strain of concrete from age of t_0 to t ($\times 10^{-6}$)

$$\varepsilon'_{ds}(t,t_0) = \frac{\varepsilon'_{ds\infty} \cdot (t-t_o)}{\beta + (t-t_o)} \tag{C5.2.7}$$

β : term representing the time dependency of dry shrinkage

$$\beta = \frac{4W\sqrt{V/S}}{100+0.7t_o} \tag{C5.2.8}$$

$\varepsilon_{ds\infty}$: final value of dry shrinkage strain ($\times 10^{-6}$)

$$\varepsilon_{ds\infty} = \frac{\varepsilon_{ds\rho}}{1+\eta \cdot t_o} \tag{C5.2.9}$$

$$\varepsilon_{ds\rho} = \frac{\alpha(1-RH/100)W}{1+150\exp\left\{-\dfrac{500}{f'_c(28)}\right\}} \tag{C5.2.10}$$

$$\eta = 10^{-4}\{15\exp(0.007 f'_c(28)) + 0.25W\} \tag{C5.2.11}$$

W : unit water content (kg/m^3) (130 kg/m^3$\leq W \leq$230 kg/m^3)

V/S : volume-surface ratio (mm) (100 mm$\leq V/S \leq$300 mm)

RH : relative humidity (%) (40%$\leq RH \leq$90%)

$f'_c(28)$: compressive strength of concrete at age of 28 days (N/mm^2)

($f'_c(28) \leq$80 N/mm^2)

α : coefficient representing the influence of the cement type

ordinary or low-heat cement (α =11)

high-early-strength cement (α =15)

t_0 and t : temperature adjusted age (days) of concrete at the beginning of drying and during drying; values corrected by Eq.(C5.2.5) should be used. (1day≤t_0≤98days, t_0=98days for t_0>98)

$\varepsilon'_{as}(t,t_o)$: autogenous shrinkage strain of concrete from age of t_0 to t ($\times 10^{-6}$). In general, the final value of autogenous shrinkage strain after age of t_0 shown in Table C5.2.3 may be used.

Table C5.2.3 Final values of autogenous shrinkage strain after age of t_o ($\times 10^{-6}$)

Compressive strength at 28days (N/mm^2)	t_0 (day)		
	1	3	7
100	230	110	50
80	160	80	40
60	150	90	50

The compressive strength is obtained after 28days curing in water. The accuracy in prediction of autogenous shrinkage is ±40%. Only Portland cement was used as binding material.

In cases where $\varepsilon'_{as}(t,t_o)$ is estimated as a function of time, the equation for predicting autogenous shrinkage strain (Eq.(C5.2.12) and Eq.(C5.2.13)) may be used.

$$\varepsilon'_{as}(t,t_o) = \varepsilon'_{as}(t) - \varepsilon'_{as}(t_o) \qquad \text{(C5.2.12)}$$

$$\varepsilon'_{as}(t) = \gamma\, \varepsilon'_{as\infty}\left[1-\exp\left\{-a(t-t_s)^b\right\}\right] \qquad \text{(C5.2.13)}$$

$\varepsilon'_{as}(t)$: autogenous shrinkage strain of concrete from the start of setting to age of t ($\times 10^{-6}$)

γ : coefficient representing the influence of the cement and admixtures type (γ may be unity for the case where only the ordinary Portland cement is used)

$\varepsilon'_{as\infty}$: final value of autogenous shrinkage strain ($\times 10^{-6}$)

$$\varepsilon'_{as\infty} = 3070\exp\{-7.2(W/C)\} \qquad \text{(C5.2.14)}$$

W/C : water-cement ratio

t_s : start of setting (days)

a,b : coefficient representing the characteristic of progress of autogenous shrinkage. The values shown in Table C5.2.4 may be used.

Table C5.2.4 Coefficient *a* and *b* in Eq.(C5.2.13)

W/*C*	*a*	*b*
0.20	1.2	0.4
0.23	1.5	0.4
0.30	0.6	0.5
0.40	0.1	0.7
over 0.50	0.03	0.8

Values in this table are for concrete in which only Portland cement was used as binding material.

t , t_s and t_0: temperature adjusted age (days) of concrete; values corrected by Eq.(C5.2.5) should be used.

Despite active study by several researchers on autogenous shrinkage of concrete, sufficient experimental data is still not available. The estimated autogenous shrinkage values given in Table C5.2.3 and the Eq. (C5.2.13) for estimating autogenous shrinkage have been determined from experiments using concrete containing Portland cement as the only binding material. It is clarified that the type of binder and water-binder ratio significantly affects the autogenous shrinkage of concrete. When a different type of binder is used, autogenous shrinkage of concrete shall be determined based on experimental results.

Apart from Eqs. (C5.2.3) and (C5.2.6), the shrinkage strain of concrete can be predicted using those given by Bazant, ACI Committee 209, and CEB/FIP Model Code (1990). These are widely applicable and can be used depending on the purpose of prediction.

When calculating the stress of concrete incorporating the shrinkage at ages less than 1 day, accurate evaluation should be made for not only the shrinkage at such ages but also physical properties of concrete, such as modulus of elasticity and creep.

Since the method of determining shrinkage strain by the method described above is complicated, and, mix proportions of the concrete may not be known at the design stage, the values given in Table C5.2.5 may be used for normal members made using normal strength concrete. If neither test data nor past performance data on the shrinkage strain of the concrete to be used are available, it is recommended that the value shown in Table E5.2.5 be multiplied by 1.5.

Table C5.2.5 Shrinkage strain of concrete

Environmental condition	Age of concrete*				
	less than 3 days	4 to 7 days	28 days	3 months	1 year
Outdoor	400	350	230	200	120
Indoor	730	620	380	260	130

* The age at which the concrete is subjected to drying

Here, "Normal strength concrete" refers to concrete having a unit water content between 160 to 180 kg/m^3, cement content between 350 to 400 kg/m^3, and a compressive strength of up to 55 N/mm^2 (up to 70 N/mm^2 when the strength is increased by reducing the *W*/*C*). "Normal members" here refer to those having a volume-surface ratio, *V*/*S*, of around 150mm. It is therefore desirable that the shrinkage strain of concrete should be estimated by different manner when using other types of concrete. The value of *V*/*S* is a major factor in determining the shrinkage strain. When the *V*/*S* differs from the value stated above, it can be incorporated by Eq. (C5.2.3). General civil structures are situated outdoors. In addition to measurements in real structures, the values in Table 3.2.2 for outdoor conditions were established to correspond to environments with an annual mean relative humidity of 65%, considering that Japan's climate is such that the annual mean relative humidity ranges between 60 and 70%. As the shrinkage strain is strongly affected by the relative humidity, at the design state, if an environment whose humidity conditions are widely different from those stated above is assumed, it is desirable that calculations for the shrinkage be carried out appropriately. Indoor relative humidity may be assumed to be 40% or less in air-conditioned areas and 50% in non air-conditioned areas. The values given in Table 3.2.2 for indoor conditions were established for a case with a relative humidity of 40%. These may be corrected to adapt to the conditions where the structure is to be situated. Note that the ages given in Table 3.2.2 denote the ages at which the concrete is subjected to drying. For prestressed concrete, for instance, these refer to the ages of applying prestress when the prestress losses due to shrinkage are considered.

The values given in Table C5.2.5, Eq. (C5.2.3) and Eq. (C5.2.6) are shrinkage strain values of concrete without reinforcement. The restraining effect of steel embedded in concrete should, therefore, be appropriately considered. Shrinkage strain, when longitudinal bars are symmetrically arranged at a reinforcement ratio of 1%, is given in Table C5.2.6. If neither test data nor past performance data on the shrinkage strain of the concrete to be used are available, it is recommended that the value shown in Table E5.2.6 be multiplied by 1.5.

Table C5.2.6 Shrinkage strain of normal concrete ($\times 10^{-6}$) (reinforcement ratio of 1%)

Environmental condition	Age of concrete*				
	less than 3 days	4 to 7 days	28 days	3 months	1 year
Outdoor	340	290	180	160	120
Indoor	620	520	310	210	120

* The age at which the concrete is subjected to drying

Shrinkage strain of lightweight aggregate concrete is greater than that of normal weight concrete because of the smaller stiffness of lightweight aggregate than that of normal aggregate, but the inner concrete is harder to dry because of influences of water contained inside lightweight aggregate for general civil structures where member size is relatively large. According to some experimental results using small specimens, the shrinkage strain in lightweight aggregate concrete is greater than that of normal concrete, but reports from some other experiments indicate that the two are about similar or the former is smaller than the latter. The values in Table C5.2.5 may be used for shrinkage strain of lightweight aggregate concrete as well as normal concrete.

(2) Forces induced by shrinkage of concrete in statically indeterminate structures, such as, a rigid frame and an arch, are usually calculated by elastic theory, assuming that the member contracts uniformly in the longitudinal direction. It has been confirmed that the actual force in statically indeterminate structures is much smaller than that obtained by elastic theory because of the effect of creep. Considering this, shrinkage strain may be reduced for the calculation of statically indeterminate forces. When this value is used, effect of creep shall not be added. This value may be taken as 150×10^{-6} for normal-weight ordinary-aggregate concrete made with a characteristic compressive strength of up to about 55 N/mm^2.

5.2.9 Creep

(1) Creep strain of concrete may generally be obtained using Eq.(3.2.9), assuming that it is proportional to the elastic strain caused by an applied stress.

$$\varepsilon'_{cc} = \varphi \cdot \sigma'_{cp} / E_{ct} \qquad \textbf{(3.2.9)}$$

where, ε'_{cc} **: compressive creep strain of concrete**

φ **: creep factor**

σ'_{cp} **: applied compressive stress**

E_{ct} **: Young's modulus at age of loading**

(2) In principle, creep coefficients of concrete shall be determined, considering the effect of factors such as, humidity around the structure, shape and dimensions of cross section of the member, mix proportion of concrete, age of concrete when stress is applied, etc.

[Commentary] (1) Creep strain of concrete can be considered to be in proportion to the elastic strain induced by an applied compressive stress in concrete, provided that this is not greater than about 40 % of the compressive strength. If the applied compressive stress is larger than that, it is not appropriate to assume that the creep strain is proportional to the elastic strain due to the applied stress.

(2) Besides the temperature and humidity around the structure, shape and size of member, mix proportion of concrete and age of concrete at loading, creep of concrete is affected by factors such as, properties of aggregate, type of cement, and compaction and curing condition of concrete. The design value for creep coefficient of concrete, thus, should be determined on the basis of results of previous tests and measurements from actual structures.

In the absence of experiments, creep coefficient may be obtained as given below.

The creep strain per unit stress, $\varepsilon'_{cc}(t,t',t_o)/\sigma'_{cp}$, for normal strength concrete having a compressive strength of up to 55 N/mm^2 (up to 70 N/mm^2 in the case the W/C is reduced to increase the strength), may generally be determined using Eq. (C5.2.15), when the concrete is subjected to drying and loading at temperature adjusted ages of t and t', respectively.

$$\varepsilon'_{cc}(t,t',t_o)/\sigma'_{cp} = \left[1-\exp\left\{-0.09(t-t')^{0.6}\right\}\right]\cdot \varepsilon'_{cr} \qquad \text{(C5.2.15)}$$

where, $\varepsilon'_{cr} = \varepsilon'_{bc} + \varepsilon'_{dc}$ (C5.2.16)

$$\varepsilon'_{bc} = 15(C+W)^{2.0}(W/C)^{2.4}(\log_e t')^{-0.67} \qquad \text{(C5.2.17)}$$

$$\varepsilon'_{dc} = 4500(C+W)^{1.4}(W/C)^{4.2}\left[\log_e(V/S/10)\right]^{-2.2}(1-RH/100)^{0.36}t_o^{-0.30} \qquad \text{(C5.2.18)}$$

ε'_{cr} : final value of creep strain per unit stress ($\times 10^{-10}$/(N/mm^2))

ε'_{bc} : final value of basic creep strain per unit stress ($\times 10^{-10}$/(N/mm^2))

ε'_{dc} : final value of drying creep strain per unit stress ($\times 10^{-10}$/(N/mm^2))

C : unit cement content (kg/m^3) (260 kg/m^3$\leq C \leq$500 kg/m^3)

W : unit water content (kg/m^3) (130 kg/m$^3 \leq W \leq$230 kg/m^3)

W/C : water-cement ratio (40%$\leq W/C \leq$65%)

RH : relative humidity (%) (45%$\leq RH \leq$80%)

V : volume (mm^3)

S : surface area in contact with outside air (mm^2)

V/S : volume -surface ratio (mm) (100 mm$\leq V/S \leq$300 mm)

t_0, t' and t : temperature adjusted age (days) of concrete at the beginning of drying, at the beginning of loading, and during loading, respectively; values corrected by Eq. (C5.2.19) should be used.

$$\text{for } t_0\text{, t' and } t = \sum_{i=1}^{n} \Delta t_i \cdot \exp\left[13.65 - \frac{4000}{273 + T(\Delta t_i)/T_0}\right] \tag{C5.2.19}$$

Δt_i : number of days when the temperature is T (°C)

T_0 : 1°C

Equation (C5.2.15) for predicting creep has been given for concrete made using ordinary Portland cement. Though the effect of cement type on the creep factor is relatively small, the creep strain is known to vary depending on the mix proportions of concrete. The effect of cement type should, therefore, be considered in an appropriate manner depending on the requirements. Comparison with past experimental data shows that Eq. (C5.2.15) is applicable also to cases when high-early-strength cement is used. The values given in Table 3.2.3 may therefore be used for high-early-strength cement.

Since Eq.(C5.2.15) assumes the temperature after loading to be around 20°C, it is applicable to normal conditions (normal temperatures) in Japan. As the creep strains at high or low temperatures may be different from those at normal temperatures, the creep strains in the case of structures continuously exposed to high or low temperatures should be determined using separate experiments carried out under appropriate conditions.

As in the case of Eq.(C5.2.3) for shrinkage, Eq.(C5.2.15) is applicable to concrete with a strength of up to 55 N/mm^2 (up to 70 N/mm^2 when the W/C is reduced to increase the compressive strength). In the case of high strength concrete (with a compressive strength exceeding 55 N/mm^2), Eq.(C5.2.20) may be used to predict the creep.

$$\varepsilon'_{cc}(t,t',t_o)/\sigma'_{cp} = \frac{4W(1-RH/100)+350}{12+f'_c(t')}\log_e(t-t'+1) \tag{C5.2.20}$$

where, $f'_c(t')$: compressive strength of concrete at the loading age t' (N/mm^2) ($f'_c(t') \leq$ 80 N/mm^2)

t' and t : temperature adjusted age (days) at the beginning of loading and during loading, respectively; values corrected by Eq.(3.2.19) should be used.

W : unit water content (kg/m^3) (130 kg/m$^3 \leq W \leq$ 230 kg/m^3)

RH : relative humidity (%) (40% $\leq RH \leq$ 90%)

Since Eq.(C5.2.20) has been derived using compressive strength data of up to 120 N/mm^2, it can be applied to concrete with a compressive strength of up to 80 N/mm^2, the upper limit of the scope of the present Specification. The scope for application of Eq.(C5.2.20) for conditions other than compressive strength is the same as that of Eq. (C5.2.15). For the range of strength range where both Eqs.(C5.2.15) and (C5.2.20) are applicable, the one that leads to a more conservative value should be used.

Apart from the equations given above, the creep strain can also be predicted using other equations, such as those given by Bazant, ACI Committee 209, and CEB/FIP Model Code (1990), which are effective in predicting creep under complex conditions. When the applied stress is variable, the principle of superposition may be applied, and the creep strain assumed to be linearly proportional to the applied stress, provided the stress level is lower than 40% of the compressive strength of concrete.

Since the method of determining creep coefficient by the method described above is complicated, and, mix proportions of the concrete may not be known at the design stage, the values given in Table C5.2.7 or Table C5.2.8 may be used. When creep strain is obtained using Eq.(3.2.9) using the creep coefficients shown in Table C5.2.7 or Table C5.2.8, the value of E_{ct} at 28days shall be used.

Table C5.2.7 Creep coefficients for normal weight concrete

Environmental condition	Age of concrete at which prestress is introduced or load is applied				
	4 to 7 days	14 days	28 days	3 months	1 year
Outdoor	2.7	1.7	1.5	1.3	1.1
Indoor	2.4	1.7	1.5	1.3	1.1

Table C5.2.8 Creep coefficients for lightweight concrete

Environmental condition	Age of concrete at which prestress is introduced or load is applied				
	4 to 7 days	14 days	28 days	3 months	1 year
Outdoor	2.0	1.3	1.1	1.0	0.8
Indoor	1.8	1.3	1.1	1.0	0.8

The creep coefficient given in Tables C5.2.7 and Table C5.2.8 have been established on the basis of the values calculated using Eq.(C5.2.15) for normal members, and are applicable to unreinforced concrete with a compressive strength of up to 55 N/mm^2 (up to 70 N/mm^2 when the W/C is reduced to increase the strength). Tables C5.2.9 and C5.2.10 give the creep coefficient for members with a reinforcement ratio of 1%, with the longitudinal bars being provided symmetrically.

Table C5.2.9 Creep coefficient for normal concrete (reinforcement ratio 1%)

Environmental condition	Age of concrete at which prestress is introduced or load is applied				
	4 to 7 days	14 days	28 days	3 months	1 year
Outdoor	2.1	1.4	1.2	1.1	0.9
Indoor	1.9	1.4	1.2	1.1	0.9

Table C5.2.10 Creep coefficient for lightweight concrete (reinforcement ratio 1%)

Environmental condition	Age of concrete at which prestress is introduced or load is applied				
	4 to 7 days	14 days	28 days	3 months	1 year
Outdoor	1.6	1.1	0.9	0.8	0.7
Indoor	1.4	1.1	0.9	0.8	0.7

Similar to the case of shrinkage, "normal members" here refer to members made of concrete proportioned with a unit water content, *W*, between 160 to 180 kg/m^3 and cement content, *C*, between 350 to 400 kg/m^3. The applied compressive stress should be lower than 40% of the compressive strength, and the volume-surface ratio, *V*/*S*, is around 150mm. The values are applicable to an outdoor temperature and relative humidity of 15°C and 65%, respectively, and indoor temperature and relative humidity of 20°C and 40%, respectively. The values of the creep coefficient should be calculated separately when the proportion of concrete mixture or other conditions differ widely from those mentioned above.

The outdoor mentioned here means a situation in which structure is exposed to open air. The outdoor conditions correspond to environments with an annual mean relative humidity of 65%, considering that Japan's climate is such that the annual mean relative humidity ranges between 60 and 70%. 'Indoor' refers to a condition when the structure is protected from open air, and where the relative humidity is approximately 50 %. When cooling in summer and heating in winter is provided, humidity may not become greater than 40 %. The values in Table C5.2.7, therefore, have been given for humidity of 40 % as a standard.

As the average temperature in Japan, except for the Hokkaido and Okinawa, is about 15 °C, temperatures of 15 °C for outdoor and 20 °C for indoor have been taken as a standard.

The creep coefficient given here may be used to compute loss of prestress and to calculate statically indeterminate forces, where the structural system varies during and after construction.

The above equations and values for creep coefficient are applicable to normal concrete for structural use. Creep in high strength concrete, concrete under temperatures higher than 40 °C, and concrete under high levels of stress should be considered separately.

In addition to the factors that affect the creep characteristics of normal concrete, in the case of lightweight aggregate concrete, these characteristics are also known to depend greatly on the type of the lightweight aggregate used and its ratio to ordinary aggregates in concrete. According to the measurement carried out using small specimens, it has been reported that creep strains in lightweight aggregate concrete are about the same as or greater than those in normal concrete. However, creep coefficient in lightweight aggregate concrete have been confirmed to be between 60 and 85 % of those of normal concrete, as the modulus of elasticity of the former is much smaller than that of the latter. The creep coefficient for lightweight aggregate concrete may, thus, be assumed to be 75% of those for normal concrete, and, the values given in Table C5.2.8 may be used.

When calculating the creep strain using Eq.(5.2.13), the modulus of elasticity at the age of loading should be estimated from the value at 28 days, using the equation given in CEB/FIP Model Code (1990) or other appropriate equation.

5.2.10 Influence of low temperature

(1) The characteristic value of concrete strength at low temperatures shall be determined with appropriate consideration of temperature and moisture content on the basis of experimental results.

(2) The characteristic value of compressive strength at low temperatures may be estimated by adding the increments in the strength determined by the temperature and moisture content.

(3) Tensile strength and modulus of elasticity of concrete at low temperatures may be estimated on the basis of the compressive strength at low temperatures.

[Commentary] (1) to (3) The compressive strength of concrete at low temperatures may be determined by adding the compressive strength and the increment in strength due to the effect of low temperature and moisture content. According to past studies, in the temperature range between 0 and -100°C, the compressive strength increases parabolically as the temperature drops, while it increases linearly as the moisture content increases. The increment in the compressive strength at -60°C may be taken to be approximately 60 N/mm^2 for moist-cured concrete and 45 N/mm^2 for concrete air-cured at 20°C and 60% RH. As the mean humidity in Japan is approximately 65%, the increment in the compressive strength is likely to fall in this range.

Since it has been found that the tensile strength at low temperatures is linearly related to a radical of the compressive strength at low temperatures with the index of the root ranging between 3/4 and 1, provided the temperature ranges between 0 and -100 °C. The tensile strength may therefore be estimated from the compressive strength at low temperatures using Eq. (3.2.1). Equation (3.2.1) approximately gives the lower limit of experimental data in the past.

As stated above, concrete shows a significant increase in strength at low temperatures, which leads to an associate increase in the load-carrying capacity of members. However, it is safer not to consider this increase in strength at low temperatures, in cases when normal temperatures may also be experienced during the design service life. Further, in cases when thermal stress due to drop and gradient in temperature is of concern, it is important to consider appropriately the temperature and the level of saturation, as it considerably affects the rigidity and tensile strength of members.

Since using an excessively low Young’s modulus leads to an underestimation of thermal stresses, it is advisable to determine the Young’s modulus considering the temperature and level of saturation. At low temperatures, the Young’s modulus of concrete has been found to vary linearly with compressive strength. An appropriate form of the stress-strain curve for concrete at low temperatures may be assumed considering the temperature and level of saturation, depending on the purpose of investigation. Poisson’s ratio in the elastic range may normally be assumed to be 0.2, as in the case of normal temperatures. However, it should be taken as zero when cracking is allowed in tension. Variation of the thermal characteristics of concrete with temperature should be accounted for.

5.2.11 Carbonation rate

The characteristic value of carbonation rate, α_k, shall be generally defined based upon experimental or past data.

[Commentary] The carbonation rate is a proportional constant when it is assumed that the depth of carbonation varies linearly with the square-root of the exposure period.

The carbonation rate defined by selected material and mix proportion must satisfy characteristic value of carbonation rate α_k for the verification:

$$\gamma_p \alpha_p \leq \alpha_k \qquad \text{(C5.2.21)}$$

where,

α_p : estimated value of carbonation rate ($mm/\sqrt{year}$).

γ_p : safety factor to account for the accuracy in determining α_p. Generally, it may be taken to be between 1.0 and 1.3.

The carbonation rate may be obtained using the following equation with effective water - binder ratio and binder type.

$$\alpha_p = a + b \cdot W / B \qquad \text{(C5.2.225)}$$

a , b : constants determined from results of actual experiments associated with binder type

W / B : effective water - binder ratio.

Strictly speaking, the constants a and b in Eq. (C5.2.22) also depend on the environmental conditions, so the effect of the environment on these parameters shall be appropriately taken into account, especially in cases where the conditions are particularly severe from the point of view of carbonation. The following regression equation was obtained based on the experimental data of 17 cases by the "Research Subcommittee on Fly Ash" using ordinary portland and moderate heat portland cements.

$$\alpha = -3.57 + 9.0W / B \quad (mm / \sqrt{year}) \qquad \text{(C 6.4.1)}$$

Where

W/B : effective water binder ratio

$$= W / (C_p + k \cdot A_d)$$

W : unit water content of concrete (kg/m^3)

B : unit effective binder content of concrete (kg/m^3)

C_p : portland cement content (kgs) per m^3 of concrete

A_d : mineral admixture content (kgs) per m^3 of concrete

k : constant representing the efficiency of the mineral admixture used.

for fly ash, k=0.

for ground granulated blast furnace slag, k=0.7.

On the basis of data from several sources and regression analysis, the "JSCE Research Subcommittee on Fly Ash" concluded that the ratio between the carbonation depth and the square-root of the exposed age (in years) has a linear relationship with the water-binder ratio. The carbonation rate is basically the slope of this straight line. In their experiments, cylindrical concrete specimens (diameter 15cm and height 30cm) were exposed outdoor after an initial 14-day period of curing under water. The carbonation depth in the experiments was determined by spraying phenolphthalein solution on the surface after cutting through the specimen. The procedure followed by the Research Subcommittee mentioned above may be used as a reference when carrying out carbonation related studies.

5.2.12 Diffusion coefficient of chloride ions in concrete

The diffusion coefficient of chloride ions in concrete, D_k, shall be generally defined based upon experimental or past data.

[Commentary] The diffusion coefficient is a proportional constant in Fick's laws of diffusion, expressing the speed of diffusion. The diffusion coefficient used in Eq. (8.3.6) in Chapter 8, has been determined with the basic assumption that the penetration of chloride ions in concrete is a one-dimensional process. The diffusion coefficient of concrete must be determined to prevent corrosion of the reinforcing bars caused by the ingress of chloride ions.

The diffusion coefficient of chloride ions in concrete determined by selected materials and mix proportion must satisfy diffusion coefficient D_k for the verification:

$$\gamma_p D_p \leq D_k \qquad \text{(C5.2.24)}$$

where,

D_p : estimated value (cm^2/ year) of diffusion coefficient of concrete.

γ_p : safety factor to account for the accuracy in determining D_p. Generally it may be taken to be between 1.0 and 1.3.

Some regressive equations have been proposed for the prediction of the diffusion coefficient of concrete based on the investigation results of chloride concentration distribution in actual structures. The following are two examples of such equation.

(a) When normal portland cement is used;

$$\log D_p = -3.9(W/C)^2 + 7.2(W/C) - 2.5 \qquad \text{(C 5.2.25)}$$

(b) When blast furnace slag cement is used;

$$\log D_p = -3.0(W/C)^2 + 5.4(W/C) - 2.2 \qquad \text{(C 5.2.26)}$$

For estimating the diffusion coefficient based on the results of the distribution of chloride ions in concrete from actual structure inspections or exposure tests, the boundary condition in terms of the surface chloride concentration is assumed and the apparent diffusion coefficient of chloride ions estimated subsequently. In short, the estimated apparent diffusion coefficient depends on the assumed surface chloride concentration.

For estimating the diffusion coefficient based on the experiment, test method for effective diffusion coefficient of chloride ion in concrete by migration (JSCE-G571-2003), test method for apparent diffusion coefficient of chloride ion in concrete by submergence in salt water (JSCE-G572-2003), Measurement method for distribution of total chloride ion in concrete structure (JSCE-G573-2003) may be referred.

5.2.13 Relative dynamic modulus of elasticity for freezing-thawing action

Relative dynamic modulus of elasticity for freezing-thawing action shall be generally determined by JIS A 1148 (A method) "The Freezing-Thawing Test Method of Concrete (Freezing-Thawing Test Method of Concrete Underwater)."

[Commentary] The relative dynamic modulus of elasticity of concrete is defined as the ratio (in percentage) between the one-dimensional resonant frequencies measured after and before deterioration of the specimen in the test carried out in accordance with the provisions of JIS A 1127 "Testing Method for Dynamic Modulus of Elasticity, Dynamic Shear Modulus, and Dynamic Poisson's Ratio of Concrete by Resonance Vibration." However, if actual freezing and thawing conditions are more severe than the condition stipulated in JIS A 1148 (A method), or, if the structure is designed for a very long service life, the test conditions in terms of the freezing and thawing temperatures, the time durations for the freezing and thawing cycles, etc. should be appropriately modified. For normal concrete with the air content of fresh concrete is between 4% and 7% , the relative dynamic modulus of elasticity is sufficient to ensure that the water to cement (or binder) ratio is less than that stipulated in Table C5.2.11.

Table C5.2.11. Water-cement ratio (%) to satisfy the relative dynamic modulus of elasticity in freezing and thawing

	Water-cement ratio (%)			
	65	60	55	45
Relative dynamic modulus of elasticity (%)	60	70	85	90

1) The value of relative dynamic modulus of elasticity of concrete having water-cement ratio between the listed values can be obtained by the linear interpolation.

2) When water-cement ratio is lower than 45%, the value of relative dynamic modulus of elasticity becomes 90%.

5.2.14 Concrete properties for verification of initial cracking

(1) Concrete properties associated initial cracking resistance shall be determined based upon experimental or past data.

(2) The values representing thermal properties to be used in thermal analysis, i.e., thermal conductivity, thermal diffusivity and specific heat, shall be adequately determined considering the mix proportion of concrete used.

(3) The adiabatic temperature rise of concrete shall be adequately determined considering such factors as the materials and mix proportion of concrete used and concrete temperature at placing.

[Commentary] (2) The thermal properties of cement paste vary depending on the progress of hydration and the water content. The thermal properties of concrete, however, may be assumed to be constant because the thermal properties of aggregate, which accounts for a major part of concrete volume, are constant. Thermal constants of concrete generally depend on the mix proportion of concrete, especially on such factors as the property and unit content of aggregate, and the moisture content of concrete. Therefore, the constants are recommended to be determined considering the effects of these factors. The constants may be obtained in 5.2.7.

For concrete to be used in ordinary concrete structures, the thermal conductivity, specific heat and thermal diffusivity may be taken as 2.6-2.8 W/m°C, 1.05-1.26 kJ/kg°C and 0.83-1.1E-6 m^2/s, respectively.

Once the thermal conductivity and density are determined, the other constants can be estimated by the following equations.

$$h^2_c = 3.34 \times 10^{-7} \cdot \lambda_c \qquad \text{(C5.2.27)}$$

$$C_c = 3.03 \times 10^3 / \rho \qquad \text{(C5.2.28)}$$

(3) The design value of adiabatic temperature rise of concrete is expressed by Eq. (C5.2.29) unless either chemical admixtures or admixtures that are highly effective for retarding hydration are used.

$$Q(t) = Q_\infty (1 - e^{-\gamma t}) \qquad \text{(C5.2.29)}$$

where, Q: ultimate adiabatic temperature rise to be determined by testing, γ : constant on rate of temperature rise to be determined by testing, t : age (days), $Q(t)$: adiabatic temperature rise at an age of t (°C). Methods for estimating Q_∞ and r in cases where typical cements are used are described in Chapter 4 of "Design of Concrete Structure."

5.3 Reinforcing steel

5.3.l Strength

(1) The characteristic value of the tensile yield strength of reinforcing steel f_{yk} and that of the tensile strength f_{uk} shall be determined based on strengths obtained by tensile tests, carried out in accordance with JIS Z2241 "Method of Tensile Test for Metallic

Materials."

(2) For reinforcing steel in accordance with JIS, the characteristic values f_{yk} and f_{uk} may be regarded as the lower-limit values specified in JIS. In general, for design of limit state, the nominal cross-sectional area may be used as the area of reinforcing steel.

(3) The characteristic yield strength of reinforcing steel in compression f'_{yk} may be taken to be equal to the tensile yield strength f_{yk}.

(4) The characteristic yield strength of reinforcing steel in shear f_{vyk} may be obtained using Eq.(5.3.1).

$$f_{vyk} = f_{yk} \Big/ \sqrt{3} \qquad \textbf{(5.3.1)}$$

(5) Material factor for steel γ_s, used for the examination of the ultimate limit state, shall be taken as given below:

For reinforcing bars and prestressing steel 1.0

For other steel 1.05

For examining the fatigue limit state, a value of 1.05 may be used.

For examining the serviceability limit state, a value of 1.0 may be used.

[Commentary] (2) The characteristic tensile yield strength and the tensile strength of reinforcing steel shall be determined based on the strengths obtained by tensile tests, in the same manner that the characteristic compressive strength and the compressive strength of concrete are determined from appropriate experiments. Past results have shown that the characteristic values are generally equal to or a little greater than the lower limit values specified in JIS. Therefore, the lower limit values specified in JIS may be taken as the characteristic values.

In order to use the test method specified in JIS Z 2241 "Method of Tensile Test for Metallic Materials", determination of the cross-sectional area of reinforcing steel is required. Cross-sectional area used for the calculation of strength in accordance with JIS varies with the type of reinforcing steel. The nominal cross-sectional area specified in JIS may be used for purposes of design. As for JIS Z 2241, strength of reinforcing bars is calculated using the nominal cross-sectional area, and that of prestressing bars is calculated using the larger of the actual or the nominal cross-sectional area. This method of calculation yields conservative results. There is no difficulty in the case of prestressing steel wire or wire strand, as the strength is given in terms of that of a single wire. On the other hand, the strength of structural steel should be calculated using the original cross-sectional area. When the cross-sectional area determined using nominal thickness is applied to design, the strength may be overestimated, compared to the actual strength, due to differences of dimension. However, it may be assumed that practical design is not greatly affected by this problem.

When threads are cut on the surface of a prestressing bar by the rolling, for large bars having a nominal diameter greater than or equal to 26 mm, the tensile strength in the threaded portion is hardly decreased due to this plastic working. On the other hand, for smaller prestressing bars with a standard diameter is not greater than 23 mm, the tensile strength of the threaded portion decreases slightly. However, except for bars whose nominal diameter is 13 mm or smaller, the decrease in the tensile strength in the threaded portion is within 5 % of the strength of the original bar and may be neglected.

(3) The behavior of reinforcing steel during the elastic to plastic deformation in a compression

test is basically the same as that observed in a tensile test. Though the tensile yield strength calculated in accordance with the provisions of JIS Z 2241 "Method of Tensile Test for Metallic Materials", where the original cross-sectional area is used, is not the true value of the stress, the effect of which is not significant for consideration of limit state, and therefore, the characteristic value of the yield strength in compression has been taken to be the same as that in tension.

(4) The yield strength of reinforcing steel in shear may be obtained by applying Von Mises' yield criterion.

(5) When operations such as threading and cutting, threaten to harmfully affect the strength of the original material, such effects should be appropriately accounted for.

5.3.2 Design fatigue strength

(1) The characteristic fatigue strength of reinforcing steel shall be determined based on the fatigue strength obtained from tests considering the type of steel, surface configuration, size, method for jointing, magnitude and frequency of stress application, environmental conditions, etc.

(2) The design fatigue strength of a deformed bar f_{srd} may be obtained using Eq.(5.3.2) as a function of the fatigue life *N* and the stress σ_{sp} in the deformed bar due to permanent load.

$$f_{srd} = 190\frac{10^a}{N^k}\left(1-\frac{\sigma_{sp}}{f_{ud}}\right)\Big/\gamma_s \quad \text{(N/mm}^2\text{)} \qquad (5.3.2)$$

for $N \le 2\times10^6$

where, f_{ud} **: design tensile strength of reinforcing bar, which may be obtained by assuming that the martial factor is 1.05.**

γ_s **: material factor of reinforcing bar, which may be taken as 1.05.**

(i) In principle, the values of a and k should be determined experimentally.

(ii) When the fatigue life *N* is less than or equal to 2×10[6] cycles, the values of a and k may be obtained using Eq.(5.3.3).

$$a = k_{0f}(0.81-0.003\phi)$$

$$k_{0f} = 0.12 \qquad (5.3.3)$$

where, ϕ **: diameter of the reinforcing bar**

k_{0f} **: a factor depending on the shape of lug and may generally be taken as 1.0**

(3) In the case of reinforcing bars with gas pressure welding joints, the design fatigue strength may be assumed to be 70% of that of the original reinforcing bar. In the case of reinforcing bars made by welding of bars, or having bent parts, the design fatigue strength may be assumed to be 50% of that of the original reinforcing bar.

[Commentary] (1) In principle, the fatigue strength shall be determined experimentally because the fatigue strength of reinforcing steel is affected by material properties, shapes, dimensions and many other factors.

In prestressed concrete structures, design that does not permit cracks to occur has been quite prevalent and, therefore, the effect of variable stresses in prestressed concrete structures is relatively small.

Consequently, it was considered that the fatigue did not have a great influence on the behavior of prestressed concrete structures. In the case of prestressed reinforced concrete structures mentioned in 15.2 (3), however, the stress variability of prestressing steel is relatively large. Fatigue resulting from such variable stress should, therefore, be investigated.

The fatigue strength of prestressing steel should be determined by fatigue tests using prestressing steel and anchorage elements to be used in the project. Where no test data or reliable references are available, the fatigue strength may generally be determined by Eqs.(C5.3.1) and (C5.3.2).

Regarding the prestressing steel, there is tendency that the fatigue strength of anchorage is lower than that of the steel itself, therefore, especially for unbonded prestressing steel and the out-cable system, it is necessary to examine the fatigue characteristics of anchorage.

For prestressing wire and wire strand

$$f_{prd} = 280 \frac{10^{ar}}{N^k} \left(1 - \frac{\sigma_{pp}}{f_{pud}} \right) \Big/ \gamma_s \quad (\text{N/mm}^2) \tag{C5.3.1}$$

For prestressing bar

$$f_{prd} = 270 \frac{10^{ar}}{N^k} \left(1 - \frac{\sigma_{pp}}{f_{pud}} \right) \Big/ \gamma_s \quad (\text{N/mm}^2) \tag{C5.3.2}$$

where, f_{prd} : design fatigue strength of prestressing steel

N : fatigue life

σ_{pp} : stress of a prestressing steel due to permanent loading

f_{pud} : design tensile strength of prestressing steel

a_r and k : values shown in Table C5.3.1 may be used in general

γ_s : material factor of prestressing steel, which may be 1.05 in general

Table C5.3.1 a_r and k

	Prestressing wire and wire strand	Prestressing bar
a_r	1.14	0.96
k	0.19	0.16

The fatigue strength of a structural steel is affected by residual stresses and stress concentration of a welded portion, because structural steel is generally welded or connected by high tension bolts for the assemblage of structural members. As for steel plate, since a lot of fatigue tests have been carried out, the fatigue strength may be determined by considering test results obtained.

(2) The fatigue strength of a deformed bar is affected by diameter of the bar, shape of lug, surface configuration, etc. Equation (5.3.2) is obtained by arranging the results of fatigue tests that have been carried out in Japan. Repeated loading cycles of almost all these data are not greater than 2x10^6 cycles. When the arc at the base of a lug does not exist and the intersectional angle between the direction of the lug and the longitudinal direction of the bar is more than or equal to 60 degrees, the value of k_{0f} in Eq.(5.3.3), in general, may be regarded as 1.00. The value of k_{0f} may be taken as 1.05 for the case of absence of the arc and the intersectional angle of less than 60 degrees, and k_{0f} may be taken as 1.10 for the case of presence of the arc. Since the number of fatigue test data, with repeated loading cycles are more than 2x10^6 cycles, is not sufficient to propose an equation for design fatigue strength, it is advisable to confirm fatigue strength by tests for the case of repeated loading cycles of more than 2x10^6 cycles. The value of a and k in Eq.(5.3.3) ,for repeated loading cycles of more than 2x10^6 cycles, generally becomes on the safe side and, therefore, the value given by Eq.(5.3.3) may be used as it is for the examining limit state.

In general, material factor, γ_s, may be taken as 1.05. The value of γ_s may be decreased if the fatigue failure of reinforcing bar will not cause the ultimate state of the structure.

(3) As for the joint of reinforcing bars, there are many kinds of mechanical joints besides gas pressure welding joint. Since there are a lot of connecting methods for mechanical joints, it is advisable to determine the fatigue strength of each mechanical joint by confirming the fatigue characteristics by tests.

It is well known that fatigue strengths of reinforcing bars decrease when they are welded or worked by machine. According to previous researches on this problem, various decreasing rates in fatigue strength have been reported and are not constant. However, it is prevalently considered that the fatigue strengths of reinforcing bars welded or worked by machine may decrease to almost 50 % of that of the original straight reinforcing bar. It is actually quite obvious that the welding for connecting reinforcing bars induces considerable decrease in the fatigue strength. Hence, if actual fatigue strength is not confirmed by tests, the design fatigue strength may be considered to be 50 % of the fatigue strength of the original straight reinforcing bar.

5.3.3 Stress-strain curve

(1) The stress-strain curve of reinforcing steel shall be assumed to have a suitable form for the purpose of the examination.

(2) As for the examination of ultimate limit state, the stress-strain curve represented in Fig.5.3.1 may be generally used.

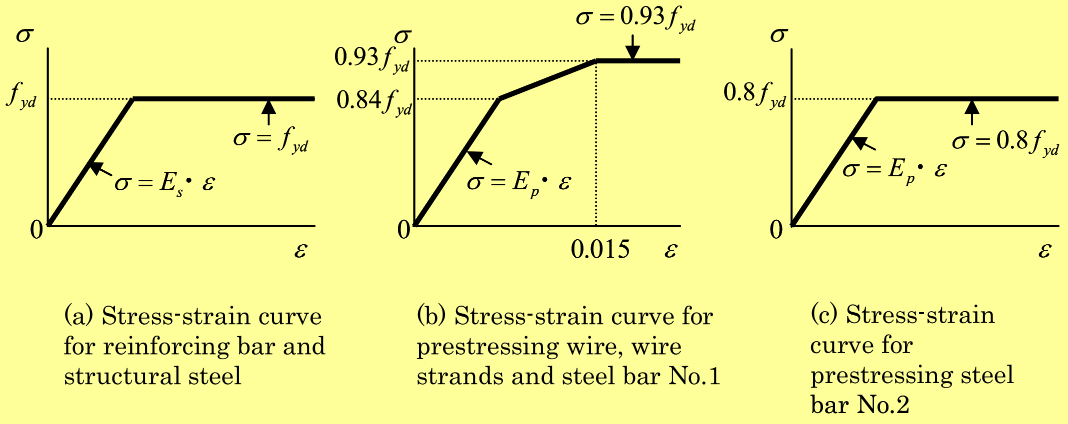


Figure 5.3.1 Stress-strain curve of reinforcing steel

(3) In the seismic performance verifications under cyclic loading, stress strain relationship shall appropriately consider yielding, strain hardening, Bauschinger effect and absorption of energy during hysteresis. Behavior in compression and tension may be taken to be identical.

(4) Material modeling of re-bar in (3) shall be used with the stress-strain relationship corresponding to the one of concrete in tension zone in consideration of the bond effect with concrete. Especially for analysis of planar members, relationship between average stress and average strain shall be used corresponding to stress strain relationship of concrete in tension zone

(5) In general, stress strain relationship in Fig 5.3.2 may be used in (3).

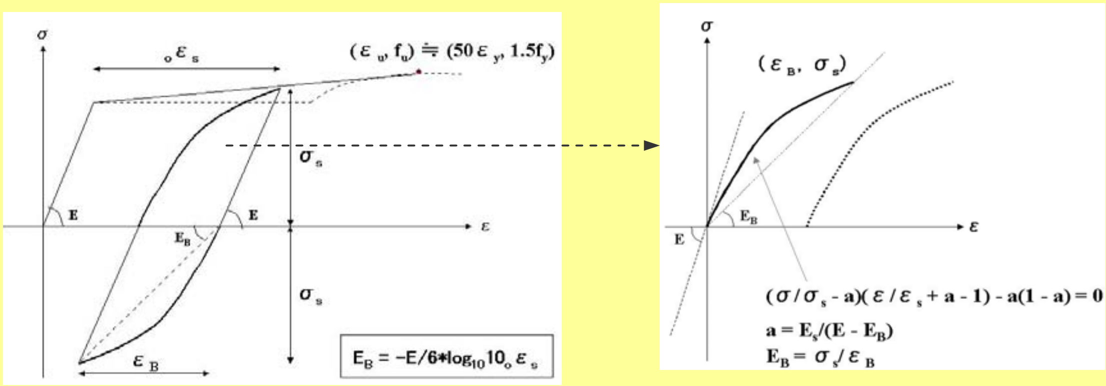


Figure 5.3.2 Hysteresis model of re-bar

$$(\sigma / \sigma_s - a)(\varepsilon / \varepsilon_s + a - 1) - a(1 - a) = 0 \quad (5.3.4)$$

$$a = E_s/(E - E_B) \quad (5.3.5)$$

$$E_B = \sigma_s / \varepsilon_B \quad (5.3.6)$$

$$E_B = -(E/6)\cdot \log_{10} 10_o \varepsilon_s \quad (5.3.7)$$

Where $_o\varepsilon_s$: Summation of experienced strain on skeleton curve part

Then, Opposite loop from tension to compression is as the symmetric form.

[Commentary] (1) and (2) The stress-strain curve of reinforcing steel is widely different according to kinds of reinforcing steel, chemical composition, manufacturing methods, etc. For example, the ratio of yield strength to tensile strength ranges from 65 % to 80 % for reinforcing bars, from 85 % to 95 % for prestressing steel and from 55 % to 80 % for structural steel without tempering. Therefore, it is necessary to assume a suitable form for the stress-strain curve corresponding to the purpose of the examination. However, according to the contents of the examinations, there are some cases where the difference of the stress-strain curve does not have a great influence on calculated results for design. The stress-strain curve given in Fig.5.3.1, in general, may be used for the examinations of stress distribution and ultimate strength of cross section of a structural member.

(3)-(5) The average stress and average strain relationship of re-bar in cracked concrete is different from the one of bare bar. The reason is that the re-bar remains in elastic state due to the bond effect everywhere else besides the vicinity of the crack plane even if re-bar yields at the crack plane. The yielding portion at crack plane extends with the increase in stress so that the elastic and plastic portion in the element is not constant. Therefore, the average stress and average strain relationship of the re-bar in concrete does not exhibit a yield plateau as in a bare bar. The yielding stress of average stress and average strain relationship in cracked concrete is lower than the yield stress of bare bar due to the bond stress between re-bar and concrete. The decrease of the yield stress in the relationship should be considered in the re-bar model because tensile stiffness after cracking still remains in material model in concrete used in planar member. These models greatly influence the response values for planar members and shell structures.

When no special examination is conducted for hysteresis process, a simplified material model described in Eqs. (5.3.4)-(5.3.7) and in Fig. 5.3.2 may be applied for the member in which stress level under the constant axial force is less than about 10 % of uni-axial compressive strength of concrete and diameter of re-bar is sufficiently small as compared with the cross section of member. The yield stress of re-bar after yielding under reversed loading become lower values, which is known as the Bauschinger effect. Equations (5.3.4)-(5.3.7) give the hysteresis loop including the effect, where the equations represents the curve from the stress reversed point (ε_s, σ_s) as the curve crossing the origin. Opposite loops from tension to compression are assumed to be the symmetric.

It is specified that tensile stress of concrete may be neglected for linear member, because the influence on the structural response is small. When this assumption is applied, a simplified bi-linear model for the skeleton curve of stress and strain relationship may be used and the yield stress of re-bar in concrete may be same as one of bare bar. This model assumes that the strain hardening occurs just after the yielding. The reason why the strain hardening is considered in 'Seismic Performance Verification' though its effect is not considered in 'Structural Performance Verification' is that the effect of strain hardening on the estimation of member deformation after yielding is very large and it is important to exhibit the ductility for member having strain gradient. The bi-linear model connecting initial yield stress point with tensile strength point is adopted, since the bond between concrete and re-bar is lost in large deformation region.

In large deformation region, main bar deforms in lateral direction under axial force and load carrying capacity in axial direction is lost with spalling of cover concrete. Therefore, it may reflect to analysis the occurrence of re-bar buckling and the residual compressive stress after swelling-out of main re-bar considering arrangement of lateral re-bar, diameter of main re-bar and material strength and so on. The effect of swelling-out of main re-bar is not small for the specimen that in which diameter is large in comparison with cross section size. However, in real structures, since the diameter in general is small in comparison with cross section size, the effect of swelling-out of main re-bar show a tendency to be relatively small in comparison with test specimen. Equations (5.3.4)-(5.3.7) are not considered the effect of buckling, because load carrying mechanism of main re-bar with swelling to lateral direction has been studying. Structural analysis factor should be set as large value, since the response value of deformation is estimated as a smaller value when the effect of swelling-out is neglected.

It is possible to take smaller value than the standard value specified in 4.5, for the structural analysis factor of response value of displacement, when the effect of swelling-out is considered in material model. It is necessary to set the structural analysis factor and the member factor judging the accuracy of modeling.

5.3.4 Young's modulus

(1) It is a principle that the Young's modulus of reinforcing steels shall be determined based on results of the stress-strain curve obtained by the tensile test specified in JIS Z 2241 "Method of Tensile Test for Metallic Materials".

(2) The Young's modulus of reinforcing steels, which are reinforcing bars, structural steel and prestressing steel, in general, may be taken as 200kN/mm^2.

[Commentary] The Young's modulus of reinforcing steel varies according to some factors, such as measuring methods employed. The value generally ranges from 190~210kN/mm^2. The value of the Young's modulus for reinforcing bar and structural steel, which is different from that of prestressing steel, has been used. However, the difference in the value of the Young's modulus of reinforcing steel, in general, has only small influence on the calculated results concerning stress distribution of cross section and deformation of a structural member. Therefore, the value of 200kN/mm^2 may be used for all types of reinforcing steel. When it is necessary to calculate exactly the deformation of a structural member or the elongation of prestressing steel for the control of prestressing force, it is advisable to use the actual value obtained by tests.

5.3.5 Poisson's ratio

Poisson's ratio for reinforcing steel, in general, may be taken as 0.3.

[Commentary] Poisson's ratio for reinforcing steel varies according to some factors, such as measuring methods employed. However, the difference in the value, in general, does not have a great influence on the calculation for design. Therefore, the traditional value that has been employed may be used.

5.3.6 Coefficient of thermal expansion

The coefficient of thermal expansion for reinforcing steel, in general, may be regarded as the same value as that of concrete.

[Commentary] Although the coefficient of thermal expansion for reinforcing steel in steel structures and composite beams consisting of steel and concrete is, in general, $12 \times 10^{-6}/°C$, the coefficient of thermal expansion for reinforcing steel contained in concrete may be taken as the same value as that of concrete.

5.3.7 Relaxation ratio of prestressing steel

(1) The relaxation ratio of prestressing steel shall be three times the value of 1000 hours test obtained by the relaxation test.

(2) The apparent relaxation ratio of prestressing steel γ, in order to calculate the loss in prestress, in general, may be regarded as the values given in Table3.3.1

Table 5.3.1 Apparent relaxation ratio of prestressing steel, γ

Kind of prestressing steel	Apparent relaxation ratio of prestressing steel, γ
Prestressing wire and wire strand	5 %
Prestressing bar	3 %
Low-relaxation prestressing steel	1.5 %

[Commentary] (1) The relaxation ratio of prestressing steel is the value represented by the percentage of the decrease in tensile stress under constant strain to the initial tensile stress introduced in prestressing steel. As for the testing method for the relaxation of prestressing steel, the provision of 10 hours test is specified in JIS G 3536 "Uncoated Stress-Relieved Steel Wire and Strand for Prestressed Concrete" and JIS G 3109 "Steel Bar for Prestressed Concrete". However, the purpose of this testing method is to confirm the quality of material. In order to determine the relaxation ratio for design, this testing method is not suitable and it is necessary to carry out a long-term relaxation test. Therefore, it is advisable to carry out the long-term test in accordance with the JSCE standard "Test Method for Long Term Relaxation of Prestressing Steel". The JSCE test method, however, is applicable only for the condition of constant temperature within normal temperature ranging from 5°C to 35°C and constant strain in prestressing steel. When relaxation tests are carried out under the condition beyond normal temperature, it is necessary to examine various factors affecting test results, such as the level of the initial load, the loading method, the method for heating or cooling, the measuring method for temperature, and the range of variation of temperatures.

When relaxation tests are not carried out, the relaxation ratio may be determined according to the assumptions (i) and (ii) below.

(i) When the ratios of the initial tensile stress to the tensile strength of prestressing steel are 50 % and 75 %, the relaxation ratios of prestressing steel may be regarded as the values given in Table C5.3.2.

Table C5.3.2 Relaxation ratio γ_0 corresponding to initial tensile stress

Kind of prestressing steel	Specified value for the initial tensile stress divided by tensile strength	
	0.50	0.75
Prestressing wire and wire strand	$\gamma_{01} = 3$ %	$\gamma_{02} = 15\%$
Prestressing bar	$\gamma_{01} = 1\%$	$\gamma_{02} = 7\%$
Low-relaxation prestressing steel	$\gamma_{01} = 1\%$	$\gamma_{02} = 4\%$

(ii) When the ratio of the initial tensile stress to the tensile strength exists between 0.50 and 0.75, the relaxation ratio may be assumed as shown in Fig.C3.3.1

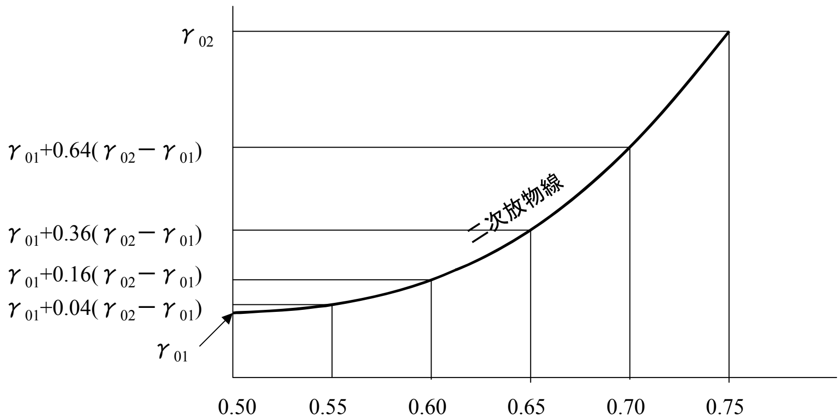


Fig. C5.3.1 Relationship between initial tensile stress and relaxation ratio

(2) The apparent relaxation ratio that is used for the design of concrete structures and members, in principle, shall be determined based on the relaxation ratio of prestressing steel, γ_0, in consideration of the influence of both drying shrinkage and creep of concrete. The apparent relaxation ratio of prestressing steel, in general, may be obtained by Eq.(C 5.3.3).

$$\gamma = \gamma_0 \left(1 - 2\Delta\sigma_{pcs} / \sigma_{pi}\right) \tag{C5.3.3}$$

where, $\Delta\sigma_{pcs}$: decrease in tensile stress of prestressing bar due to the shrinkage and the creep of concrete

σ_{pi} : tensile stress of prestressing bar just after prestressing

The apparent relaxation ratio of prestressing steel γ, for calculating the decrease in prestress stresses, in general, may be taken as the values given in Table 5.3.1.

However, the apparent relaxation ratio is influenced by the longitudinal compressive stress of concrete at the position of prestressing steel caused by permanent load, even though the initial tensile stresses of prestressing steel introduced in a structure are identical with those of other structures.

The apparent relaxation ratio becomes small or large when the longitudinal compressive stress becomes large or small, respectively. The value of the apparent relaxation ratio of prestressing steel with low relaxation is calculated by Eq. (C5.3.3), assuming that the ratio of the initial tensile stress to the tensile strength is equal to 0.7 and using the similar relaxation ratio to those of normal prestressing steel wires and wire strands.

The prestressing steels with low relaxation explained here are the prestressing steel wires and wire strands whose relaxation ratios are definitely low. It means that the prestressing steels whose relaxation ratio after 1000 hours test in accordance with the testing method of the JSCE standard is less than or equal to 2.5 %.

5.3.8 Influence of low temperature

The characteristic values of tensile yield strength and tensile strength of steel at low temperatures shall be established on the basis of respective test strengths.

[Commentary] At low temperatures, steel should not be used unless the required quality is confirmed, even if it conforms to JIS. Though the tensile yield strength and tensile strength of steel increase as the temperature drops, the values for normal temperatures may be used as the characteristic values regardless of the effect of temperature. According to past study, electric steel reinforcement has strength properties comparable to those of blast-furnace steel, provided the bar diameter is 32mm or less.

The elongation of reinforcing steel in air of both blast-furnace steel and electric steel tends to decrease at low temperatures, but the losses are not appreciable down to around -100°C, when compared with those at normal temperatures. According to past experiments, the elongation of prestressing steel in air tends to decrease at low temperatures, but at -120°C or higher, it meets the JIS requirements for normal temperatures. However, prestressing steel constantly subjected to tension with a high stress is sensitive to flaws particularly at low temperatures. It is, therefore, necessary to exercise care not to make deleterious flaws in prestressing steel.

The stress-strain curves of reinforcing steel and prestressing steel at low temperatures should be determined based on testing in general. Where the temperature can rise to the normal range during the design service life, the curves for normal temperatures should be used.

Though the modulus of elasticity tends to increase at low temperatures, the rate of increase is marginal. The modulus for normal temperatures may be used. Poisson’s ratio and thermal expansion coefficient for normal temperatures are also applicable to low temperatures.

The use of gas pressure welded joints and ark welded joints of reinforcing steel should be avoided in low temperatures, as the elongation of even blast-furnace steel can abruptly decrease at a range below temperatures between -50 to -100°C. The low temperature performance of cold-swaged joints with sleeves depends on the sleeve material and swaging method. The quality of joints should, therefore, be confirmed beforehand.

A wide variety of anchorage elements for prestressing steel are available for various methods. Their performance at low temperatures should be confirmed by testing to meet the requirements.

CHAPTER 6　LOAD

6.1　General

(1) Structures shall be designed for appropriate combinations of loads likely to act during the construction stage and the design life of the structure with consideration of limit states for performance requirements being considered.

(2) Design load shall be obtained by multiplying characteristic value of load by appropriate load factor.

(3) Combinations of design loads shall be determined according to the limit states for the required performance of the structure, as shown in Table 6.1.1.

Table 6.1.1　Combinations of design loads

Required performance	Limit state	Combination to be considered
Durability	Every limit state	Permanent load + variable load
Safety	Cross-sectional failure	Permanent load + Primary variable load + Secondary variable load
		Permanent load + Accidental load + Secondary variable load
	Fatigue	Permanent load + Variable load
Serviceability	Every limit state	Permanent load + Variable load
Earthquake resistance	Every limit state	Permanent load + Accidental load + Secondary variable load

[Commentary]　(1) Loads can be generally divided into permanent loads, variable loads, and accidental loads, according to their frequency of occurrence, the duration of action and the extent of variation.

Permanent loads act continuously on the structure, and the variation is rare or the magnitude of variation is negligible compared to the average. Dead load, earth pressure, water pressure, load produced by prestressing, shrinkage and creep of concrete should, in principle, be considered as permanent loads.

Variable loads are act frequently on the structure, and the variations in the magnitude cannot be neglected compared to the average. Live loads, loads produced by change in temperature, wind loads and snow loads should be considered as variable loads.

Accidental loads occur rarely during the design life of the structure, but once an accidental load act on the structure, serious damage will happen. Earthquake loads, collision loads, loads by strong wind, etc. should be considered as accidental loads.

Appropriate combinations shall be selected from permanent loads, primary variable loads, secondary variable loads, and accidental loads according to the limit states for performance requirements to be considered during the construction stage and the design life of the structure. Design loads shall be appropriately determined for each limit state.

(2) Although a combination of loads may be defined as a design load in some cases, design loads shall be determined for individual type of load in the Specification.

(3) Examining the limit states for verification of safety and earthquake resistance, it is practical to distinguish a set of primary variable loads from secondary variable loads. In general, different

combinations of primary variable loads are compared to make a selection. Only one accidental load may be considered when accidental loads are taken to be part of a load combination. When both accidental and variable loads are included in the combination of loads, only the secondary variable loads need to be considered. In general, permanent loads do not change over time. Permanent loads, therefore, are always combined with other types of loads, and variable loads are combined either as primary variable loads or as secondary variable loads. It must be noted that accidental loads are never combined as secondary load. When secondary variable loads are combined with primary variable loads, the probability that the maximum expected value of the secondary variable loads occurs concurrently with the maximum expected value of the primary variable loads is generally thought to be low. It is therefore desirable that characteristic values of secondary variable loads be determined appropriately (e.g., reduction of characteristic value).

In performance verification of safety of the structure, the resultant design member force can generally be given by Eq. C6.1.1.

Distinction between primary and subsidiary variable loads is not necessary in the performance verification of serviceability and durability because combinations of loads are specified corresponding to cracking, deformation, and other limit states.

$$S_d = \sum \gamma_{ap} S_p (\gamma_{fp} \cdot F_p) + \sum \gamma_{ar} S_r (\gamma_{fr} \cdot F_r) + \sum \gamma_{aa} S_a (\gamma_{fa} \cdot F_a) \tag{C6.1.1}$$

where

S_d : Design member force

S_p, S_r, S_a : Functions to calculate member force resultants due to permanent loads, primary variable loads, and secondary variable load, respectively

F_p, F_r, F_a : Characteristic values of permanent loads, primary variable loads, and secondary variable load, respectively

$\gamma_{fp}, \gamma_{fr}, \gamma_{fa}$: Load factors for permanent loads, primary variable loads, and secondary variable loads, respectively

$\gamma_{ap}, \gamma_{ar}, \gamma_{aa}$: Structural analysis factors for permanent loads, primary variable loads, and secondary variable loads, respectively

6.2 Characteristic Values of Loads

(1) The characteristic values of load shall be determined respectively for each of the limit states, for which verification is carried out.

(2) The characteristic values of permanent load, primary variable load, and accidental load used for verification of safety shall be calculated from the expected maximum values of these loads during construction and the design life of the structure. In cases where the minimum value of the load governs the design, the expected minimum value for the load shall be adopted. The characteristic values of secondary variable loads shall be determined in accordance with the combination of primary variable loads and accidental loads. The characteristic values of loads used for verification of safety against fatigue shall be determined considering varieties in the loads expected to occur during the design life of the structure.

(3) The characteristic values of loads used for verification of serviceability shall be calculated using the loads that frequently occur during construction and the design life of the structure. The values shall be appropriately chosen depending upon the limit state and the combination of loads being considered.

(4) The characteristic values of loads to be used for the verification of earthquake resistance shall be determined considering the predetermined level of earthquake resistance provided that the value must not be greater than the maximum expected value during the design life of the structure.

(5) The characteristic values of loads to be used for the verification of durability shall be calculated using the loads that frequently occur during construction and the design life of the structure.

(6) When specified or nominal values of loads are given instead of characteristic values, the characteristic value of loads for performance verification shall be determined by multiplying the specified or nominal values by the load modification factor, γ_f.

[Commentary] (2) Although the maximum or minimum loads during the return period more than the design life of the structure should be essentially employed as the characteristic value of permanent load, primary variable load, and accidental loads that are used for the verification of safety, measured data concerning the maximum or minimum loads are not necessarily sufficient to determine the characteristic values. Therefore, in the Specification, it is determined that the expected values for the maximum or minimum loads are regarded as the characteristic values of the loads.

Secondary variable loads are additionally taken into account in combination with primary variable loads or accidental loads. Therefore, the characteristic value of the secondary variable load may be smaller than the value of the same load being treated as primary variable load.

(3) The characteristic values of loads that “frequently occur”, which are used for the verification of serviceability, means that the loads with the values would not cause the limit state regarding cracking, deformation, etc.. Therefore, they shall be determined according to the characteristics of the structure, the type of load, and the limit state to be considered.

6.3 Load Factors

Values given in Table 6.3.1 may be used as load factors for calculating design loads.

Table 6.3.1 Load factors

Required performance	Limit state	Kind of load	Load factor
Durability	Every kind of limit state	Every kind of load	1.0
Safety	Cross-sectional failure	Permanent load	1.0～1.2*
		Primary variable load	1.1～1.2
		Secondary variable load	1.0
		Accidental load	1.0
	Fatigue	Every kind of load	1.0
Serviceability	Every kind of limit state	Every kind of load	1.0
Seismic resistance	Every kind of limit state	Every kind of load	1.0

* In cases when the minimum value of the permanent load other than dead load governs the design, the load factor for the permanent load may be taken to be between 0.9 and 1.0.

[Commentary] Although the load factor is required when verifying the safety of the structure, load factors are formally introduced in verification of serviceability and seismic performance since the design load was defined as in Section 6.1(2).

Variations in the unit weight of the material used and the cross-sectional dimensions of the structure could cause some variation in the self-weight (dead load), which is one of the components of the permanent loads acting on a structure. Though small, such variations need to be accounted for through appropriate load factors. The load factor for self-weight, whose variation is likely to be small, can be taken to lie between 1.0 and 1.1, when verifying safety of the structure. The load factor for the dead load can be set to 1.0 because an increase in the dead load generally causes an increase in the load bearing capacity of the structure. Here, ‘dead load’ refers to the total weight of members contributing to the load bearing capacity of the structure, and is calculated using the unit weights as shown in Table 6.4.1.

Loads on account of external fittings such as railings, pavement and ballast load, may be considered as additional dead loads and a part of the permanent loads. However, because of the more variable nature of the unit weights and the possibility of major changes in this component in the future, the load factors for such loads, should be taken to vary between 1.1 and 1.2.

The load factor for prestressing forces may be taken to be 1.0, as discussed in Chapter 15.

The load factor for accidental loads, which occurs infrequently during the design life of the structure, may be taken as 1.0, provided that its characteristic value does not fall into a dangerous decision.

6.4 Kind of Load

6.4.1 General

In performance verification, the following kinds of loads shall be considered as a general rule.

- Dead load

- Live load

- Earth pressure

- Hydrostatic water pressure

- Fluid dynamic force

- Wave action

- Prestress

- Wind load

- Snow load

- Shrinkage and creep of concrete

- Effect of temperature

- Effect of earthquake

- Loads during construction stage

- Others

[Commentary] In this Specification, a load is defined as every action causing stress or deformation in the members or structure. The loads consist of physical forces acting on the structure concentratedly or distributedly, deformation in the structure and actions causing confinement in the structure.

6.4.2 Dead load

Calculation of the nominal values of dead loads shall be made on the basis of the dimensions of the members given in drawings and specifications. The unit weight of materials used shall be taken from the values given in Table 6.4.1, unless the actual weights of the members are available, in which case they can be used.

The nominal values of dead loads shall be divided into two components - fixed dead loads and additional dead loads. The load modification factors to obtain characteristic values from nominal values, in general, may be taken as 1.0 for fixed dead loads. The load modification factors for the additional dead loads shall be determined considering the variation in the magnitude of loads.

Table 6.4.1 Unit weights of materials

Material	Unit weight (kN/m^3)	Material	Unit weight (kN/m^3)
Steel, Cast steel, Forged steel	77	Concrete	22.5-23.0
Cast iron	71	Cement mortar	21.0
Aluminum	27.5	Wooden material	8
Reinforced concrete	24.0-24.5	Bituminous material	11
Prestressed concrete	24.5	Asphalt pavement	22.5

[Commentary] Dead load can be defined as load caused by the self-weight of constituent materials and attached to a structure. In principle, the characteristic values of dead loads should be determined using actual data, which shows some extent of variation in unit weights. However, since the scatter in unit weights of materials is not very large, the nominal values of dead loads of the structure may be determined using the dimensions given in drawings and specifications.

Since the unit weight of lightweight concrete generally varies according to the type and mix proportion of aggregates used, the values of the unit weight of lightweight concrete shall be determined considering these factors.

6.4.3 Live load

The characteristic values of live loads shall be determined taking into consideration the characteristics of the variation in the magnitude of loads.

For the examination of fatigue failure, the loads that have adequate magnitude and corresponding equivalent repeated cycles shall be determined in consideration of all of variable loads during the design life of structures.

[Commentary] The term "live load" refers to the load of an automobile, train, crowd of people, etc., moving on a structure plus the resulting dynamic response such as impact and braking loads. A dynamic response due to live loads may be replaced with a static load by evaluating the ratio of increase from a static response appropriately. If there are standard-specified values of live loads as for railway bridges and road bridges, the characteristic values are determined by multiplying them by load modification factors, ρ_f, according to the verification item.

6.4.4 Earth pressure

(1) Earth pressure acting on a structure shall be determined by taking into consideration overburden thickness, mass of soil, loosening of soil, construction methods, etc.

(2) Earth pressure is classified into vertical earth pressure and lateral earth pressure. Lateral earth pressure consists of static earth pressure, active earth pressure and passive earth pressure. The earth pressure to be used in performance verification shall be determined depending on the type and stiffness of structure, the limit state being considered according to the type of soil. In that case, changes over time in the state of the structure and the ground should be taken into account.

(3) When nominal values of earth pressures are already determined, the characteristic values of earth pressures shall be obtained by multiplying these nominal values by the load modification factor, ρ_f.

[Commentary] Earth pressure is classified into vertical earth pressure and lateral earth pressure. Lateral earth pressure consists of static earth pressure, active earth pressure and passive earth pressure. Vertical earth pressure and static earth pressure may be estimated taking into consideration the conditions of the ground around the structure. Active and passive earth pressures are interaction due to the deformation of the structure as shown in Fig. C6.4.1, being indirect loads. Earth pressures due to surcharge on the ground shall be determined considering the distribution of surcharge and overburden thickness.

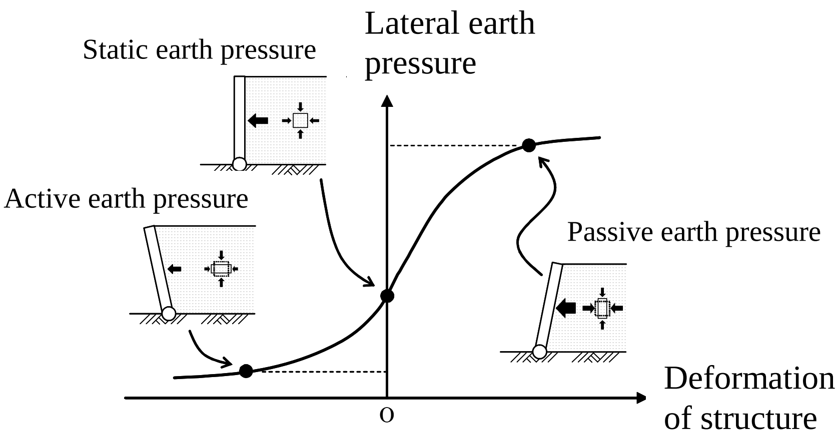


Fig. C6.4.1 Lateral earth pressure as interaction with a structure

Active earth pressure and passive earth pressure are earth pressures that occur when a structure is deformed. Because of differences in the behavior of the structure, therefore, the two types of earth pressure need to be distinguished when considering earth pressure. Earth pressure in general, including vertical earth pressure and static earth pressure, may also change because of not only deformation of the structure but also changes over time in the state of the ground. It is desirable, therefore, that earth pressure assumed at the design stage be determined by taking into consideration changes over time in the state of the ground and the structure. Expected earth pressure changes include earth pressure changes due to the aging of the structure, earth pressure changes due to changes of soil over time, earth pressure changes due to coupling effect with groundwater and earth pressure changes due to excavation or other earth-disturbing work carried out in the vicinity of the structure. Design earth pressure should be preferably determined in view of these earth pressure changes over time. Since, however, it is still difficult at present to calculate these changes, it is recommended that design earth pressure be determined on the basis of experiment results or measurement data. If that is not possible, design earth pressure may be determined on the basis of basic theories such as Coulomb's earth pressure theory or Rankine's earth pressure theory.

6.4.5 Water pressure, fluid dynamic force and wave pressure

(1) Loads on account of hydrostatic pressure, dynamic water pressure during earthquake, fluid dynamic force, and wave pressure shall be determined depending upon the type of structure, environmental conditions, geometries, which are shape and size of structural members, etc.

(2) The characteristic values of forces on account of dynamic water pressure during earthquake shall be determined taking into consideration the shape of the structure, the water depth, etc.

(3) The characteristic values of force on account of wave pressure shall be determined taking into consideration the location and shape of the structure and the wave properties.

[Commentary] (1) The characteristic values of hydrostatic pressure, p_w, acting on the structure in contact with water or below the ground water level shall, in general, be obtained using Eq. C6.4.1.

$$p_w = w_0 \cdot h \qquad \text{(C6.4.1)}$$

where, p_w : characteristic value of hydrostatic pressure (kN/m^2)

h : water depth (m), to be determined according to considered limit states

w_0 : unit weight of water (kN/m^3)

However, in the case of structures below the groundwater level, static water pressures may be appropriately reduced from values given by the above equation when it is obvious from investigation on pore water pressures, etc, that the actual distribution of hydrostatic pressures is different from that calculated by Eq.C6.4.1. As for structures in contact with liquids other than water, the hydrostatic pressures acting on the structures may be estimated using Eq.C6.4.1, replacing the unit weight of water by the unit weight of that liquid.

The characteristic values of fluid dynamic force due to flow of water, P_w, shall be obtained using Eq. C6.4.2.

$$P_w = \frac{1}{2}\rho \cdot v^2 \cdot C_v \cdot A \qquad \text{(C6.4.2)}$$

where, P_w : characteristic values of fluid dynamic force (N)

ρ : density of water (1000 kg/m^3)

v : speed of flow (in m/s), to be determined in accordance with the considered limit states

C_v : coefficient of resistance, or resistance factor, depending on the cross section of the structure

A : projected cross-sectional area of the member perpendicular to the direction of flow (m^2)

6.4.6 Prestress

The characteristic values of prestressing loads shall be determined according to Chapter 10.

6.4.7 Wind load

(1) Wind loads shall be determined keeping in view the type of structure, environmental conditions, and geometries, which are shape and size of structural members.

(2) Dynamic effects duet to wind shall also be taken into account in case of structures and members are flexible.

[Commentary] (1) The characteristic values of wind loads, W, shall, in general, be obtained using Eq. C6.4.3.

$$W = \frac{1}{2}\rho \cdot v_v^2 \cdot C \cdot A \qquad \text{(C6.4.3)}$$

where, W : characteristic values of wind loads (N)

ρ : density of air, which may be taken to be 1.25 kg/m^3

v_s : design wind speed (m/s), to be determined according to intended limit states

C : coefficient of drag, depending on the shape of cross- section of structural member

A : projected cross-sectional area of the member perpendicular to the direction of wind (m^2)

Design wind speed shall be defined for each limit state taking into account recorded data of wind speed, the design life of the structure, and the return period of a certain wind speed. In general, design wind speed may be determined on the basis of the 10-minute average wind speed at the ground surface or at that at a height of 10 m above the sea water surface with appropriate corrections for effects of the elevation of the site, the horizontal and vertical length of the structure, the ground surface roughness, the shielding and convergence effect caused by nearby features, wind speed variations in time and space, etc.

Table C6.4.1 may be used to determine the coefficient of drag for the different cross-sectional shapes of a structural member.

(2) Wind-induced vibrations should be accounted for in the case of flexible structures and members, such as suspension bridges and cable-stayed bridges, and of slender members such as hangers.

Table C6.4.1 Coefficient of drag

Shape of cross section	Coefficient	Shape of cross section	Coefficient
Circular section	1.2	Wind direction; Rectangular section (2 × 1, corner radius r, width d)	1.5(1.1)
Flat plate, or other section similar to (I-section)	2.2	Square section	2.1(1.5)
Flat plate, or other section similar to (flat plate)	1.8	Rectangular section (1 × 0.7)	2.7
Square section (diagonal direction)	1.5	Rectangular section (1 × 0.5)	2.3(2.1)

6.4.8 Snow load

The characteristic values of snow loads shall be determined according to the location and characteristics of the structure and the environmental conditions in the region.

[Commentary] In cases where the snow loads need to be considered, the characteristic values of the snow loads, *SN*, should be determined on the basis of actual data on snow fall, etc. in the region. *SN* may in general be obtained using Eq. C6.4.4.

$$SN = w_s \cdot z \cdot I \qquad \text{(C6.4.4)}$$

where, SN : characteristic values of the snow loads (N/m^2)

w_s : unit design weight of snow (N/m^3)

z : design depth of snow above the ground (m)

I : factor considering the gradient, as defined below:

$$I = 1 + (30 - \theta)/30$$

In cases when $\theta < 30^{\circ}$ and $\theta > 60^{\circ}$, I should be taken as 1.0 and 0.0, respectively.

θ : gradient of the surface of the structure where snow falls ($^{\circ}$)

Unit weight of snow varies greatly due to properties of snow and conditions of snow fall. Since the depth of snow varies greatly by the region, and is also related to snow removal procedures, which may be carried out, it is desirable that snow loads should be determined on the basis of not only the observed snowfall data in the region, but also the maintenance of the structure and the design life of the structure.

6.4.9 Effect of shrinkage and creep of concrete

(1) The characteristic values of shrinkage and creep of concrete shall be determined taking into consideration material properties, environmental conditions, geometries, which are shape and size of the structural member. The values are given in Section 5.2.8 and 5.2.9.

(2) In design for statically indeterminate structures, such as rigid frames and arches, the shrinkage and creep of concrete may, in general, be assumed to be uniform throughout the cross section of members.

[Commentary] In general, the influence of shrinkage and creep of concrete may be considered in verification for serviceability and safety against fatigue failure. In the case of statically indeterminate structures, the deformation of a member under the influence of shrinkage or creep of concrete generates statically indeterminate forces. The influence of such forces, therefore, must be taken into consideration. Also, in the case of statically determinate structures, the influence of such forces needs to be taken into account if deformation is restrained or shrinkage deformation varies considerably in a cross section.

6.4.10 Effect of temperature

(1) The effect of temperature shall be determined according to the type of the structures, environmental conditions and the geometries, which are shape and size of structural members.

(2) In the design of statically indeterminate structures, such as rigid frames and arches, any increase and/or decrease in temperature may, in general, be assumed to be uniform throughout a cross-section of members. If it is clear that the temperature cannot be assumed to be uniform, the difference in temperature among members or parts thereof shall be appropriately accounted for. The characteristic values of increase and decrease in temperature shall be defined based on the differences between the annual average temperatures and the highest or lowest value of the monthly average temperatures.

[Commentary] The difference between the annual average temperatures and the highest or lowest values of the monthly average temperatures in Japan lies between -15°C and 15°C, as represented in Table C6.4.2. These values, i.e. -15 °C and 15 °C, have thus, been recommended for use as the characteristic values of increase and decrease in temperature, in general.

Table C6.4.2 Regional differences between annual average temperature and monthly average temperature (ºC)

District	Kyusyu	Shikoku	Chugoku	Kinki	Chubu		Kanto	Tohoku	Hokkaido		
					General	Pacific coast			General	Pacific coast	Inland
The highest value	+11.5	+11.5	+12.5	+12.5	+13.0	+12.0	+12.5	+14.0	+14.0	+13.0	+15.0
The lowest value	-11.5	-11.0	-11.5	-12.0	-13.0	-11.5	-12.0	-13.0	-13.5	-13.0	-15.5

Temperature differences occur between members exposed to and those not exposed directly to sunlight. If the statically indeterminate forces caused by these differences in such structures cannot be neglected, the characteristic values of temperature difference shall be determined taking into consideration the location of the structure and climate conditions around the structure. The effect of temperature difference between inside and outside of the structure should be taken into account in the cases of such as low- or high-temperature storages.

In some cases the temperature at the time of completion of the structure does not coincide with the annual average temperature. When the structure completed in summer or winter, the variations of temperatures are different from those given here at the initial stage. However, thermal stresses vary up and down around the annual average temperatures because of creep of concrete. Therefore, it is not necessary to consider the variation of temperatures at the construction stage. Calculations of camber, extent of expansion and contraction, etc. should, however, be based on the temperature at the construction stage.

6.4.11 Influence of earthquake

(1) To allow for the influence of earthquakes, all ground motions and all loads generated by the ground motions shall be considered.

(2) As for the direction of ground motion, ground motions in only two orthogonal horizontal directions shall be considered. Depending on the characteristics of the structure under consideration, however, it may also be necessary to consider the ground motion in the vertical direction.

(3) As a general rule, ground motions used for the purpose of seismic performance verification shall be expressed in the form of time-history acceleration waveform.

(4) Ground motions shall be defined at the engineering bedrock surface at the construction site.

(5) As a general rule, ground motions shall be defined in view of such factors as the level of seismic activity at and around the construction site, hypocenter characteristics and the propagation characteristics of ground motions from the hypocenter to the construction site so that the influence on the structure is exaggerated. In general, ground motions may be defined on the basis of two or more observed earthquake waveforms or by methods that takes into consideration the crust failure process in the hypocentral area.

(6) For Level 1 ground motions, it may be assumed that the return period occurs

several times during the life of the structure.

(7) In the case of Level 2 ground motions, of the two types of ground motions listed below, the one with greater influence on the structure may be adopted. The ground motions to be considered shall be the strongest ground motion expected at the construction site.

(i) Ground motions caused by an inland active fault directly under or near the construction site

(ii) Ground motions due to a large-scale near-coast plate boundary earthquake

[Commentary] (1) In general, to evaluate the influence of earthquakes, the following factors are taken into consideration:

1) Mass of the structure and the mass of the load

2) Dynamic interaction between the structure and the ground

3) Dynamic water pressure during earthquake

4) Liquefaction of the ground and ground flow caused by liquefaction

From the factors listed above, necessary factors need to be selected according to the type of structure, structural conditions, environmental conditions around the structure to be constructed, etc. Masses that generate inertia forces because of earthquake-induced shaking include the mass loaded on the structure as well as the mass of the structure itself. Generally, in view of the case where the loaded mass acts simultaneously with earthquake loading, permanent loads and secondary variable loads need to be taken into consideration.

The dynamic interaction between the structure and the ground results from the difference in the dynamic response characteristics between them. Structures that require the consideration of this interaction include bridge abutments, retaining walls, underground structures and foundation structures such as piles and caissons.

In the case of fluid-containing structures such as tanks, dynamic water pressures caused by the loaded mass and dynamic water pressure resulting from sloshing of the fluid during earthquake need to be taken into consideration.

As for the liquefaction of ground, the basic rule in seismic design is to take measures to prevent liquefaction. If taking such control measures is technically difficult or highly uneconomical, it is necessary to design the structure by giving careful consideration to the influence of liquefaction on seismic performance of the structure. If the ground surface is inclined or if uneven earth pressure is always acting, liquefaction might cause ground flow. The influence of this, therefore, needs to taken into consideration.

(2) In general, seismic force is dominated by horizontal ground motions. When seismic performance is verified, therefore, only horizontal ground motions needs to be considered. Although horizontal ground motions act in any direction, seismic performance of the structure may be verified independently in two horizontal directions. In the case, however, of a curved bridge, a structure subjected to torsion or a column acted upon by a highly eccentric axial force, multi-axis response may result even if ground motion is unidirectional. In such cases, the influence of multi-directional inputs of the ground motions should be considered.

If it is judged that the influence of the vertical component of ground motions cannot be ignored in connection with Seismic Performance Grade 2 and 3 because of the type and shape of structure, its stiffness distribution, etc., it is necessary to perform verification against seismic force in the vertical direction. The magnitude of the earthquake input may be taken as 1/2 to 2/3 of the horizontal input on the basis of the records on past earthquakes.

(3) In order to properly evaluate the level of safety of a concrete structure during an earthquake, it is necessary to perform a nonlinear dynamic response analysis using time-history ground motion waveform data as inputs. Ground motion waveforms can be expressed with displacement, velocity and acceleration. Since, however, it is common practice to give an input by acceleration waveform in vibration equations, it is required that a waveform input expressing ground motions be given in the form of time-history acceleration waveform.

(4) In order to properly evaluate the influence of the surface layers of the ground at the construction site of a structure, it is required that ground motions to be used for the verification of seismic performance be defined at the depth of engineering bedrock surface. The engineering bedrock surface may be taken as the top surface of a continuous layer of the ground with a shear wave velocity (in the case of infinitesimal strain) of about 400 m/s or more (generally, an SPT N-value of 50 or more for sandy soil and 30 or more for cohesive soil).

(5) to (7) As a general rule, ground motions to be used for verification purposes must be determined in view of such factors as the level of seismic activity at and around the construction site, hypocenter characteristics, and propagation characteristics of ground motions from the hypocenter to the construction site. The preparation of such a time-history acceleration waveform, however, is a laborious task requiring, for example, detailed surveys on the state of the ground at the construction site, surveys on the location and activity level of active faults, and surveys on the influence of plate boundary earthquakes. An alternative is to use a number of ground motion waveforms of past earthquakes, but it is not possible to judge whether the ground motions under consideration is appropriate for use in seismic performance verification. If ground motions at the construction site cannot be defined appropriately, the simulated waveform of ground motions containing vibration components that exaggerates the influence on the structure be used.

One way to concretely define acceleration waveforms of Level 1 and Level 2 ground motions on the basis of past records is a probabilistic prediction method. The return period as an index for the intensity of ground motions is used to evaluate variability of the distance-attenuation relationship by which the intensity of ground motions is evaluated by assuming a logarithmic normal distribution, and the probability of the occurrence of an earthquake is determined as that of a Poisson process event. In order to clearly define earthquakes corresponding to Level 1 and Level 2 ground motions, this Specification requires that ground motions be expressed in terms of the return period, and indicates rough values as stipulated above.

As examples of simulated ground motion waveforms, Fig. C6.4.2 shows a Level 1 ground motion waveform, and Fig. C6.4.3 and Fig. C6.4.4 show ground motion waveforms of Level 2 inland earthquakes (Nos.1 and 2) and Level 2 plate boundary (ocean trench) earthquakes (Nos.1 and 2). Fig. C6.4.5 and Fig. C6.4.6 show absolute acceleration response spectra (damping constant, h = 0.05) of Level 2 ground motion waveforms. These are examples of ground motions at the engineering bedrock surface (free surface).

The ground motion waveform No.1 due to inland earthquake (Fig. C6.4.3 (a)) was determined on the basis of inland earthquake records by the following procedure described below.

- Acceleration response spectra of many observation records are converted by using the distance-attenuation relationship to ones directly above a fault, and modifications are made so that the probability of non-exceedance among those samples becomes 90%.

- To determine the phase characteristics to be used when making a time-history acceleration waveform, the asperity and the failure starting point were varied parametrically in a 40km×20km fault plane and the phase characteristics were defined in view of the failure process of the fault, and a waveform was synthesized.

The ground motion waveform No.2 due to inland earthquake (Fig. C6.4.3 (b)) was defined by calculating the ground motion (NS component) at the free bedrock surface from the data recorded by a strong motion seismograph at Kobe Port Island (installation point: GL = -83m) during the Hyogoken Nanbu Earthquake of 1995.

The ground motion waveform No.1 due to plate boundary (ocean trench) earthquake (Fig. C6.4.4 (a)) was defined by the following procedure described below.

- The acceleration response spectrum of the observation record is corrected so that the magnitude of the earthquake became 8 (M8) or so and the epicentral distance became about 40 to 50km by using the distance-attenuation relationship.

- Phase characteristics were defined by an empirical method by using ones corresponding to a magnitude 8 (M8) earthquake and an epicentral distance of 50km, and a waveform was synthesized.

The ground motion waveform No.2 due to plate boundary (ocean trench) earthquake (Fig. C6.4.4 (b)) was determined by calculation as a simulated ground motion waveform at a free bedrock surface near a fault (epicentral distance: about 50km) based on the fault model of the assumed Tokai Earthquake (moment magnitude: MW = 8.0) proposed by the Central Disaster Prevention Council, Japan.

It is generally known that the magnitude of ground motions varies locally. If the influence of the level of seismic activity, the distance from the hypocenter, etc., in the region concerned can be evaluated appropriately, waveforms obtained by multiplying these acceleration waveforms by an appropriate reduction factor may be used.

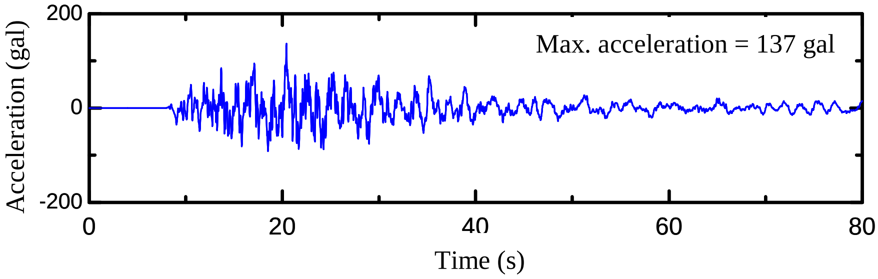
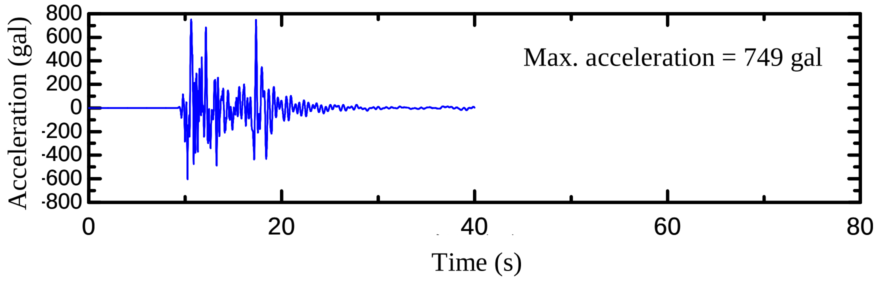
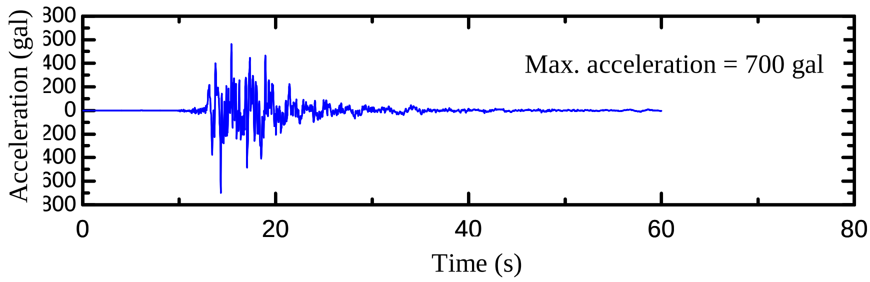


Fig. C6.4.2　Example of time-history acceleration waveform of Level 1 ground motion

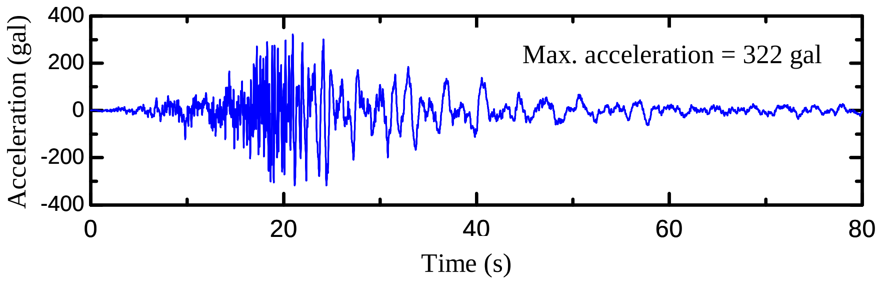


(a) No.1

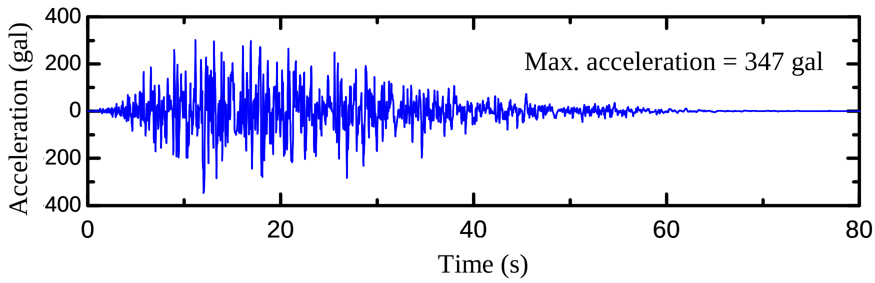


(b) No.2

Fig. C6.4.3 Example of time-history acceleration waveform of Level 2 ground motion
(due to inland earthquake)



(a) No.1



(b) No.2

Fig. C6.4.4 Example of time-history acceleration waveform of Level 2 ground motion
(due to ocean trench earthquake)

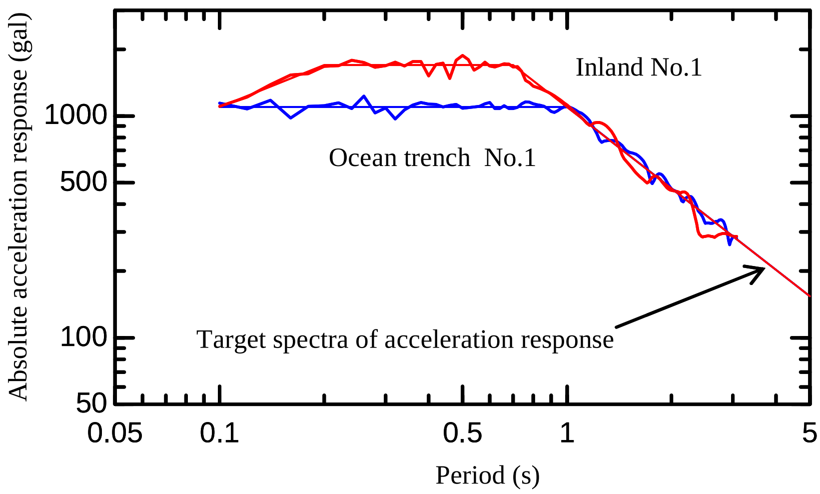


Fig. C6.4.5 Acceleration response spectra of Inland Earthquake No.1 and Ocean Trench Earthquake No.1 (damping constant, *h* = 0.05)

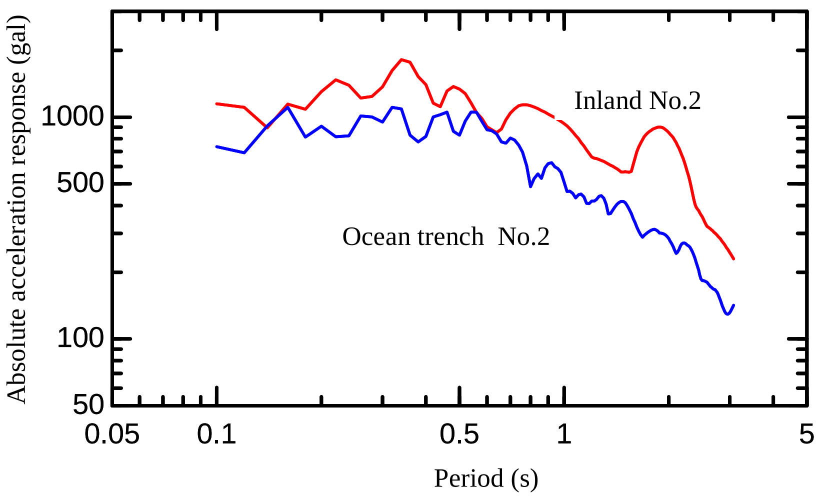


Fig. C6.4.6 Acceleration response spectra of Inland Earthquake No.2 and Ocean Trench Earthquake No.2 (damping constant, *h* = 0.05)

6.4.12 Loads during construction stage

The following loads shall be considered to act on the structure, or part of it, during construction: dead load on account of self weight of the structure, loads on account of construction equipments, wind loads, and earthquake loads. The actual values of these loads should be chosen in consideration to the method of construction, the structural system during construction, length of construction period.

[Commentary] Loads during the construction stage are loads that act on the structure during construction. If loads during the construction stage are different from those after the completion, it is necessary to determine the loads during the construction stage appropriately with consideration of the structural system under construction and the construction method.

Wind loads and earthquake loads during the construction stage may be reduced from the characteristic values determined according to 6.4.7 and 6.4.11, in consideration with length of construction period and importance of the structure.

6.4.13 Other loads

In cases when it is necessary to consider other loads than those specified above, the characteristic values of such loads shall be determined in accordance with the actual situation of loads.

[Commentary] Impact load is one of the other loads mentioned in this clause. In the case of impact by automobiles, the loads may be evaluated and taken to act in a manner similar to static load. In the case of impact by flying objects, on the other hand, the particular failure mode of the structure should be taken into consideration. In the former case, performance verification against safety can be done by converting the impact loads to static loads considering the mechanical equivalence. In the latter case, three special limit states caused by impact of flying objects should be considered: incomplete penetration, backside peeling (i.e., concrete bursting on the back side of impact) and complete penetration. Characteristic values of velocity, weight and size of the flying objects shall be determined when carrying out the performance verification.

In some structures, the ground conditions may be changed due to ground settlement after the completion of the structure, resulting in movement and rotation of supporting points of the statically indeterminate structures. Such effects shall be considered appropriately.

CHAPTER 7 CALCULATION OF RESPONSE VALUES

7.1 General

(1) Response value calculation shall involve modeling the structure to be constructed according to the shape of the structure, support conditions, the state of loads and the limit states to be considered, perform a structural analysis by using an analysis model of proven reliability and accuracy and calculating response values such as sectional force, deflection, stress, strain, crack width, etc., according to verification indices.

(2) The structure shall be modeled appropriately according to the analysis method used for verification.

(3) Loads shall be modeled appropriately according to the load characteristics and the influence on different types of limit states to be taken into consideration. Loads may be modeled equivalently or conservatively by, for example, simplifying the state of load distribution or replacing dynamic loads with static loads.

(4) In response value calculation, loads shall be considered so that response values are verified under the least favorable conditions.

(5) Sectional forces such as bending moment, shear force, axial force and torsional moment and deflection shall be calculated in accordance with an appropriate analytical theory according to each limit state.

[Commentary] (1) Dealing with matters related to the response value calculation methods used for the verification of the limit states related to each required performance attribute specified in Chapter 4, this chapter describes modeling, the selection of structural analysis methods, and methods of calculating design response values. Many limit values for the limit states related to each required performance attribute are given as constraints, while design response values are obtained as a result of design work. The calculation of design response values, therefore, is important in ensuring the reliability of verification results. In order to ensure the reliability of verification results, therefore, response values must be calculated in strict accordance with the intent of this chapter. When calculating response values by using a sophisticated analysis method such as a nonlinear analysis method, it is necessary to ensure the required quality and accuracy by referring to the Design: Appendix, Part 3, of this Specification, etc.

(2) Fiber modeling used to be a widely used approach to structural modeling, but modeling by the finite element method also has become a commonly used approach because of recent advances in numerical analysis technology. In the finite element method, the shape of the structure to be analyzed can be modeled with considerably high accuracy by using two-dimensional or three-dimensional elements. The method, however, also poses some problems such as the need for huge computing capacity. It is also necessary to keep in mind that the accuracy of obtained response values varies among different analysis methods.

(3)(4) If response values of a structure are calculated by replacing dynamic influences such as the impact of vehicles, wind and earthquakes with static loads, load distribution patterns and the structure must be modeled so that static response values are equivalent to or more conservative than dynamic response values. Response values concerned vary depending on the type of load. When the influence of an earthquake is evaluated, post-yielding displacement is usually used as an index for response values, and a nonlinear analysis approach is required. Because not only earthquake response acceleration but also the relationship between the period characteristics of the structure and loads are important factors affecting response displacement, it is necessary to model the loads and the structure appropriately.

(5) An analysis theory can be broadly classified either as a linear (material linearity) theory or a nonlinear theory depending on whether the assumed stress–strain relationship for the material under consideration is linear and either as a first-order theory or a second-order theory depending on whether the secondary effect (geometric nonlinearity) of deformation is ignored. Linear analysis is performed according to the first-order theory of material linearity, and combination cases are analyzed by nonlinear analysis Analytical theories applicable to the limit state of a mechanism include plastic analysis assuming a rigid-plastic body, etc.

In a verification, it is not always necessary to apply the same structural model or analytical theory to all limit states. Structural models and analytical theories suitable for different limit states may be used. In such cases, settings need to be determined appropriately by giving consideration to the influence on response values and accuracy. It is also necessary to select an appropriate structural analysis factor (γ_a) according to the analytical theory used for response value calculation.

7.2 Modeling

7.2.1 General

(1) Modeling shall involve determining the extent of the region to be analyzed and the dimensionality of analysis and modeling structural members according to the load response characteristics of the structure under consideration.

(2) In modeling, the extent of the region to be analyzed consisting of the structure of interest, ground and boundary elements, etc., shall be defined according to the extent of the region in which response occurs.

(3) If an analysis region including ground is defined, modeling shall be performed so that the influence of such coverage can be evaluated appropriately.

[Commentary] When modeling a structure, it is necessary to define an analysis region so that the region in which load response occurs is covered. If the influence of such coverage is small or if such influence can be allowed for by boundary conditions, the analysis region may be divided into structural elements for modeling purposes.

For example, if the influence of an earthquake is to be considered, the analysis region is the entire structural system including the foundation and the surrounding soil because the response of a structure during an earthquake is strongly affected by the surrounding soil, etc. Depending on the type and characteristics of structure and ground, however, the dynamic interaction between the structure and the soil can be ignored or modeled appropriately. In such cases, it is possible to model the structure and the soil separately and perform an analysis of the structure separately. If the influence of the surrounding soil is expressed simply with soil springs, etc., only the structure is included in the analysis region, and soil springs are treated as boundary elements.

Even when the influence of earthquakes is not taken into consideration, if there is a need to define an analysis region including soil, modeling must be done by an appropriate method by referring to the soil modeling during an earthquake described in Section 7.3.4.

7.2.2 Modeling of structure

(1) A structure shall be modeled two-dimensionally or three-dimensionally as an assembly of structural members in view of its shape, etc.

(2) Structures may be analyzed assuming them to be made of simplified elements such as slabs, beams, frames, arches, shells and their combinations.

(3) Members should be modeled as linear or planar members.

(4) Linear members or planar members shall be modeled by use of finite elements or fibers.

[Commentary] (1) Structures should be modeled as an assembly of member models in which columns and beams are modeled as linear members and members with planar spread such as wall or floor are modeled as planer members such as slab or shell. Strictly speaking, structures should be modeled three dimensionally, because it is composed of members connected in three dimensions. Two dimensional modeling of the structure may, however, be applied if only the response behavior in a two dimensional plane is considered according to the direction of input earthquake ground motion and characteristics of structural response.

(2) A structural analysis of a structure may be performed by assuming a simplified structural analysis model consisting of slabs, beams, columns, rigid frames, arches, shells or their combinations. Such structural analyses and related verifications should be performed as specified in the Design: Standards, Part 1, of this Specification.

(3) and (4) There are a number of methods for modeling a structure. Modeling is also closely related to analytical theories. It is therefore necessary to take the applicability of analytical theories into consideration when selecting a modeling method. Modeling methods can be broadly classified into two types: the method of modeling a structural member as a set of infinitesimal elements and the method of modeling a structural member as a single element. In the finite element method, a complex problem is solved by approximation by dividing the region to be analyzed into infinitesimal elements and expressing the solution in each region with a relatively simple function. From the point of view of modeling, all methods can be deemed to fall into the category of the finite element method. Some models, however, such as mass system modeling is usually distinguished from the finite element method. In this Specification, the method of modeling a structure as a system consisting of two- and three-dimensional element regions is defined as finite element modeling, and the method of modeling a structure as a system consisting of one-dimensional element regions is defined as fiber modeling.

Finite elements that have two-dimensional element regions can be classified into triangular plane elements, quadrilateral plane elements and planar shell elements. Planar elements can be used for modeling in a two-dimensional analytic space, and planar shell elements enables modeling in a three-dimensional analytic space. Finite elements that have three-dimensional element regions, which can be classified into solid elements such as pentahedrons and hexahedrons, cylindrical shell elements and curved shell elements, can be used for modeling in a three-dimensional analytic space. In the finite element method, the stress–strain relationship is applied as a constitutive law. One-dimensional element regions, which can be classified as linear elements (trusses and beams), can be used for modeling in a three-dimensional analytic space. For beam elements, for example, various relationships such as the moment–rotation angle relationship, moment–curvature relationship and the stress–strain relationship can be used as constitutive laws. A beam model in which a beam element is divided into infinitesimal fibers so that various relationships such as the stress–strain relationship can be used as constitutive laws is generally referred to as a fiber model.

When a structural boundary or member-to-member joint is to be modeled, spring elements or joint elements are widely used.

Rod members can be modeled by use of fibers or finite elements. Because different response values can be calculated with different models, it is necessary to perform modeling by an appropriate method, taking into consideration response values to be used for verification. In general, however, modeling by using beam element is effective, since the computation time becomes shorter and accuracy does not decrease much. Also, planar members should be generally modeled by using finite elements, but they can also be modeled by use of linear members if an appropriate method is used.

7.2.2.1 Modeling of members by use of finite elements

(1) Members of a structure shall be modeled by using an appropriate set of finite elements according to the response characteristics of the structure.

(2) The dimensions of elements shall be determined appropriately in view of the limit states to be taken into consideration and the compatibility with the material models used.

(3) I a planar member region where in-plane force is predominant over out-of-plane force and the distribution of stress is mostly uniform, relatively large elements that are several times as wide as they are thick may be used.

(4) When evaluating the influence of earthquakes, etc., in a region of a member where deformation is concentrated such as an end region of a member where the maximum bending moment acts, the longitudinal dimension of any element should not be greater than the effective depth of the member and should be around 20 cm.

[Commentary] (1) Although all members can be modeled if solid elements are used, it is necessary to fully understand the characteristics of the elements used. If the deformation of a bar member is limited to a two-dimensional analytic space, modeling by use of planar elements is a good approach. If flexural deformation is dominant and shear deformation can be ignored, the use of linear elements usually reduces computing time. Modeling by use of linear elements must be performed in accordance with Section 7.2.2.2. If the deformation of a planar member is dominated by out-of-plane deformation, laminar shell elements may be used for the superposition of planar elements capable of resisting planar stress in the direction of plate thickness. In general, laminar shell elements reduce computing time. If the deformation of a member is limited to two-dimensional deformation, modeling by use of planar elements is acceptable.

(2) and (3) In the finite element method, usually the stiffness method, in which a displacement function is given, is used. If first-order elements are used, the displacement interpolation function becomes a first-order equation. Consequently, the deformation of the member of interest is polylinearly approximated, and the deformation of the member might be underestimated (i.e., stiffness might be overestimated) depending on the degree of refinement of meshing. In such cases, it is necessary to think of meshing refinement or the use of higher-order elements. Prior to element decomposition, therefore, it must be ascertained that the deformation of the member can be interpolated with a sufficient level of accuracy.

The element decomposition scheme and element dimensions need to be determined in view of the following:

1) In element decomposition, effort should be made to avoid random meshing as much as

possible. The aspect ratio of elements should be made as close as possible to 1:1, and utmost effort should be made to avoid flat elements. For example, if a low-order displacement function is used in cases where localized deformation occurs in the member of interest, the localization behavior of deformation might be affected. As another example, in the case of a material model using failure energy, analytical results might be affected because of a difference in the method of equivalent length determination.

2) Element decomposition should be performed taking into consideration concrete cover and reinforcement patterns. For example, if reinforcement is modeled in a single element in a distributed reinforcement model, element decomposition needs to be performed so that the location of the reinforcement roughly coincides with the center of gravity of the element. If reinforcement is modeled in multiple elements, element decomposition needs to be performed so that the location of the reinforcement roughly coincides with the center of gravity of the elements in which the reinforcement is distributed.

3) Element decomposition should be performed in view of the expected stress gradients.

(2) When concrete strain exceed the strain corresponding to the compressive strength due to the large deformation of member, the concentrated region of progressive compressive deformation (softening region) appears in the part of the member. It is recognized that the size of the softening region is almost equal to the size of concrete cylinder for compressive test (20cm). On the contrary, elastic recovery occurs due to the stress release in the neighborhood of softening region. Therefore, when element size is larger than the softening region, the analysis can not reasonably predict the deformational behavior of real member. Since the stress strain relationship described in this chapter was obtained from the specimen with size almost equal to that of concrete cylinder specimen, it is necessary to modify the stress strain relationship of concrete in softening region when the element size differs significantly. It is, however, confirmed that the influence on the response is small if the element size is 0.5 to 2.0 times the size of concrete cylinder.

If the structure under consideration is large, it is generally difficult to handle element dimensions of 20 cm or so in analysis because of limitations related to analytic capacity and analysis time. If the structure under consideration is small, element dimensions of 20 cm or so may make the mesh too coarse. In such cases, by using a model allows for failure energy in dealing with the stress–stress relationship for concrete, it can be made possible to freely set element dimensions.

If the finite element method is used as a nonlinear analysis method, it is advisable to refer to the Design: Appendix, Part 3, of this Specification.

7.2.2.2 Modeling of members by use of linear elements

(1) Bar members shall be modeled as linear elements with axial stiffness and flexural stiffness. In general, shear deformation may be ignored.

(2) Mechanical characteristics of a bar member shall be either determined directly from the stress–strain relationship for the material or allowed for by defining a mechanical model reflecting the dimensions of the member and the mechanical characteristics of the material.

(3) In general, a mechanical model of a bar member shall be capable of evaluating the mechanical characteristics listed below, and the skeleton curve may be expressed by a trilinear model that takes into consideration the softening gradient after the maximum load-carrying capacity point is reached, which is determined by drawing a line from the

origin through (i) to (ii).

(4) If the out-of-plane deformation of bar members or planar members whose shear deformation cannot be ignored or the in-plane deformation of planar members is modeled with linear elements, appropriate mechanical characteristics shall be set, and a fiber model capable of dealing with shear deformation shall be used.

[Commentary] (1) In a linear analysis of a statically determinate member, sectional forces can be calculated from the equilibrium of forces. In an analysis of a statically indeterminate member, however, the compatibility of deformation is necessary, so modeling that allows for axial stiffness and flexural stiffness is needed. In such cases, nonlinearity must be taken into consideration if stress redistribution is expected. If the correlation between axial force and moment can be ignored, axial stiffness may be ignored.

(2) In order to reproduce the dynamic response behavior of a structure, it is necessary to use a model capable of accurately expressing the mechanical characteristics of the structural members. There are two methods of modeling the mechanical characteristics of a bar member: modeling based on the moment–curvature relationship derived numerically by dividing the cross section into infinitesimal regions and applying the stress–strain relationship for the material to each infinitesimal region and modeling based on the relationship empirically determined from experiment results. A fiber model in which the stress–strain relationship for the material used is applied to beam elements automatically takes into account the moment–curvature relationship and the axial force–axial strain relationship in the numerical algorithm. Because such a fiber model is capable of allowing for the influence of changes in axial force, it is useful and has been widely used in recent years. When using a fiber model, it is necessary to set element dimensions in view of the items specified in Section 7.2.2.1 and the following items:

1) When based on the stiffness method, element dimensions in the direction of the member axis must be set so that the equilibrium of forces among the elements is maintained.

2) If the cross section of a member is divided, the strain gradient becomes large when the member is acted upon by flexural moment. The accuracy of analysis decreases if the peripheral zone of the cross section is divided. It is therefore necessary to set element dimensions in view of such factors as the shape of the cross section, reinforcement patterns and axial force. In general, it is recommended that the peripheral zone be divided, in view of the location of the center of gravity of the longitudinal reinforcement, so that element width is not greater than the concrete cover. Preferably, it should be ascertained that solutions do not vary even when the number of cross-sectional segments is varied to some extent.

It is also possible to model the moment–curvature relationship derived from cross-sectional analysis and use the model thus obtained as a mechanical model. When dividing a cross section, it is necessary to take into consideration items similar to the considerations in fiber modeling.

The applicability of a mechanical model of a bar member derived empirically from experiment results is in many cases guaranteed only within a limited range of member dimensions or material characteristics. It is therefore necessary to use such a model after carefully considering the scope of application of the mechanical model. If a mechanical model of a bar member is defined in terms of the relationship between flexural moment and the average curvature of the hinge, the average curvature of the hinge must be set at a value corresponding to the determined hinge length, and the determined hinge length and the average curvature need to be used as a pair.

(3) An example of skeleton envelope curves of reinforced concrete linear members is shown in Fig.C7.2.1. This skeleton curve was defined with reference to study results based on past experimental results including results related to flexural failure type members. When no special

investigation is conducted, skeleton envelope curve may be defined based on the following method. In the case of prestressed concrete members, the yielding point of a member may be evaluated as 90% of the maximum load carrying capacity point of a member when no special investigation is conducted. In steel frame reinforced concrete members, skeleton envelope curve may be defined converting steel frame into re-bars, on condition that the ends of steel frame have sufficient development length in concrete.

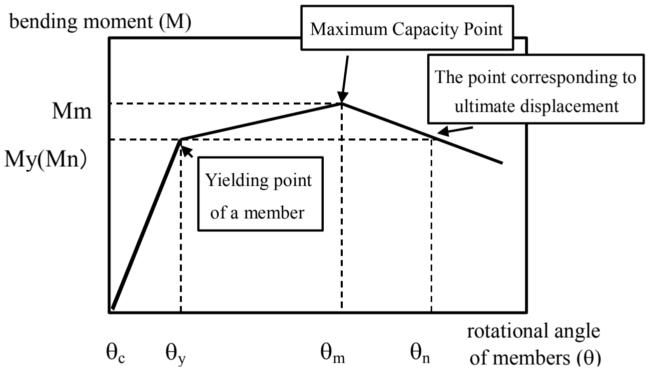


Figure C7.2.1 Example of skeleton envelope curves of linear member model

(i) Yielding Point of Member

Bending moment (M_y): Bending moment at the yielding of tensile re-bars

Rotational angle of member (θ_y) : It may be calculated by Eq.(C7.2.1)

$$\theta_y = \delta_{y0} / L_a \tag{C7.2.1}$$

where, δ_{y0} : displacement of member body at the yielding point of member

L_a : shear span

When the effects of the pullout of longitudinal re-bars from the member joints are large, rotational angle at the end of a member (θ_{y1}) due to the pullout of longitudinal re-bars should be added to rotational angle of member. θ_{y1} may be calculated by Eqs. (C7.2.2) - (C7.2.4).

$$\theta_{y1} = \Delta L_y / (d - x_y) \tag{C7.2.2}$$

$$\Delta L_y = 7.4\alpha \cdot \varepsilon_y(6+3500\varepsilon_y)\phi/(f'_{fcd})^{2/3} \tag{C7.2.3}$$

$$\alpha = 1 + 0.9\, e^{\,0.45(1-Cs/\phi)} \tag{C7.2.4}$$

where, ΔL_y: pullout of longitudinal re-bars from the joint of members at the yielding point of member(mm)

d : effective depth(mm)

x_y : neutral axis at the yielding point of member(mm)

ε_y : yielding strain of tensile re-bar

ϕ : diameter of tensile re-bar(mm)

c_s : space of re-bars(mm)

f'_{fcd} : compressive strength of concrete at the joint of members(N/mm^2).

(ii)Maximum Load Carrying Capacity Point

Bending moment (M_m): The maximum bending moment calculated by using the provision 6.2.1(2) specified in 'Structural Performance Verification'. However, member factor should be taken as 1.0.

Rotational angle of member (θ_m): It may be calculated by Eqs.(C7.2.5) - (C7.2.9).

$$\theta_m = \delta_{m0} / L_a \tag{C7.2.5}$$

$$\delta_{m0} = \delta_{mb} + \delta_{mp} \tag{C7.2.6}$$

$$\delta_{mp}=\theta_{mp}(L_a - L_p/2) \tag{C7.2.7}$$

$$\theta_{mp}=(0.021k_{w0}p_w+0.013)/(0.79p_t+0.153) \tag{C7.2.8}$$

here, $0.021k_{w0}p_w+0.013 \leq 0.04$, $0.79p_t+0.153 \geq 0.78$

$$L_p = 0.5d + 0.05L_a \tag{C7.2.9}$$

where, δ_{m0} :displacement of member body at the maximum load carrying capacity point

δ_{mb}: displacement due to the flexural deformation except the plastic hinge

δ_{mp}: displacement due to the flexural deformation at the plastic hinge

θ_{mp}: rotational angle at the plastic hinge

p_w: lateral reinforcement ratio(%)

p_t: tensile reinforcement ratio(%)

k_{w0}: factor to consider the strength of lateral re-bars, it should be taken as 0.85 for SD295, 1.0 for SD345, 1.15 for SD390

L_p: length of plastic hinge

d: effective depth of section

When the effects of the pullout of longitudinal re-bars from the member joints are large, rotational angle at the end of a member (θ_{m1}) due to the pullout of longitudinal re-bars should be added to rotational angle of member. θ_{m1} added to Eq.(C7.2.5) may be calculated by Eq.(C7.2.10).

$$\theta_{m1}=(130\theta_{mp} - 0.47)\ \theta_{y1} \tag{C7.2.10}$$

here, $1.0 \leq 130\theta_{mp} - 0.47 \leq 4.7$

(iii) Softening region after the maximum load-carrying capacity

In the verification of Seismic Performance 2 and 3, the mechanical behaviors of members in softening range beyond the maximum capacity point should be modeled. At present, it is difficult to accurately evaluate the softening region after the maximum load-carrying capacity point is reached. For this reason, the method of reducing skeleton curve values at a uniform ratio (usually 0.1) is permitted. It is recommended that in large-axial-force cases or in cases where shear capacity is close to flexural capacity, the value of η be made smaller.

$$\Delta\theta = \eta\left(\frac{M_n - M_m}{M_m}\right) \tag{C7.2.11}$$

where, $\Delta\theta$: increase in rotation angle from θ_m

η: Usually, 0.1 may be used.

M_m: maximum bending moment

M_n: Usually, M_y may be used.

<u>(4)</u> In general, shear deformation of bar members and out-of-plane shear deformation of planar members are small enough to be ignored. If shear deformation cannot be ignored, recommended practice is to use linear elements capable of allowing for shear deformation such as Timoshenko beams or take a finite element modeling approach. It is desirable that the out-of-plane deformation of a planar member be modeled by use of finite elements because the influence of shear deformation cannot be ignored. When taking a fiber modeling approach, it is necessary to define the shear force–shear deformation relationship appropriately and use linear elements capable of allowing for shear deformation.

<u>(5)</u> In order to model the response of structures accurately during an earthquake, the mechanical models of members should consider the load hysteresis properties. In the case of reinforced concrete members, Clough model or Takeda model which is classified as stiffness degrading type are used. In the case of prestressed concrete members, Takeda model that considers the effects of the amount of PC steels and re-bars to the unloading stiffness is used. In the case of steel frame reinforced concrete members, modified Takeda model is used. When stiffness decreasing type models are used for reinforced concrete members, unloading stiffness may be determined with Eq.(C7.2.12).

$$k_r = k\left|\frac{\theta_{\max}}{\theta_y}\right|^{-\beta} \tag{C7.2.12}$$

where, k_r : unloading stiffness

k : yielding stiffness

$\theta_{\max}$: response rotational angle of member

θ_y : rotational angle of member at the yielding of a member

β : stiffness decreasing rate（in general, 0.5)

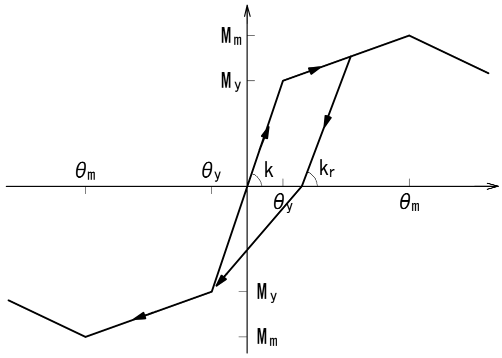


Fig. C7.2.2 Hysteresis model of bar member

7.3 Structural Analysis

7.3.1 General

(1) In structural analysis, a structural analysis method that meets the load and structural modeling needs and makes verification indices available shall be used. If verification indices cannot be obtained directly from structural analysis, a structural analysis method that enables conversion to verification indices by an appropriate method may be used.

(2) An analysis of a structural member shall give consideration to the influence of nonlinearity according to response. A structural member may be regarded as a linear member if it is evident that the nonlinearity of the member does not affect verification indices such as sectional forces or that conservative and rational evaluation results are obtained.

[Commentary] (1) Structural analysis must be made by using an analysis method selected according to the load and structural modeling needs. In order to verify the performance of a structure, it is desirable that verification index values be obtained directly from structural analysis, and it is necessary, wherever possible, to select a structural analysis method that makes it possible to obtain verification index values directly. If, however, verification index response values cannot be obtained directly from structural analysis, it is necessary to convert the index response values obtained from the analysis to design response values of verification indices by an appropriate method.

(2) 1) Because the stiffness of a concrete member varies depending on such factors as cracking, it is necessary to appropriately evaluate the nonlinearity of the member material. In general, the mechanical models of materials indicated in Chapter 5 may be used for material nonlinearity modeling. The nonlinearity of a structure or a member includes geometric nonlinearity besides material nonlinearity. If geometric nonlinearity cannot be ignored, an analysis theory that makes it possible to allow for the influence of geometric nonlinearity must be applied.

If the influence of the nonlinearity of a member on response values that can be used as verification indices such as sectional forces can be ignored, the stiffness of the member may be deemed to be elastic.

2) Analysis methods other than linear analysis include exact nonlinear analysis that takes into consideration material and geometric nonlinearity, equivalent linear analysis that approximately

evaluates the influence of nonlinearity by conducting a linear analysis using a member stiffness reduced from elastic stiffness, and plastic analysis applied to beams, slabs, etc.

The moment redistribution method applied to statically indeterminate structures such as continuous beams, rigid frames and continuous slabs is a practical sectional force calculation method based on the plastic analysis method. For slabs, there are yield line theory, which gives an upper bound, lower bound theory, which gives a lower bound, and the strip method. When applying a method other than these linear analysis methods, it is necessary to select an appropriate value of the structural analysis factor γ_a.

3) When conducing an analysis of a member that takes into consideration nonlinear characteristics including the plastic range of the member as when verifying the failure or collapse of a structure or calculating design response values to be used for verification with respect to the influence of an earthquake, it is necessary either to determine a nonlinear model or restoring force model of the member by using a nonlinear hysteresis model as a material model or to use a nonlinear model or restoring force model based on previous study results.

When safety from failure is to be verified by using a discretization method such as finite element analysis based on nonlinear theory, the safety of a structure or member can be verified by using verification indices other than sectional forces or sectional capacity. In such cases, it is recommended that verification be made by setting indices suitable for the type of failure and the limit state and comparing the obtained design response values and design limit values. For example, in the verification of bending-dominated failure or shear failure involving an extensive damage region (e.g., shell structure), indices based on such deformation as element plastic strain or element rotation angle can be used. Indices that can be used for highly localized failure modes include indices related to shear force calculated from element stresses, the progress of cracking or the state of material damage. In all cases, the structural analysis factor γ_a needs to be determined through the experimental verification of the scope of application and accuracy.

7.3.2 Structural analysis related to safety verification

7.3.2.1 Structural analysis for verification of cross-sectional failure

(1) As a general rule, the structural analysis for the verification of the cross-sectional failure of a member shall take into consideration the influence of the nonlinearity of the member. If the influence of the nonlinearity of a member on the design response values such as sectional forces can be ignored, design response values may be calculated by assuming the linearity of the member. In such cases, the structural analysis factor γ_a of 1.0 shall be used.

(2) As a general rule, when the nonlinearity of a member is considered, the effect of material nonlinearity shall be considered, and the effect of geometric nonlinearity shall also be taken into consideration on an as-needed basis.

(3) If sectional forces are calculated through linear analysis, the effect of nonlinearity may be evaluated by a simple method on condition that detailed structural requirements are met.

[Commentary] (1) In general, deformation properties of a member in the limit state of cross-sectional failure are nonlinear. When calculating deformation in the limit state of cross-sectional failure, therefore, it is essential that nonlinearity be taken into consideration. It is also rational to use nonlinear analysis when calculating sectional forces.

If, however, the ultimate strain of concrete in the modeled stress–strain curve for concrete shown in Chapter 5 is used as the design limit value for the limit state of cross-sectional failure for a flexural failure type member, it is good practice to apply, for example, the equivalent linear analysis method, which approximates the effect of nonlinearity by using stiffness equivalent to the stiffness at the stress level under consideration, because the influence of the nonlinearity of the member on response values is small if an appropriate value of flexural stiffness of the member is used. In such cases, the structural analysis factor γ_a of 1.0 may be used to calculate design response values in the limit state of cross-sectional failure.

If the design limit value in the limit state of cross-sectional failure is greater than the ultimate strain mentioned above, appropriate verification indices such as compressive strain in concrete, curvature and the joint translation angle must be set and the design limit value verified by applying a nonlinear analysis method such as the incremental analysis method. In this case, the nonlinearity of the member concerned must be in accordance with Section 7.3.4.

If, however, sectional forces are used as verification indices for a statically determinate structure such as a girder or a cantilever beam, the stiffness of the member may be assumed, for convenience, to be elastic because changes in sectional forces due to the stiffness of a member are usually small.

(2) In the limit state of cross-sectional failure, as a general rule, the effect of material nonlinearity is to be taken into consideration. Since, however, it may not be possible to ignore geometric nonlinearity depending on structural configuration as the amount of member deformation increases, it is also required that the effect of geometric nonlinearity be taken into consideration on an as-needed basis.

(3) By specifying structural details by using the linear analysis method, the method of allowing for the nonlinearity of a member or structure easily is permitted. Simple methods include those listed below. In this case, the structural analysis factor γ_a of 1.0 may be used.

1) If linear analysis is used, plastic rotation capacity needs to be secured because moment redistribution may occur in reality. As a general rule, the steel ratio in all cross sections must be 75% or less of the balanced steel ratio.

2) When designing for flexural moment at supports or joints in continuous beams and slabs, frames, etc. moment redistribution may be considered according to values obtained using linear analyses. Flexural moment to be thus redistributed shall be within 15% of the values obtained by linear analysis, and the flexural moment at any cross section shall not be smaller than 70% of the values before redistribution. In such cases, it shall be ensured, or checked, that the reinforcement at any cross section does not exceed 50% of the balanced reinforcement ratio.

3) Member forces associated with imposed deformations due to temperature changes, drying shrinkage and creep in ordinary conditions may be neglected. In such cases, the ratios of reinforcement at any cross section shall not exceed 50% of the balanced strain reinforcement ratios. If, however, the structural system during construction differs from the completed structural system, changes in member forces due to creep shall be taken into account.

If sectional forces are calculated through nonlinear analysis, sectional forces reflecting the influence of those factors can be obtained because such influence has been allowed for in structural analysis. If, however, sectional forces are calculated through nonlinear analysis, the influence of those factors is not taken into account in structural analysis. When calculating design response values for the verification of cross-sectional failure, therefore, response values due to those types of forced deformation may be ignored on condition that a certain level of plastic rotation capacity of the member under consideration is secured. This Specification requires, therefore, that the steel ratio be 50% or less of the balanced steel ratio in all cross sections. This requirement does not need to be met if verification is made through nonlinear analysis.

If, however, the structural system under construction differs from the completed structural system for reasons associated with the construction method used, changes in member forces, particularly due to creep, may be unignorably large. In such cases, appropriate consideration to changes in member forces due to creep is required. If the influence of temperature changes, etc., that do not result from weather conditions on design response values related to failure verification cannot be ignored, it is recommended that design response values be determined in view of the decrease in stiffness of the member due to cracking, etc., on the basis of experimental results or a well-grounded theory.

7.3.2.2 Structural analysis for verification of fatigue failure

(1) As a general rule, the structural analysis for the verification of the fatigue failure of a member shall take into consideration the influence of the nonlinearity of the member according to structural characteristics. If, however, the influence of the nonlinearity of the member on design response values can be ignored, design response values may be calculated by assuming the linearity of the member. In such cases the structural analysis factor γ_a of 1.0 shall be used.

(2) As a general rule, when the nonlinearity of a member is considered, the effect of stiffness reduction due to cracking shall be taken into consideration. As a general rule, the influence of normal temperature changes, shrinkage, creep, etc., shall be taken into account when calculating response values.

[Commentary] (1) Design response values used for fatigue failure verification needs to be calculated taking into consideration such factors as changes in the stiffness of the structural member under consideration due to loads and other influences during the design service life.

Under normal service conditions, a structural member usually shows nonlinearity. It is therefore required that calculation be made taking into consideration the influence of stiffness reduction due to cracking of the member. Although there are various analysis methods that take into account the influence of member stiffness reduction, it is good practice to apply an equivalent linear analysis method that uses stiffness equivalent to that at a particular stress level. In such cases, the structural analysis factor γ_a of 1.0 may be used.

(2) In the verification of fatigue failure, the state of stress under normal service conditions is considered. Sectional forces, therefore, due to such factors as normal temperature changes, shrinkage and creep cannot be ignored. In the verification of fatigue failure, it is also necessary to take into account the state of stress under permanent loads. This is why it is required, as a general rule, that the influence of normal temperature changes, shrinkage and creep be taken into consideration.

7.3.2.3 Structural analysis for verification of structural safety

(1) As a general rule, the structural analysis for the verification of the stability of a structure shall take into consideration the stiffness of the ground and the foundation.

(2) As a general rule, in the structural analysis for the verification of the displacement, deformation and mechanism of a structure, an analysis method takes into consideration the influence of the nonlinearity of the structural members and the ground. As a general rule, material nonlinearity and geometric nonlinearity shall be taken into consideration as part of the nonlinearity of structural members.

[Commentary] (1) The verification of the stability of a structure is to be verified by conducting a structural analysis using a model capable of estimating the behavior of the entire structure including the structural elements and the ground. For the verification of the stability of a structure under the influence of the deformation of the foundation structure, it is necessary to conduct a structural analysis using a model capable of allowing for structural deformation caused by the ground.

(2) When verifying the limit state of the displacement, deformation and mechanism of a structure, it is necessary to conduct a structural analysis, taking into consideration the post-peak range of the structure or its members. In this case, since reinforced concrete members are subject to buckling of the longitudinal reinforcement, spalling of cover concrete, etc., it is necessary to conduct a structural analysis by using a material model or member model capable of allowing for the influence of these factors. This structural analysis may be conducted by using the method described in Section 7.3.4.

In the limit state of displacement, deformation or the mechanism of a structure, usually its members undergo large displacement and deformation. As a general rule, therefore, it is required that the influence of geometric nonlinearity be taken into account in addition to material nonlinearity.

7.3.3 Structural analysis for verification of serviceability

(1) As a general rule, appearance verification using crack width or material stress shall take into consideration the influence of stiffness reduction due to cracking. If, however, the influence of the nonlinearity of the member under consideration can be ignored, design response values may be calculated by assuming the nonlinearity of the member. In such cases, the structural analysis factor γ_a of shall be used. As a general rule, response values shall be calculated taking into account the influence of such factors as normal temperature changes, shrinkage and creep.

(2) If the comfortability of a structure is to be verified by using displacement or deformation, as a general rule, the influence of stiffness reduction due to cracking and shrinkage and creep occurring during the design service life shall be taken into consideration.

(3) Structural analysis for the verification of vibration shall be conducted by a method appropriate for the type of structure concerned.

(4) As a general rule, structural analysis used in a study on stress limits shall take into consideration the influence of stiffness reduction due to cracking, shrinkage and creep.

[Commentary] (1) For members that are expected to have cracks at the serviceability limit state, the reduction in the stiffness of the cross-section due to cracking should be accounted for, when calculating member forces caused by changes in ambient temperature and shrinkage. This ensures rationality because the more realistic structural behavior is considered in design and over-reinforcement of the section is avoided. For calculating decreases in the stiffness of a member due to cracking, it is good practice to determine stiffness experimentally or according to well-grounded theoretical analysis. As an approximation approach, stiffness may be determined in accordance with Section 7.5.4 in consideration of the influence of shrinkage and creep.

There are many types of concrete cracks. Cracks considered in this section are mechanically caused cracks, particularly cracks caused by bending moment, axial force, shear force or torsional moment.

Cracking verification is to be made under normal service conditions. If there are differences between the structural system being constructed and the completed structural system, it is necessary to take their influence into consideration when calculating sectional forces. In general, in the state of stress under service conditions, the effect of sectional forces under the influence of such factors as normal temperature changes, shrinkage and creep cannot be ignored. As a general rule, therefore, it is required that the influence of those factors be taken into consideration. To allow for the influence of temperature change, shrinkage and creep, the values indicated in Chapter 5 may be used.

(2) During normal use, loads vary widely. As a result, crack patterns vary among members depending on the loads to be verified, and displacement and deformation vary with stiffness. When evaluating displacement and deformation as verification indices, therefore, it is necessary to take into consideration the influence of stiffness reduction due to cracking according to the loads concerned.

In general, structural members are classified broadly into those that are subject to cracking and those that are not subject to cracking. For members that are not subject to flexural cracking, elastic theory is applied assuming a fully effective cross section. For members that are subject to flexural cracking, an analysis method that appropriately allows for stiffness reduction due to cracking is to be used. The influence of stiffness reduction due to cracking may be allowed for by using the method described in Section 7.4.5. Since, however, that method calculates the stiffness of a member under virgin loading, if the limit state to be verified is one under cyclic loading during normal use, it may be necessary to use the stiffness of a member under repeated loading.

When considering displacement and deformation, it is necessary to take into consideration the influence of the shrinkage and creep of concrete besides cracking.

(3) The structural analysis method used for vibration verification must be one appropriate for the type and characteristics of the structure under consideration. Vibration-related studies have been underway from the viewpoints of the determination of basic characteristics, prediction methods, etc., but generation and propagation mechanism are very complicated. It is therefore necessary to use an analytical method in conjunction with available measurement results obtained under similar conditions.

(4) Studies related to stress limits involve the verification that response values meet the limit requirements for permanent in-service loads and the maximum loads of variable loads. Because structural members are normally subject to cracking, it is required as a general rule that the influence of stiffness reduction due to cracking be taken into consideration. It is also required that the influence of shrinkage and creep be taken into account.

7.3.4 Structural analysis for earthquake resistance evaluation

7.3.4.1 General

(1) As a general rule, structural analysis used for the verification of earthquake resistance shall be conducted by using a time history response analysis method that allows for the influence of the nonlinearity of the member under consideration and the ground. If a structural analysis method with proven accuracy of analysis is used, the structural analysis factor of 1.0 may be used.

(2) In general, when the nonlinearity of a member is considered, the influence of material nonlinearity shall be considered. The influence of geometric nonlinearity shall also be taken into consideration on an as-needed basis.

(3) If the analysis region for structural analysis to evaluate earthquake resistance includes ground, a structure–soil coupled analysis shall be conducted, and modeling shall be performed as stipulated in Section 7.3.4.3. However, if dynamic interaction between the structure and the ground can be neglected or it can be appropriately modeled, the structure may be analyzed independently in accordance with 7.2.4.2.

(4) The integrating time interval shall be determined considering both the accuracy of the response values and the stability of response analysis, depending upon the method adopted for time integration.

(5) Viscous damping of structures should, in principle, be neglected. However, it may be accounted for when carrying out the verification of the structures in elastic region.

[Commentary] (1) Recently, numerical simulation technique has been developed notably, and reliability of analytical results has also been improved sufficiently through comparisons with several kinds of experimental results. Therefore, it is in principal in this specification that time history response analysis is performed to estimate response values. When it is obvious that the response of a structure and members are in elastic region, other reliable methods may be used for verification.

(2) The geometrical nonlinearity generally may be taken into account by the effect of additional moment due to response displacement. When large plastic deformation occurs in structures and members, or high axial force is acted to slender columns, load carrying capacity of a member decreases due to large additional moment and it is also strongly related to reach the mechanism of load resistant. Therefore, this effect should be considered. If damage of a member due to shear failure is avoided, collapse of the whole structure system is reached by the effect of geometrical nonlinearity. However, the effect needn’t to be considered in case that large deformation where the effect of geometrical nonlinearity occurs is not allowed.

(3) Since the response of a structure during an earthquake is strongly affected by neighboring ground and others, the whole structural system including foundation or neighboring ground should be analyzed. To do so, a coupled analysis taking the interaction between structures and soil into account directly should be used. Table C 7.3.1 shows a general analytical method. In modeling of ground, the influence of distant ground and others should be sufficiently examined. An example of finite element model of a structure is shown in Fig. C 7.3.1.

According to types or characteristics of structures and ground, there are some cases which dynamic interaction between structures and ground can be neglected or modeled appropriately. In these cases, the responses of the structures and the ground may be analyzed independently according to the provision in 7.3.4.2, because the response can be estimated with sufficient reliability without using coupled analysis.

Table C73.3.1 Method for a coupled analysis of structure and ground

Structure Type	Ground structure, Underground Structure
Analytical Method	Time History Response Analysis
Analytical Model of	Finite Element Model or Beam Element Model
Analytical Model of	Finite Element Model
Input Value	Time History Acceleration Wave Form
Input Place	Base Layer

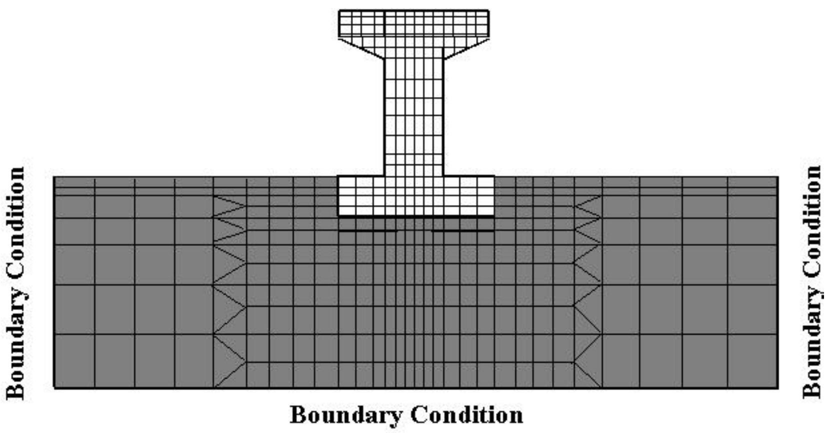


Fig. C 7.3.1 Example of modeling for structure and ground in a coupled analysis

(4) We have explicit and implicit integration methods of acceleration in time history response analyses. Moreover, we have central difference calculus, Runge-Kutta and others in the explicit method, and Newmark's β method, Wilson's θ method and others in the implicit method. Among them, Newmark's β (β=0.25) method or Wilson's θ method are frequently used because of the stability of the solutions. Convergent calculation is necessary in these methods in order to satisfy the equation of motion in each time step, and if the convergent calculation is skipped, it should be noted that the analytical accuracy will be decreased. On the other hand, central difference calculus is representative in the explicit method. In the central difference calculus, convergent calculation used in the implicit method is not necessary and is quite accurate. However, the stability of the solution is poor and definition of fine integration time interval for natural period of the highest order in the vibration system is required.

(5) According to dynamic loading tests changing response velocity of member, it has been confirmed for linear members and planar members that, in general, the characteristics of restoring force including inelastic region are hardly affected by the displacement velocity. This means that the effect of viscous damping in proportion to the response velocity is negligible in inelastic region, and

it can explain that the effect of hysteresis damping is dominant. In order to consider the hysteresis damping in time history response analysis, path-dependent mechanical models of material and member are adopted in this specification. Consideration of excessive viscous damping leads to dangerous evaluation due to estimating smaller structural response, and the physical significance of viscous damping constant is ambiguous in nonlinear region. Therefore, it is specified in principle not to consider viscous damping constant. However, viscous damping constant may be determined for a structure in verification of the Seismic Performance 1 because nonlinear response is not considered. Since there may be cases where viscous damping is considered for numerical calculation stability, in certain cases such as when the solution diverges, viscous damping may be considered after ascertaining that the influence on the structure under consideration is sufficiently small.

7.3.4.2 Method for analyzing the structure and the ground independently

(1) In calculating the dynamic response of the structure on the ground, the input wave should be taken to act at the depth, which is obtained from the response analysis of only the ground, and dynamic computations should be carried out in time domain.

(2) The response of the ground should, in principle, be carried out using dynamic analysis by the model following 7.3.4.4.

(3) The response of the foundation and the underground structure may be calculated using the response displacement method. The value of the response displacement may be taken as the displacement at the time when the relative displacement of the ground in the position of the structures reaches the peak.

[Commentary] Table C7.3.2 lists the methods for analyzing the structure and the ground independently.

(1) When analyzing a structure on the ground independent of the ground, earthquake ground motion at the base of the structure should be estimated by dynamic analysis of the subsurface ground at the site, and then dynamic response analysis of the structure should be performed using the estimated earthquake ground motion as shown in Fig. C 7.3.2.

(2) Analysis of seismic response of the ground, in general, is applied to the time history response analysis method and to the response analysis method in frequency domain. In the time history response analysis, the nonlinearity of the soil can be modeled as a stress-strain relationship. On the other hand, in the response analysis in the frequency domain, it is pointed out that the conformity with the actual action of the ground reduces in the large strain area where the strain level of the soil exceeds 10^{-3}, because the method approximates the nonlinearity of the soil by an equivalent linearized technique. Therefore, a proper analytical method corresponding to the strain level of the soil should be chosen.

Table C7.3.2 Methods to analyze the structure and the ground independently

Structural Type		Structure on the ground	Underground structure
Analysis of Structure	Analytical method	Time History Response Analysis	Response displacement method
	Analytical model of structure	Finite Element Model or Beam Element Model	
	Interaction model	Spring model	
	Input value	Time History Acceleration Wave Form calculated from analysis of ground	Displacement of ground calculated from analysis of ground
	Input place	Base part of structure	Side part of structure
Analysis of Ground	Analytical method	Time history response analysis or response analysis in frequency domain	
	Analytical model of ground	One-dimensional continuous model or finite element model divided in different layers	
	Input value	Time History Acceleration Wave Form	
	Input place	Engineering Base layer	

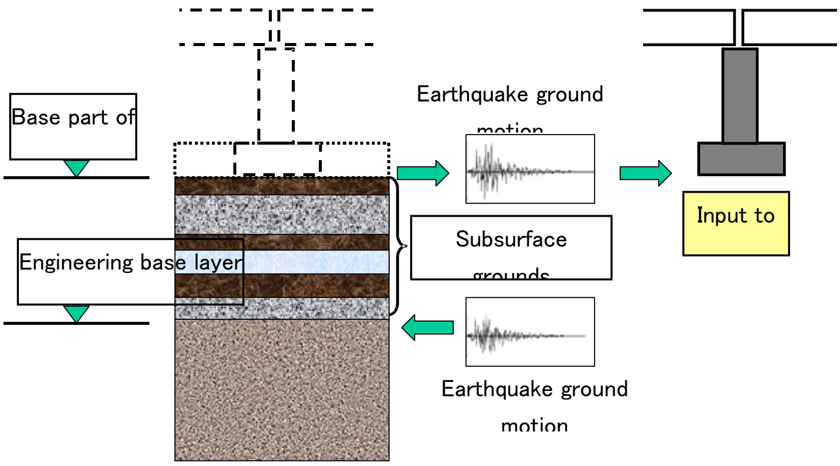


Fig. C7.3.2 Method to analyze a structure independent of the ground

(5) Relative vibration against the ground is hard to occur on usual underground structure constructed in the subsurface ground. The structural response follows the displacement and deformation of the surrounding ground during an earthquake, and the influence of the inertia force caused by the mass of the structure is small. When the oscillation mode of the ground coincides with that of the structure, the forced displacement, which is obtained as the deformation of the ground during the earthquake (response displacement), may be acted statically on the structure.

Behavior of an underground structure during an earthquake is affected not by the scale of the absolute displacement of the ground at the position of the structure, but by the rate of vertical change of the horizontal displacement. Therefore, the displacement used for the response displacement method is determined as the displacement of the ground at the time when the relative displacement of the ground between the top and the bottom ends of the structure is the maximum. As for long structures such as piles, the time when the relative displacement of the ground at each depth is the maximum may be different due to the stratum condition. In such a case, some ground displacements that give a great influence in each section of the pile should be chosen from the time history displacement.

7.3.4.3 Ground model for coupled analysis

(1) In coupled analysis considering the superstructure, the foundation and the ground, the ground should be modeled using finite element over a sufficiently large region. When ground has any irregularity, its effects shall also be considered.

(2) If the influence of an earthquake is to be taken into consideration, a model that covers a sufficiently large region including the ground and allows the use of boundary conditions so that the propagation of ground motion can be allowed for shall be used.

(3) When separation or sliding between the ground and the structure affect the response of the structure, joint elements that can represent the effects of the separation and the sliding may be used between the structure and the ground.

(4) Constitutive models used to model the behavior of the ground shall be able to take the characteristics of plastic behavior of the ground as well as its liquefaction into account.

[Commentary] (1) When a coupled analysis is performed, the ground shall be modeled as large region as possible so that the range in which the influence of the dynamic interaction of the structure and the ground may become small enough. When the ground has remarkable irregularity as in the case that the engineering base layer and the ground surface are inclined, wave propagation becomes complicated. This sometimes leads to local amplification of earthquake ground motion or elongation of the duration of the earthquake ground motion due to the occurrence of a secondary wave such as surface wave. Therefore, if the ground has remarkable irregularity, the effect should be taken into account. When the ground is modeled with finite elements, the layer thickness may be about 1/4 –1/6 of the minimum wavelength (the period × the shear wave velocity of the considered stratum) in the considered frequency band for the response analysis of the structure and the ground, because the wave motion beyond a certain frequency level is eliminated corresponding to the layer thickness of the element.

(2) Since earthquake ground motion shown in 2.2 is defined on the engineering base layer and the influence of the reflected wave of the subsurface ground is not included, the viscous boundary, which absorbs a reflected wave may be arranged on the bottom boundary of the ground model. When the bottom boundary of the ground is modeled as a fixed boundary, input earthquake ground motion should be applied considering the influence of the reflected wave. The side of the ground should be modeled with enough distance and extension, and the boundary condition may be modeled to absorb the reflected waves.

(3) When a strong earthquake ground motion such as Level 2 earthquake motion acts, a separation and a slide phenomenon may occur between the structure and the ground due to the

difference in relative movement between the structure and the ground. This may possibly affect the whole response of the structure. Joint elements between the structure and the ground can be used to model such phenomenon in finite element model. In such a case, the behavior of the osculating plane between the ground and the structure should be investigated and its results should be reflected on the characteristics of the joint element. Since separation occurs when a normal stress at the osculating plane between the structure and the ground exceeds a tensile stress, relations between the normal stress and the normal strain should be evaluated. Since a slide occurs when a shear stress at the osculating plane exceeds the shear strength, relations between the shear stress and the shear strain should be evaluated.

(4) Dynamic shear stress-strain relationship of soil should be evaluated properly corresponding to the presumed scale of shear strain amplitude of the ground during an earthquake. Generally, dynamic shear stress-strain relationship of soil is divided into hysteresis curves and their skeleton curve as shown in Fig.C7.3.3. This skeleton curve shows a remarkable nonlinearity when the shear strain amplitude becomes large. In order to obtain a hysteresis model representing such characteristics, in many cases, the skeleton curve is expressed by a function such as a hyperbolic one and Masing rule is applied to the hysteresis curve. While constructing a hysteresis model of the soil, the strain dependency of elastic shear coefficient and damping factor of the soil may be evaluated by in-situ tests such as PS logging and laboratory tests such as deformation characteristics test using a sample of the concerned ground.

When the ground liquefies, the stiffness of the soil is affected by the magnitude of excessive pore water pressure. Therefore, the reduction in strength and stiffness of the soil caused by the rise of the excessive pore water pressure should be considered in addition to shear stress-strain relationship. An effective stress analysis is conducted for taking such a phenomenon into consideration. Not only a dynamic deformation characteristics test but also a liquefaction strength test is necessary to perform an effective stress analysis.

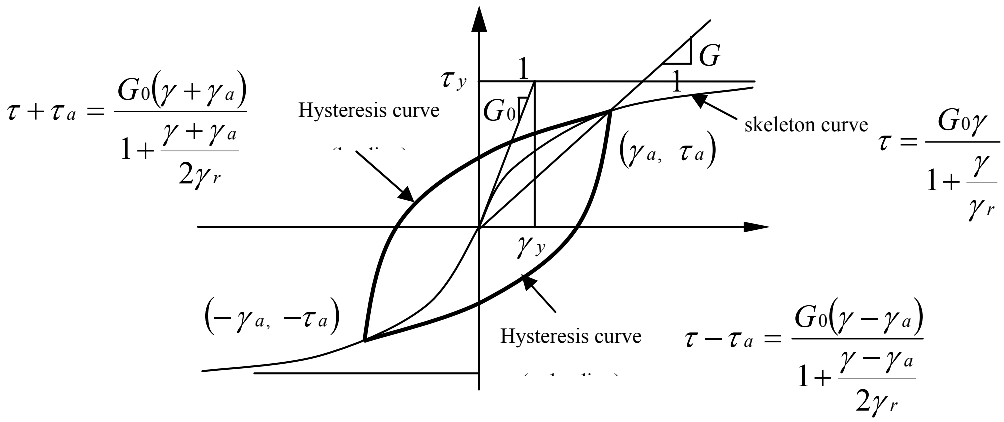


Fig. C7.3.3 Example of the dynamic shear stress-strain curve of the soil

7.3.4.4 Ground model to analyze a structure independent of the ground

(1) The boundary between the structures and the ground may be modeled by nonlinear springs to account for interaction between the foundation of structures and the ground. Generally, these springs should be modeled in vertical, horizontal and rotating directions.

(2) The interface between the ground and the foundation or the underground structure may be modeled using nonlinear springs, taking into consideration the characteristics of the surrounding ground.

(3) The boundary between the ground including the foundation and the structure on the ground may be modeled using nonlinear springs that support the structure on the ground.

(4) An analysis model of the ground to be used to take earthquakes into consideration should be a one-dimensional layered continuum model or a finite element model.

[Commentary] (1), (2) and (3) In general, the nonlinear support springs that support superstructures are vertical, horizontal and rotation springs. They should be arranged appropriately according to the ground condition, the form of the foundation and the bearing capacity characteristics.

As to the nonlinear support springs that support underground structures, a normal directional spring at the interface between the structure and the ground as well as a tangential one should be considered. In this case, the characteristics of the springs should be determined appropriately corresponding to the ground condition, bearing capacity of the ground and the form of the structure. For design of the support springs, the load-displacement relationship considering the nonlinearity of the ground should be applied.

When determining the restoring force characteristics of a support spring, it is necessary to use the load–displacement relationship reflecting the nonlinearity of the ground or of the ground and the structure as a skeleton curve and take appropriate hysteresis characteristics into consideration.

In such cases, support springs may be defined to allow for the influence of dissipation damping or internal damping appropriate for the foundation structure. The purpose of the modeling described here is to calculate the response values of a surface structure. When calculating the response values of a structural foundation, therefore, it is necessary to use a method that makes it possible to allow for the influence of the vibration mode of the foundation such as modeling the ground and the structural foundation with springs and masses.

(4) As dynamic analysis methods applicable to the ground, time history response analysis in the time domain and response analysis in the frequency domain are generally used. Time history response analysis, which keeps track of the nonlinear stress–strain relationship of soil according to time history, takes considerable time for response value calculation. Response analysis in the frequency domain approximates the nonlinear stress–strain relationship of soil by the equivalent linearization method. It has been pointed out, therefore, that the degree of agreement with ground behavior falls in a large strain range. When conducting a dynamic analysis, therefore, it is necessary to select an appropriate analysis method according to the soil strain level.

The equivalent linearization method is a method of response analysis in the frequency domain. This method is often used in cases where the shear strain amplitude of the ground during an earthquake is relatively small (about 10^{-3} or smaller). An equivalent linear model is defined as a viscoelastic body reflecting the stiffness and viscosity of soil. Equivalent shear moduli and viscous damping ratios during ground motion are determined according to the amplitude of shear strain (effective strain) occurring during an earthquake. In the equivalent linearization method, linear analysis using an equivalent linear model is repeated until the shear strain amplitude converges. The method, therefore, is a practical approach because stable solutions can be obtained and computation time is short.

When determining time history acceleration waveforms at a specified depth, it is common practice to divide the ground around the structure of interest above the engineering bedrock surface into layers or finite elements. If the ground around the structure consists of horizontal layers, response analysis can be simplified by using a one-dimensional ground model.

7.4 Calculation of Design Response Values

7.4.1 General

(1) Design response values shall be calculated by converting the response values obtained as described in Section 7.3 into verification indices by an appropriate method.

(2) Design response values of materials, design crack width and design response values such as the displacement and deformation of structural members shall be calculated taking structural characteristics into account. In general, calculation shall be made by the method described in this section by using the design sectional forces, etc., described in Section 7.3.

[Commentary] (1) There are two methods of structural modeling: the method of using fiber-model-based sectional forces for calculation and the method of calculating stress, strain, etc., as response values by the finite element method. This section describes a method for calculating design response values by converting sectional forces to verification indices such as material stress. If stress or strain is used as verification indices or stress or strain response values are converted to verification indices in the finite element method, it is necessary to conduct a careful study because at present there are various restrictions.

7.4.2 Calculation of sectional forces

7.4.2.1 Calculation of sectional forces in bar members by the finite element method

(1) Computation of axial forces and flexural moments in the member shall be calculated by the integration of the stress distribution obtained from the strain distribution in the cross section, on the basis of following assumptions:

i) Strain distribution across a cross section is linear

ii) Hysteresis is included in the stress-strain relationships of concrete and re-bar. In general, the relationships used in the structural analysis may be used.

(2) Shear forces may be computed using the equilibrium condition for distribution of flexural moment obtained in (1).

[Commentary] (1) and (2) The method of calculating sectional forces from the results of structural analysis that uses the constitutive equation for the stress–strain relationship is the same as the assumption used to calculate the flexural strength of bar members. The validity of Navier's hypothesis decreases in a large deformation range, but because calculated values are conservative with respect to the response values of steel strain and the load-carrying mechanism along the axis of the member, the application of Navier's hypothesis is allowed even in a large deformation range.

In general, compared with the flexural deformation component, the deformation component in the out-of-plane shear direction of a bar member is small. This component, therefore, may be ignored. If, however, the length of the member is small relative to the cross sectional dimension of the member or if shear cracks are widely distributed in the member, it is recommended that either a model capable of allowing for shear deformation be used or the shear deformation component be added to the response displacement. In such cases, deformation may be determined according to Section 7.4.5.

7.4.2.2 Calculation of sectional forces in the case where a fiber-based model Is used

If a fiber-based model of a structural member is used, sectional forces obtained from structural analysis may be used as calculated sectional forces in the member.

[Commentary] If an analysis method that expresses nonlinearity with the relationship between bending moment and the rotation angle, etc., is used, sectional forces obtained as a result of structural analysis may be used as are as calculated sectional forces in the member. If a fiber model is used, sectional forces derived by replacement from element stresses may be used as in the case where the finite element method is used.

7.4.3 Calculation of material stress

Design stresses in materials under normal service conditions shall be calculated as described in Items (1) to (3):

(1) Design stresses in materials due to bending moment or due to bending moment and axial force shall be calculated on the basis of the assumptions described in (i) to (iv):

(i) Fiber strain is proportional to the distance from the neutral axis of the member cross section.

(ii) In general, concrete and steel are assumed to be elastic.

(iii)Tensile stress in concrete is negligible.

(iv)Elastic moduli of concrete and reinforcement may be obtained in accordance with 5.2.5 and 5.3.4.

(2) Design stress in torsion reinforcement due to torsion shall be calculated from Eq. 7.4.3. Design response values in shear reinforcement due to shear force shall be calculated from Eq. 7.4.1 and Eq. 7.4.2.

$$\sigma_{wrd} = \frac{(V_{pd} + V_{rd} - k_r \cdot V_{cd})s}{Aw \cdot z \cdot (\sin\theta + \cos\theta)} \cdot \frac{V_{rd}}{V_{pd} + V_{rd} + V_{cd}} \quad (7.4.1)$$

$$\sigma_{wpd} = \frac{(V_{pd} + V_{rd} - k_r \cdot V_{cd})s}{A_w \cdot z \cdot (\sin\theta + \cos\theta)} \cdot \frac{V_{pd} + V_{cd}}{V_{pd} + V_{rd} + V_{cd}} \quad (7.4.2)$$

where, σ_{wrd} **: design variable stress in shear reinforcement**

σ_{wpd} **: design stress in shear reinforcement due to permanent load**

V_{pd} **: design shear force due to permanent load**

V_{rd} **: design shear force due to variable load**

V_{cd} **: design shear force acting on a bar member without shear reinforcement; as per Section 9.2.2.2. Where, parameter γ_b may be taken as 1.0 and 1.3 in the verifications for serviceability and fatigue**

failure, respectively.

k_r : **coefficient to take into account the effect of variable loads. It may, in general, be taken as 0.5. However, it may be taken as 1.0 for members without significant effect of fatigue loads**

s : **spacing of shear reinforcement**

A_w : **total cross-sectional area of shear reinforcement in section s**

θ : **angle between shear reinforcement and member axis**

z : **distance between the resultant of compressive forces and the centroid of tension reinforcement. It may be taken as d/1.15**

d : **effective depth**

(3) Design stress in torsion reinforcement due to torsion shall be calculated from Eq. 7.4.3.

$$\sigma_w = \frac{M_{tpd} - 0.7M_{t1}}{M_{t2} - 0.7M_{t1}} \cdot f_{wd} \qquad (7.4.3)$$

where, σ_{wpd} : **design stress in lateral torsion reinforcement due to permanent load**

M_{tpd} : **design torsional moment under permanent loads**

$$M_{t1} = M_{tcd}\left(1 - 0.8V_{pd}/V_{yd}\right)$$

$$M_{t2} = 0.2M_{tcd}\,V_{pd}/V_{yd} + M_{tyd}\left(1 - V_{pd}/V_{yd}\right)$$

M_{tcd} : **design pure torsion capacity in the case where there is no torsion reinforcement; to be determined in accordance with Section 9.2.3.2, where the material factor γ_c for concrete and the member factor γ_b are usually 1.0**

M_{tvd} : **design torsion capacity determined by yielding of torsion reinforcement; to be determined in accordance with Section 9.2.3.3, where the member factor γ_b is usually 1.0**

V_{pd} : **design shear force produced by permanent load**

V_{vd} : **design shear capacity of a bar member; to be determined in accordance with Section 9.2.2.2, where the material factor γ_c for concrete and the member factor γ_b are usually 1.0**

f_{wyd} : **design tensile yield strength of transverse torsion reinforcement**

[Commentary] (1) For the calculation of concrete and steel stresses in reinforced concrete structures under normal service conditions, conventional assumptions are indicated. If the finite element method or an analysis method that directly uses the material stress–strain relationship such as a fiber model is used as a structural analysis method, material response values may be calculated as described in this section by using sectional forces obtained in accordance with Section 7.4.2.

(2) If both vertical stirrups and bent-up bars are used for shear reinforcement, stresses in them may be calculated by using Eqs. (C7.4.1) - (C7.4.4).

vertical stirrups : $$\sigma_{wrd} = \frac{V_{pd} + V_{rd} - k_r V_{cd}}{\dfrac{A_w z}{s} + \dfrac{A_b(\cos\theta_b + \sin\theta_b)^3}{s_b}} \cdot \frac{V_{pd}}{V_{pd} + V_{rd} + V_{cd}} \quad \text{(C7.4.1)}$$

$$\sigma_{wpd} = \frac{V_{pd} + V_{rd} - k_r V_{cd}}{\dfrac{A_w z}{s} + \dfrac{A_b(\cos\theta_b + \sin\theta_b)^3}{s_b}} \cdot \frac{V_{pd} + V_{cd}}{V_{pd} + V_{rd} + V_{cd}} \quad \text{(C7.4.2)}$$

bent bars : $$\sigma_{brd} = \frac{V_{pd} + V_{rd} - k_r V_{cd}}{\dfrac{A_w z}{s(\cos\theta_b + \sin\theta_b)} + \dfrac{A_b(\cos\theta_b + \sin\theta_b)}{s_b}} \cdot \frac{V_{pd}}{V_{pd} + V_{rd} + V_{cd}} \quad \text{(C7.4.3)}$$

$$\sigma_{bpd} = \frac{V_{pd} + V_{rd} - k_r V_{cd}}{\dfrac{A_w z}{s(\cos\theta_b + \sin\theta_b)} + \dfrac{A_b(\cos\theta_b + \sin\theta_b)}{s_b}} \cdot \frac{V_{pd} + V_{cd}}{V_{pd} + V_{rd} + V_{cd}} \quad \text{(C7.4.4)}$$

where, σ_{wrd} : design variable stress in vertical stirrups

σ_{wpd} : design stress in vertical stirrups due to permanent load

σ_{brd}: design variable stress in bent bars

σ_{bpd}: design stress in bent bars due to permanent load

s : spacing of vertical stirrups

s_b : spacing of bent bars

A_w : total cross-sectional area of vertical stirrups in section s

A_b : total cross-sectional area of bent bars in section s_b

θ_b : angle between bent bar and member axis

(3) Eq. 7.4.3 was derived by assuming that stress in lateral torsion reinforcement increases as cracking occurs and reaches f_{wyd} at failure. A previous study has shown that the torsion crack width calculated by assuming that when torsional moment acts repeatedly after the occurrence of cracking, stress in lateral torsion reinforcement becomes zero at or below 70% of the cracking moment M_{tc} and increases linearly as torsional moment increases thereafter shows close agreement with experimental results.

In cases when flexural moments are applied to a beam having torsional cracks on both the upper and lower sides, or in cases when torsional moments are applied to a beam that has flexural cracks on both its upper and lower sides, the direction of principal tensile stress due to torsional moment and the direction of principal tensile stress due to flexural moment do not coincide. Therefore, the effect of their interaction is less than the interaction between the torsional moment and shear force. Further, in cases of vertically long rectangular section subjected to torsional moment, tensile stresses on upper and lower sides are smaller than those on the right and left sides. Thus, the effect of torsional moment on the stresses in longitudinal flexural reinforcement is considered small.

Hence, the examination may be omitted in cases when a beam is subjected to combined loading due to flexural and torsional moments.

Cracking associated with compatibility torsion does not need to be considered mainly because torsion moment is released considerably owing to initial cracks, because such cracks, which occur under normal service conditions, are relatively small and crack width does not tend to increase and because each member has at least a minimum amount of shear reinforcement so as to prevent torsion cracking. The use of U-shaped stirrups in a beam, however, may cause excessively large cracks on the upper surface of the beam. For this reason, if the height of a rectangular cross section is not greater than three times the member width, closed stirrups should be used.

7.4.4 Examination for flexural cracks

(1) The crack width, w, may be calculated using Eq.(7.4.4).

$$w = 1.1 k_1 k_2 k_3 \{4c + 0.7(c_s - \phi)\} \left[\frac{\sigma_{se}}{E_s} \left(or \ \frac{\sigma_{pe}}{E_p} \right) + \varepsilon'_{csd} \right] \quad \textbf{(7.4.4)}$$

where, k_1 **: a constant to take into account the effect of surface geometry of reinforcement on crack width. It may be taken to be 1.0 for deformed bars, 1.3 for plain bars and prestressing steel.**

k_2 **: a constant to take into account the effect of concrete quality on crack width. It may be calculated using Eq.(7.4.5).**

$$k_2 = \frac{15}{f'_c + 20} + 0.7 \quad \textbf{(7.4.5)}$$

f'_c **: compressive strength of concrete (N/mm^2). In general, it may be taken to be equal to the design compressive strength,** f'_{cd} **.**

k_3 **: a constant to take into account the effect of multiple layers of tensile reinforcement on crack width. It may be calculated using Eq.(7.4.6).**

$$k_3 = \frac{5(n+2)}{7n+8} \quad \textbf{(7.4.6)}$$

n **: number of the layers of tensile reinforcement.**

c **: concrete cover (mm).**

c_s **: center-to-center distance of tensile reinforcements (mm).**

ϕ **: diameter of tensile reinforcement (mm).**

ε'_{csd} **: compressive strain for evaluation of increment of crack width due to shrinkage and creep of concrete.**

σ_{se} **: increment of stress of reinforcement from the state in which concrete stress at the portion of reinforcement is zero (N/mm^2).**

σ_{pe} : **increment of stress of prestressing steel from the state in which concrete stress at the portion of reinforcement is zero (N/mm^2).**

(2) In the examination for flexural cracks, in principle the reinforcement or prestressing steel nearest to the surface of concrete, should be considered, and their stresses should be calculated in accordance with Section 7.2.

[Commentary] (1) Flexural cracking in reinforced and prestressed concrete is affected by various factors. According to the previous studies, the main factors are the types of reinforcement, increase of stress in reinforcement, concrete cover, effective cross sectional area of concrete, diameter of reinforcement, ratio of reinforcement, number of layers of reinforcement, surface geometry of reinforcement, quality of concrete, magnitude of prestress, etc.

Equation (7.4.4) has been formulated on the basis of the existing equations for prediction of crack width and results of recent studies. A more appropriate equation for crack width calculation should be used in cases when reinforcement of different diameters are used together, high tensile strength deformed bars with strength of 490 N/mm^2 or more are used, and the effect of repeated load is significant

Stresses increment in reinforcement, σ_{se} (or σ_{pe}), should be computed considering the state of stress of both concrete and reinforcement. When the concrete stress at the same position of the reinforcement induced by stress resultant is in compression, σ_{se} (or σ_{pe}) should be zero even if stress in the reinforcement increases due to stress resultant. When concrete stress changes from compression to tension under stress resultant σ_{se} (or σ_{pe}) is defined as the stress difference between the reinforcement stress when concrete stress changes from compression to zero and the reinforcement stress induced by stress resultant. In case that autogenous or drying shrinkage of concrete my not be neglected, σ_{se} (or σ_{pe}) is defined as the stress difference between the reinforcement stress when concrete stress changes from tension to zero and the reinforcement stress induced by stress resultant. The stress in the reinforcement when concrete stress changes from tension to zero depends also on the ratio of reinforcement and may be more than reinforcement stress corresponding to compressive strain of 300x10^{-6}. However, it can be reduced to a value less than 200x10^{-6} if appropriate curing conditions for concrete are provided.

If shrinkage-compensating concrete is used, unlike in the case of shrinkage, the state of compression of concrete is regarded as the initial state. The amount of change, therefore, in the steel stress caused by sectional forces from the steel stress at the time the concrete stress reaches zero from compression is taken as the amount of increase σ_{se} (or σ_{pe}). The steel stress occurring until the concrete stress reaches zero from compression is calculated as the sum of the steel strain corresponding to the chemical prestress introduced into the concrete and the chemical prestrain multiplied by Young's modulus. Because those values are affected by such factors as concrete mix proportions, the shape and dimension of the member, the state of reinforcement, curing method and environmental conditions, it is necessary to determine those values appropriately according to past performance records, reliable literature and test results. If experimental results are not used, those values may be determined by applying the concept of the amount of work. For shrinkage-compensating concrete, the calculation of steel stress occurring until the concrete stress reaches zero from compression may be omitted. In such cases, ε'_{csd} may be reduced according to the chemical prestress introduced by expansion and the elastic strain of concrete. If compressive strength is higher than about 60 N/mm^2, chemical prestress and chemical prestrain may be calculated through linear creep analysis allowing for the time-dependent mechanical characteristics of concrete.

Spacing of flexural cracks is affected by the bond between the reinforcement and the concrete. Coefficient k_1 in Eq.(7.4.4) is a constant to represent the effect of surface geometry of reinforcement, which is one of the bond factors affecting crack width. Equation (7.4.5) should basically be applied to members with deformed bars. In the case of plain bars and prestressing steel, although changes in the crack width have not been clarified, k_1 may tentatively be taken to be 1.3. In the case of prestressing steel in pretensioned members, k_1 may be set to a value not greater than 1.3 (but not less than 1.0) provided its surface geometry provides good bonding

Coefficient k_2 in Eq.(7.4.4) is a constant to represent the effect of changes in bonding characteristics between the reinforcement and the concrete due to changes of concrete quality on crack width. It has been reported that concrete with little material segregation and having a dense pore structure, possesses not only high resistance to cracking but also reduces crack width due to good bonding. Equation (7.4.5) expresses the effect of concrete quality on crack width in terms of compressive strength of concrete that represents properties of hardened concrete. Even for concrete with a low strength, k_2 may be reduced to 0.9 if bleeding of concrete is kept small and homogenous concrete cover is assured.

Coefficient k_3 in Eq.(7.4.4) is a constant to represent the effect of the reinforcement in the second and higher layers in members with multi layers of reinforcements on the surface crack width. Equation (7.4.6) has been formulated by simplifying the term representing reinforcement layers in the prediction equation for crack width in a previous study.

ε'_{csd} in Eq.(7.4.4), which represents the effect of shrinkage and creep of concrete on crack width, should be determined with the consideration of shape of cross section of the member, environmental condition, magnitude of stress, and others. Since the value of ε'_{csd} differ in each verification, it is necessary to calculated by using the value listed in 8.3.3, 10.3.2, 15.4.4, etc.

Loads used in calculation of σ_{se} (or σ_{pe}) shall be determined considering time-dependent changes in crack width due to sustained and repeated application of variable loads, duration of sustained crack opening and other factors. For structures where the ratio of permanent to total loads is small, and the frequency of application of variable loads is high, the effect of variable loads should be set at a higher level, while for structures where the ratio of permanent to total loads is high and the frequency of application of variable loads is low, the effect of permanent loads may be set to a smaller level.

The method for examination of crack width given here is not applicable to cases where members have only prestressing steel as longitudinal reinforcement, because there have been only a few studies on crack widths in prestressed members without longitudinal reinforcing steel bars. Further, deformed bars should be used as longitudinal reinforcement in prestressed concrete, because only limited experimental data with the use of plain bars is available, and deformed bars are considered effective for the purpose of crack control.

(2) In cases when longitudinal reinforcement is provided in several layers, in general, the stress in the reinforcement nearest to the concrete surface should be used. In such cases, although the stress acting at the centroid of reinforcement may be taken, to simplify the computation, coefficient, k_3, in Eq.(7.4.4) that accounts for the presence of several layers of reinforcement, shall be taken as 1.0, regardless of the number of layers. In the case of thin members with several layers of reinforcement, the stress of reinforcement nearest to the surface is preferably used, as using the method described above, may provide results, which may not be on the conservative side.

7.4.5 Examination for displacement and deformation of member

(1) Short-term displacement and deformation of concrete members without cracks may be computed based on the theory of elasticity assuming that the gross cross section is effective.

(2) Short-term displacement and deformation of concrete members with flexural cracks shall be computed considering reduction of stiffness due to the cracks.

(3) Long-term displacement and deformation of concrete members shall be computed considering effects of creep due to permanent loads, shrinkage and crack of concrete.

(4) Short-term shear displacement and deformation of concrete members with shear cracks shall be computed considering reduction of stiffness due to the cracks.

[Commentary] (1) In cases when precise computation of displacements and deformations are not required, displacements and deformations of cracked reinforced and cracked prestressed concrete members, may be computed using the moment of inertia, I_g, of a gross cross section assuming no flexural crack.

(2) and (3) In computation of short-term or long-term displacements and deformations, for cases when reduction of stiffness due to flexural cracking and the influence of creep and shrinkage are taken into account, the effective flexural stiffness evaluated using Eq.(C7.4.5) or Eq.(C7.4.6) may be used.

(i) In cases when the effective flexural stiffness is assumed as a function of flexural moment

$$E_e I_e = \left(\frac{M_{crd}}{M_d}\right)^4 \frac{E_e I_g}{1-\dfrac{\Delta M_{csg}}{M_d - P(d_p - c_g)}} + \left\{1-\left(\frac{M_{crd}}{M_d}\right)^4\right\} \frac{E_e I_{cr}}{1-\dfrac{\Delta M_{cscr}}{M_d - P(d_p - c_{cr})}} \qquad \text{(C7.4.5)}$$

(ii) In cases when the effective flexural stiffness is assumed constant in the longitudinal direction

$$E_e I_e = \left(\frac{M_{crd}}{M_{d\max}}\right)^3 \frac{E_e I_g}{1-\dfrac{\Delta M_{csg}}{M_{d\max} - P(d_p - c_g)}} + \left\{1-\left(\frac{M_{crd}}{M_{d\max}}\right)^3\right\} \frac{E_e I_{cr}}{1-\dfrac{\Delta M_{cscr}}{M_{d\max} - P(d_p - c_{cr})}} \qquad \text{(C7.4.6)}$$

where, E_e : effective elastic modulus which can be computed using Eq.(C7.4.7)

$$E_e = \frac{E_{ct}}{1+\varphi} = \frac{E_{ct}}{1+(E_{ct}/E_c)\varphi_{28}} \qquad \text{(C7.4.7)}$$

E_{ct} : Young's modulus when dead loads are applied

E_c : Young's modulus at the age of 28 days

φ : creep coefficient since dead loads is applied, which is computed using Young's modulus at the age when the dead loads start to act

φ_{28} : creep coefficient since dead loads are applied, which is computed using Young's modulus at the age of 28 days

I_e : effective moment of inertia of transformed cross section for either short-term or long-term

M_{crd} : critical flexural moment when a flexural crack occurs in the cross section; or the flexural moment when flexural tensile stress in concrete at the extreme tension edge, in which axial force and prestressing force are considered, is equal to flexural cracking strength, f_{bck}, where γ_c and γ_b are set equal to 1.0 in general

M_d : design flexural moment to be used in the computation of either short-term or long-term displacements and deformations

$M_{d\max}$:maximum value of design flexural moment to be used in the computation of either short-term or long-term displacements and deformations

P : axial force or prestressing force in either short-term or long-term

ΔM_{csg} : apparent flexural moment due to shrinkage in gross section and restraint by reinforcements, which can be computed by Eq.(C7.4.8)

$$\Delta M_{csg} = E_s \left\{ \frac{I'_{sg}}{c_g - d'} - \frac{I_{sg}}{d - c_g} - \frac{I_{pg}}{d_p - c_g} \right\} \varepsilon'_{cs} \tag{C7.4.8}$$

$\Delta M_{\mathrm{csc}r}$: apparent flexural moment due to shrinkage in the section excluding concrete under tensile stress (hereafter "cracked section") and restraint by reinforcements, which can be computed using Eq.(C7.4.9)

$$\Delta M_{\mathrm{csc}r} = E_s \left\{ \frac{I'_{scr}}{c_{sr} - d'} - \frac{I_{scr}}{d - c_{cr}} - \frac{I_{pcr}}{d_p - c_{cr}} \right\} \varepsilon'_{cs} \tag{C7.4.9}$$

I_g : moment of inertia of gross cross section about its centroid for either short-term or long-term

I'_{sg} : moment of inertia of compression reinforcement about the centroid of gross cross section for either short-term or long-term

I_{sg} : moment of inertia of tensile reinforcements about the centroid of the gross cross section for either short-term or long-term

I_{pg} : moment of inertia of the prestressing steel about the centroid of gross cross section for either short-term or long-term

I_{cr} : moment of inertia of cracked cross-section around the centroid of cracked cross-section for either short-or long-term

I'_{scr} : moment of inertia of compression reinforcement around the centroid of cracked cross-section for either short- or long-term

I_{scr} : moment of inertia of tensile reinforcement around the centroid of cracked cross-section for either short- or long-term

I_{pcr} : moment of inertia of prestressing steel around the centroid of cracked cross section for either short-term or long-term

c_g : distance from the compression edge to the centroid of gross cross section for either short-term or long-term

c_{cr} : distance from the compression edge to the centroid of cracked cross section for either short-term or long-term

d' : distance of compression reinforcement from the extreme compression fiber edge to

d : distance of tensile reinforcement from the extreme compression fiber

d_p : distance of prestressing steel from the extreme compression fiber

ε'_{cs} : shrinkage strain

Both Eq.(C7.4.5) and Eq.(C7.4.6) can be used for practical computation of short-term and long-term displacements and deformations of both cracked and uncracked reinforced and prestressed concrete members. The effect of creep is considered in terms of the effective elastic modulus. It is assumed that shrinkage of concrete is induced uniformly across the cross-section. Prestressing force is taken as part of permanent loads. In computing deformations due to permanent loads, though bonded prestressing steel may be treated in a manner similar to ordinary reinforcing steel, unbonded prestressing steel may be neglected. Time-dependent changes in tension stiffness of concrete are neglected as approximate computation though, in reality, bonding stress reduces slightly with time.

Flexural stiffness of a member without flexural cracks may be obtained using Eq.(C7.4.5) or Eq.(C7.4.6), assuming that $M_{crd} = M_d$ or $M_{crd} = M_{d\max}$.

In the computation of short-term displacements and deformations, both the creep coefficient and shrinkage of concrete may be assumed to be zero. The effect of reinforcement may be neglected in computing approximate values for moment of inertia of gross section.

In computation of long-term displacements and deformations, creep coefficient and shrinkage of concrete may be calculated using the concrete age since the time of application of dead loads. The effect of reinforcement shall be taken into account in computing the moment of inertia of the gross section. In cases when the age when prestressing is introduced and the age when loading starts are considerably different, or in cases when total deformation under dead and live loads is required, it is advisable to consider the effect of the difference in ages at application of such loads for a precise computation of displacements and deformations.

In cases when Eq.(C7.4.6) is applied to continuous beams, $M_{d\max}$ may be taken as the maximum positive flexural moment.

Displacements and deformations in the case of (i) may be obtained using numerical integration of curvature at each cross section, $\lfloor \{M_d - P(d_p - c_e)\} / E_e I_e \rfloor$, computed with $E_e I_e$ obtained using Eq.(C7.4.5). The distance c_e to be used in computation of displacements and deformations may be evaluated using either Eq.(C7.4.10) or Eq.(C7.4.11).

(i) In cases when the effective flexural stiffness is assumed to be a function of flexural moment,

$$c_e = \left(\frac{M_{crd}}{M_d}\right)^4 c_g + \left\{1 - \left(\frac{M_{crd}}{M_d}\right)^4\right\} c_{cr} \qquad \text{(C7.4.10)}$$

(ii) In cases when the effective flexural stiffness in the longitudinal direction is assumed to be constant,

$$c_e = \left(\frac{M_{crd}}{M_{d\max}}\right)^3 c_g + \left\{1 - \left(\frac{M_{crd}}{M_{d\max}}\right)^3\right\} c_{cr} \qquad \text{(C7.4.11)}$$

Approximate deflections in statically indeterminate structures, such as continuous beams, may be computed using numerical integration for each member, using the flexural moment obtained from elastic analysis

Equation (C7.4.6) in (ii) is originally for computing deflection in cases when the distribution of flexural moment is parabolic. Though in a strict sense, the power term in the equation should depend on the shape of distribution of flexural moment, third power has been adopted because it had been found that using the third power does not introduce major errors.

In cases when an axial force is applied, the ratio of flexural moments in Eq.(C7.4.5) and Eq.(C7.4.6) may be substituted by the ratio of the resulting tensile forces in the reinforcement.

In cases when there are no cracks in cross sections, approximate long-term displacements and deformations may be computed as the sum of short-term displacements and deformations due to permanent loads and their creep components as given in Eq.(C7.4.12).

$$\delta_l = (1+\varphi)\delta_{ep} \tag{C7.4.12}$$

where, δ_l : long-term displacements and deformations

δ_{ep} : short-term displacements and deformations due to permanent loads

(4) Though shear stiffness of concrete members is reduced after onset of flexural or tensile cracks, the effect of the reduction of shear stiffness on deformations of members is, in general, small compared with the effect of flexural stiffness. The reduction of shear stiffness can be computed using methods similar to those used to compute reduction in flexural stiffness. It has been found from experimental work, however, that since shear stiffness of members is substantially reduced after shear cracking, deformations after shear cracking are underestimated when a conventional computational method in which only reduction of flexural stiffness is taken into account, is used.

In the computation of shear deformation of members with shear reinforcements subjected to flexural shear, the following method may be used after flexural and shear cracks occur.

(i) After flexural cracking and before shear cracking, shear deformation, δ_s, may be computed using Eq.(C7.4.13), assuming a reduced shear stiffness, GA_e, as given in Eq.(C7.4.14).

$$GA_e = G\left[A_g\left(\frac{M_{crd}}{M_{d\max}}\right)^3 + A_{cr}\left\{1-\left(\frac{M_{crd}}{M_{d\max}}\right)^3\right\}\right] \le GA_g \tag{C7.4.13}$$

$$\delta_s = k\int\frac{V_d}{GA_e}dx \tag{C7.4.14}$$

where, G : shear elastic modulus, which may be assumed to be the same as the shear elastic modulus of concrete, G_c

A_g : gross cross-sectional area before flexural cracking

A_{cr} : area of flexural compression zone in the cross section after flexural cracking

k : coefficient depending on shape of cross section

V_d : design shear force

(ii) After shear cracking, whether flexural cracks have occurred or not, the shear deformation may be computed by Eq.(C7.5.14), where, as shown in Fig. C7.4.1, shear deformation of the member may be taken as the deformation of the truss mechanism, assuming concrete as the compression diagonal member (*BE* in the figure) and shear reinforcements as the tensile diagonal member (*CE* in the figure).

$$\delta_s = \int \gamma dx = \int \frac{1}{z(\cot\theta + \cot\alpha)^2} \left[\frac{V_{sd}}{E_c b_w \sin^4\theta} + \frac{V_{sd} s}{E_w \left(A_w + \frac{E_c}{E_w} A_{ce} \right) \sin^3\alpha} \right] dx \qquad \text{(C7.4.15)}$$

where, γ : shear strain

z : distance from compression resultant to centroid of tension reinforcement. In general it may be taken as $d/1.15$

θ : angle between compression diagonal member and longitudinal axis of member. It may be computed using Eq.(C7.4.16)

$$\theta = 45° - k \frac{V_d - V_{cd}}{b_w d} \qquad \text{(C7.4.16)}$$

$$k = (3.2 - 7800 p_t p_w)(a/d) \qquad \text{(C7.4.17)}$$

p_t : ratio of tensile reinforcement, $A_s/(b_w d)$

p_w : ratio of shear reinforcement, $A_w/(b_w s)$

a : shear span, which is the distance from loading point to the front of the support

d : effective depth

b_w : width of web

α : angle between shear reinforcement and the longitudinal axis of member

V_{sd} : design shear capacity of shear reinforcements, $V_d - V_{cd}$

V_{cd} : design shear capacity from sources other than shear reinforcements. It may be computed using Eq.(6.3.3),where the material factor, γ_c, may be taken to be equal to 1.0

E_c : elastic modulus of concrete

E_w : elastic modulus of shear reinforcement

A_w : area of shear reinforcements provided over the spacing, s

A_{ce} : area of concrete around the reinforcements effective as tensile diagonal member, which may be computed using Eq.(C7.4.18)

$$A_{ce} = A_{ceo}\left(V_c / V\right)^3 \tag{C7.4.18}$$

A_{ceo} : A_{ce} immediately after shear cracking

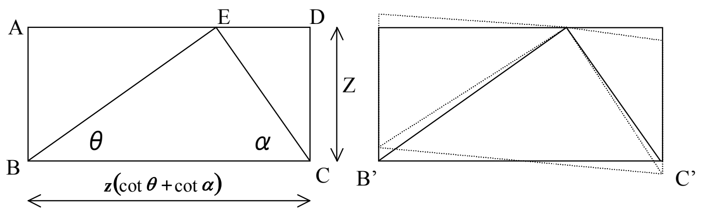


Fig. C7.4.1 Shear deformation of truss mechanism

(iii) Tensile force in the tension reinforcement increases after formation of shear cracks as explained by the truss mechanism. This phenomenon is known as the "moment shift". The increase in deformation due to this increase in tensile force may be computed as flexural deformation assuming that flexural moment, which induces the same magnitude of tensile force in the reinforcement, is applied. The increment of tensile force may be computed using Eq.(C7.4.19).

$$\Delta T = \left[\cot\theta - \frac{\sin(\theta+\alpha)}{2\sin\theta\sin\alpha}\right]V_s \tag{C7.4.19}$$

CHAPTER 8　VERIFICATION OF DURABILITY

8.1　General

(1) Every structure shall maintain the required performance throughout the design service life of the structure.

(2) Verification to ascertain that the required performance of a structure will not be lost because of steel corrosion due to chloride attack or carbonation or concrete deterioration due to freezing damage or chemical attack may be made by the methods described in this chapter. If two or more of these occur in combination, their influence should be taken into consideration.

[Commentary]　(2) In order to maintain the required performance of a structure throughout its design service life, it is common practice to prevent the deterioration or changes in the state of the materials in the structure due to environmental factors or to design the structure so that material deterioration can be kept within the range in which the performance of the structure does not decline.　This chapter describes methods for checking whether a structure meets these requirements with respect to representative types of deterioration of concrete structures, namely, steel corrosion due to chloride attack and carbonation and concrete deterioration due to freezing damage and chemical attack.　Study methods associated with alkali–aggregate reaction and chloride attack caused by chloride ions existing in concrete mixes are described in the Construction section of this Specification because they should be considered at the stage at which materials to be used are selected.

A structure can be deemed to be capable of meeting the safety, serviceability and earthquake resistance requirements throughout the design service life if (1) the structure meets the verification criteria described in this chapter to ensure that problems due to the deterioration of materials in the structure will not occur during the design service life and (2) the structure meets the safety, serviceability and earthquake resistance criteria indicated in Chapters 9, 10 and 11.

Although factors affecting the durability of a concrete structure may act on the structure independently, usually two or more of those factors act in combination.　In many cases, evaluating the influence of dominant factors independently suffices.　There is as yet no established verification technique for evaluating the effect of a combination of two or more factors.　This section, therefore, describes performance verification methods for individual factors.　If a structure is subjected to a combination of two or more factors, the deterioration of the structure may be accelerated.　If a high degree of influence is expected, therefore, it is desirable that control measures such as using a large factor of safety be taken.

Even in cases where the performance of a structure has deteriorated over years of use, there is no problem if the performance requirements are met.　Research and development efforts in the coming years make highly reliable prediction of structural deterioration possible, and performance verification technology for structures that takes changes over time into consideration may be established.　If such changes occur, it may become no longer necessary to assume no deterioration over time, and it may make economical design with a high degree of freedom possible.

8.2 Environmental Factors

(1) The degree of dryness should be taken into consideration when predicting the progress of carbonation of concrete.

(2) The influence of adventitious chlorides on the penetration of chloride ions into the concrete may be evaluated on the basis of the amount of adventitious chlorides arriving at the structure.

(3) Environmental factors related to freezing damage shall be determined according to the number of freeze–thaw cycles and the water content of concrete.

(4) Environmental factors related to chemical attack shall be determined according to the type of chemically corrosive substances and the intensity of chemical attack.

[Commentary] (1) The carbonation of concrete is caused by carbon dioxide penetration from the atmosphere. Because civil engineering structures are in many cases constructed outdoors, the degree of dryness of concrete, which affects carbon dioxide penetration, is taken into consideration. This means that the rate of carbonation of concrete exposed to sunlight and located on the south side of a structure tends to be higher than that of concrete in a rainfall-affected environment or a relatively humid environment.

When predicting the rate of carbonation by the method described in Section 8.3.6, the influence of environmental factors on the rate of carbonation may be evaluated in terms of a factor (βe) indicating the degree of environmental influence shown in Table C8.2.1.

Table C8.2.1 Degree of environmental influence on prediction of the carbonation of concrete

Environmental conditions	Factor indicating the degree of environmental influence (βe)
Relatively dry environment	1.6
Relatively humid environment	1.0

(2) When chloride ion penetration into concrete is predicted by the method described in Section 8.3.7, the influence of adventitious chlorides on the rate of chloride ion penetration is expressed with chloride ion concentration at the concrete surface. The amount of airborne chlorides deposited on the concrete surface is affected by such factors as the climate of the region in which the structure is constructed, the distance and height from the coast, topography and wind direction. In general, as the amount of airborne chlorides increases, the amount of chloride ions that penetrate into concrete increases. Even when the amount of airborne chlorides is large, however, the amount of chlorides that penetrate into concrete may not become large if chlorides are washed off by, for example, rain. Also, it is generally known from measurement results that chloride ion concentration at the concrete surface of a structure is affected by not only the environmental conditions surrounding the structure but also the quality of concrete. When conducting verification for individual structures, it is desirable that values of chloride ion concentration at the concrete surface, based on performance data on similar structures and measurement data, that accurately express the environmental conditions surrounding the structure

be used.

If neither performance data on similar structures nor measurement data are used, chloride ion concentration at the concrete surface may be determined from Table C8.2.2 according to the type of the area in which the structure is located and the distance from the coast.

Table C8.2.2 Chloride ion concentration C_0 at concrete surface (kg/m^3)

<table>
<tr><th colspan="2" rowspan="2"></th><th rowspan="2">Splash zone</th><th colspan="5">Distance from coast (km)</th></tr>
<tr><th>Near shoreline</th><th>0.1</th><th>0.25</th><th>0.5</th><th>1.0</th></tr>
<tr><td>Region with high airborne chloride concentration</td><td>Hokkaido, Tohoku, Hokuriku, Okinawa</td><td rowspan="2">13.0</td><td>9.0</td><td>4.5</td><td>3.0</td><td>2.0</td><td>1.5</td></tr>
<tr><td>Region with low airborne chloride concentration</td><td>Kanto, Tokai, Kinki, Chugoku, Shikoku, Kyushu</td><td>4.5</td><td>2.5</td><td>2.0</td><td>1.5</td><td>1.0</td></tr>
</table>

With respect to height in the coastal zone, C_0 may be calculated assuming that a height of 1 m corresponds to a distance of 25 m from the shoreline.

(3) Major factors affecting the occurrence and acceleration of freezing damage are freeze–thaw cycles and the water content of concrete. When considering environmental factors affecting freezing damage, it is therefore necessary to take into consideration such factors as the lowest temperature, the number of freeze–thaw cycles and meteorological phenomena that supply water to the concrete. Since, however, there is as yet no established verification method that incorporates these factors, combinations of the meteorological phenomena and the states of exposure of the structure listed in Table C8.2.3 may be regarded as environmental factors affecting freezing damage.

Table C8.2.3 Environmental factors affecting freezing damage

Meteorological phenomena	State of exposure of structure
• Environment with frequent freeze–thaw cycles	• Continuously or frequently saturated with water *
• Environment in which below-zero temperature is rare	• Normal state of exposure (other than the cases mentioned above)

* Parts of a structure such as water channels, water tanks, abutments, piers, retaining walls and tunnel lining that are close to a water surface and are saturated with water, and parts of a structure such as girders and floor slabs besides those mentioned above that are distant from a water surface but are saturated with water because of snowmelt, flowing water, splashes of water, etc.

(4) Substances that chemically attack concrete can be largely classified, according to the mechanisms of attack, into two categories. One category includes substances that react with cement hydrates in the concrete to alter them into soluble substances so as to cause the concrete to deteriorate. Many acids, some types of inorganic salts and corrosive gases such as hydrogen

sulfide belong to this category. The other category includes substances that react with cement hydrates in the concrete to produce expansive compounds so as to cause the concrete to deteriorate by expansion pressure. It can be said that not only certain types of structures such as sewer-related facilities and chemical plants but also structures located in hot spring areas, acid rivers, acid or sulfate soil areas, etc., are in a chemically corrosive environment. In connection with those structures, it is necessary to determine the intensity of chemical activity as an environmental factor affecting chemical attack, taking into consideration the type and concentration of chemical substances that attack concrete, temperature, humidity, etc.

8.3 Verification Related to Steel Corrosion

8.3.1 General

During the design service life of a structure, the required performance of the structure shall not be lost under given environmental conditions because of steel corrosion due to chloride ion penetration, carbonation, etc. In general, verification may be conducted by checking on items (i), (ii) and (iii) listed below.

(i) Crack width at the concrete surface is not greater than the critical crack width for steel corrosion.

(ii) Chloride ion concentration at the steel location does not reach the critical concentration for steel corrosion during the design service life.

(iii) Carbonation depth does not reach the critical depth for steel corrosion during the design service life.

In general, studies on steel corrosion do not need to be conducted for structures with a very short service period, temporary structures, etc.

[Commentary] Chloride ion penetration into concrete and the carbonation of concrete cause the corrosion of the steel in the concrete. This chapter describes a method of study on steel corrosion due to chloride ion penetration and a method of study on steel corrosion due to carbonation. Both methods assume one-dimensional migration of substances from the concrete surface to the reinforcing steel. These verification methods are valid only when local corrosion does not occur at the crack locations. In order to make this assumption valid, crack width must be small. This is why it is verified by check on Item (i) that crack width is not greater than the critical crack width for steel corrosion, and then Item (ii), chloride ion concentration at the steel location, and Item (iii), carbonation depth, are checked.

Checks on Items (ii) and (iii) are not necessary for a structure used in a normal outdoor environment where airborne chloride ions do not exist or in an environment free from the risk of carbonation. Even in such cases, however, it is desirable that crack width be kept within the allowable crack width because excessively large crack widths are undesirable.

8.3.2 Limit value of crack width for corrosion of reinforcement

(1) The limit value of crack width for corrosion of reinforcement may be determined depending on the environmental conditions of structure. In general, environmental conditions may be classified into "normal environment", "corrosive environment" and "severely corrosive environment" as given in Table 8.3.1.

Table 8.3.1 Classification of environmental conditions for reinforcement corrosion

Normal environment	Normal outdoor environment with ordinary conditions without any airborne salt, underground, etc.
Corrosive environment	1. In comparison to the normal environment, environment with more frequent cyclic drying and wetting, and underground environment below the level of underground water containing especially corrosive (or detrimental) substances, which may cause harmful corrosion of reinforcement. 2. Environment of marine structures submerged in seawater, or structures not exposed to severe marine environment, etc.
Severely corrosive environment	1. Environment in which reinforcement is subjected to detrimental influences considerably. 2. Environment of marine structures subjected to tides, splash, or exposed to severe ocean winds, etc.

(2) The limit value of crack width for corrosion of reinforcement may be determined depending on the environmental conditions, concrete cover and type of reinforcement, as given in Table 8.3.2. However, the concrete cover c used in Table 8.3.2 should not exceed 100mm.

Table 8.3.2 Limit value of crack width w_a

Type of reinforcement	Environmental conditions for reinforcement corrosion		
	Normal	Corrosive	Severely corrosive
Deformed bars and plain bars	$0.005c$	$0.004c$	$0.0035c$
Prestressing steel	$0.004c$	----	----

[Commentary] In general, cracks occurring in concrete structures become a cause for reduction in durability due to reinforcement corrosion, deterioration in functions such as watertightness and airtightness, large deformations, impairment of appearance, etc. Therefore, it shall be examined by an appropriate method that such functions of structures are not impaired due to cracking in concrete. This clause deals with examination of crack from the viewpoint of prevention of reinforcement corrosion in concrete. Examination of crack for watertightness and appearance is dealt in Chapter 10.

This clause deals with reinforced concrete members and prestressed concrete members in which cracking is allowed. Examination for cracking in composite steel-concrete structures is given in Chapter 16.

(1) Durability of concrete structures is greatly affected by corrosion of reinforcement. Cracks in concrete cover with excessive width may cause local corrosion of reinforcement. Depending upon the extent to which they affect the corrosion, environmental conditions have been classified into three categories as shown in Table 8.3.1. For the different environmental conditions, the limit value of crack width should be determined. The limit states of corrosion of reinforcement due to chloride ingress and carbonation shall be examined on the assumption that crack widths are smaller

than the permissible crack width.

"Severely corrosive environment" given in Table 8.3.1 corresponds to environments such as tidal zones with cyclic wetting and drying or even air with occasional splashing of seawater. Structures frequently exposed to deicing agents such as calcium chloride shall be regarded as being in "severely corrosive environment." "Corrosive environment" corresponds to environments that are more severe than the "normal environment" but are not as severe as "severely corrosive environment". For instance, the condition in which concrete is permanently submerged in seawater or exposed to air containing salt. Structures located between 0.1 and 1 km from the shoreline may be regarded as being in "corrosive environment".

The term "marine" in Table 8.3.1 includes harbor, coast and ocean. In classification of environmental conditions of a structure, not only the environment where the structure is located but also the importance of the structure and other factors should be taken into account.

In the case of indoor or similar members, which are not exposed to wind and rain, examination of crack widths for durability is not required. An environment with frequent freezing and thawing may be classified as "corrosive" because an increase in crack width indirectly accelerates the corrosion of reinforcement.

(2) Corrosion of reinforcement in concrete does not depend only on crack width. Hence, the permissible crack widths indicated in this clause are minimum requirements to avoid risk of corrosion of reinforcement. It does not mean that reinforcement do not corrode once crack widths are kept to levels smaller than the permissible width. For structures under "corrosive" or "severely corrosive" environments not only should the crack widths be kept to levels smaller than the limit value of crack width, but also an examination for the concentration of chloride ions at reinforcement level given in Section 8.2.7 should be carried out.

It is reasonable to consider that limit value of crack width at the surface of members, which has a great influence on corrosion of reinforcement, depends on concrete cover. Thus, assuming that the permissible crack width may be increased with increasing concrete cover, the limit value of crack widths given in Table 8.3.2 have been determined depending on the environmental conditions and the type of reinforcement. However, since the values given in this table are not definite values, the permissible crack widths may be determined considering the environmental conditions of the structure and the method used to estimate the width of cracks.

Permissible crack widths in the case of prestressing steel in "corrosive" and "severely corrosive" environments are not given in Table 8.3.2 because flexural cracking can be avoided in prestressed concrete and that special attention to corrosion needs to be paid for prestressing steel. In general, the members in such environments should be designed avoiding the occurrence of cracks. However, in cases when cracking is allowed, an appropriate permissible crack width should be determined on the basis of investigations of environmental and loading conditions and the durability of the structure. In case that reinforcement nearest the surface is deformed bar in PRC structure, the limit value of crack width in Table 8.3.2 may be adopted.

8.3.3 Examination for flexural cracks

(1) Examination for flexural cracks should be made, by ensuring that the crack width under service conditions is not greater than the limit value of crack width.

(2) Examination for flexural cracks need not be carried out in cases when the tensile stress in concrete due to flexural moment and axial forces is less than flexural cracking

strength of concrete, f_{bck} .

[Commentary] (1) The basic approach to crack control from the viewpoint of resistance to steel corrosion is to control the crack width at the concrete surface at or below the critical crack width for steel corrosion determined by the environmental conditions and concrete cover. Flexural crack width for reinforced concrete and prestressed concrete may be calculated by the method described in Section 7.4.4 of Chapter 7.

ε'_{csd} in Eq.(7.4.4), which represents the effect of shrinkage and creep of concrete on crack width, should be determined with the consideration of shape of cross section of the member, environmental condition, magnitude of stress, and others. The value of ε'_{csd} may be set 150x10^{-6} in general cases and 100x10^{-6} for high strength concrete, considering the coherence with the past prediction equation, measured data in actual structures, the influences of cyclic drying and wetting on crack width and corrosion of reinforcement and others. It has been reported that because the crack width calculated by this method tends to be smaller than measured values obtained from existing structures, in order to make calculated crack width values closer to measured values, it is necessary to make the value of ε'_{csd} greater than 150×10^{-6}. If, however, the value of ε'_{csd} is increased on the basis of measured values, it is necessary to reconsider the limit values of crack width, too. This is why this section states that when evaluating the critical crack width for steel corrosion in accordance with Section 8.3.2, the value of ε'_{csd} in Eq. 7.4.4 may be assumed to be about 150×10^{-6} or so (about 100×10^{-6} for high-strength concrete) for calculation purposes.

If the structure under consideration is located in a normal environment where harmful substances such as chloride ions do not exist and the members of the structure are ones that are normally deemed appropriate, a steel stress study may be conducted in place of a crack width study described in this section. In such cases, verification may be made by ascertaining that the amount of increase in reinforcing steel stress σ_{se} and the amount of increase in prestressing steel stress σ_{pe} caused by sectional forces under permanent loads are smaller than the values shown in Table C8.3.1.

Table C8.3.1 Limiting values for stresses σ_{se} (N/mm^2) due to permanent load, in ordinary reinforcement and prestressing steel, when examination of crack width may be omitted

Type of reinforcement	Limiting values for the stress induced in steel (N/mm^2)
Deformed bars	120
Plain bars	100
Prestressing steel	100

Values in Table C8.3.1 have been determined considering the presence or absence of prestress application, type of reinforcement, thickness of concrete cover, etc. using normally used design values of stresses for railway and highway bridges, as references. If, however, the influence of variable loads is considered to be significantly greater than that of permanent loads, a study on crack width needs to be conducted.

(2) If flexural cracking does not occur, the influence of flexural cracking on steel corrosion does not need to be taken into consideration.

8.3.4 Examination for shear cracks

(1) For members subjected to shear forces, examination for shear cracks may not be carried out in cases when the design shear force, V_d, is smaller than 70% of the design shear capacity of concrete, V_{cd}, calculated using Eq.(9.2.5). In such cases, γ_b and γ_c should, in general, be taken as 1.0.

(2) In cases when the examination for shear crack is required from the viewpoint of reinforcement corrosion, it shall be carried out using an appropriate method.

[Commentary] (2) Because the mechanism of the occurrence and growth of shear cracks differs from that of flexural cracks, shear cracking needs to be studied separately by an appropriate method. Not many studies have been done, however, on shear cracking. In the verification for shear cracking, usually crack width is not directly checked. Instead, a common approach is to indirectly make sure that adverse effects of cracking do not occur by keeping shear reinforcement stress at or below the limit value. Since shear cracks may have a significant effect on the durability and the deterioration of watertightness and airtightness of structures, a careful examination shall be carried out, if necessary. It has been reported that shear cracks do not cause any problems in case of linear members, in which strains of stirrups are not greater than 1000×10^{-6} and the service load is about 0.5 – 0.7 times as large as the ultimate load. In general, if it is confirmed that the stress in shear reinforcement under the permanent loads is not greater than the values in Table C8.3.1, precise examinations may not be required. The stress in stirrups under the permanent loads may be estimated by the method shown in 7.4.3 in Chapter 7.

8.3.5 Examination for cracks due to torsion

(1) Examination for cracks due to torsion may not be carried out in cases when the design torsional moment, M_{td}, is smaller than 70% of design torsional moment capacity of a member without torsion reinforcement, M_{tud}, obtained using 9.2.3.2. In such cases, γ_b and γ_c should, in general, be taken as 1.0.

(2) When the examination for torsional cracks is required from the viewpoint of reinforcement corrosion, it shall be carried out using an appropriate method.

[Commentary] (1) At serviceability limit states, torsional cracks do not easily occur if the design torsional moment, M_{td}, is smaller than the design torsional moment capacity of a member without torsion reinforcements, M_{tud}, which is the torsional moment when torsional crack occurs. However, in order to take some safety margin, an examination of torsional crack may be omitted only when the design torsional moment is smaller than 70% of the design torsional moment capacity. In examinations of ultimate limit states, examinations for compatibility torsion are omitted, considering reduction of torsional stiffness due to cracks.

In general, verification with respect to compatibility torsion crack width does not need to be conducted mainly because (1) compatibility torsion moment is largely released because of initial cracking, (2) these cracks that occur under normal service conditions are relatively small, and crack width does not tend to increase and (3) at least a minimum amount of shear reinforcement is provided in each member so that the growth of torsion cracks can be prevented.

However, when U-shaped stirrups are used in beams, excessive cracking in the upper side of beams may occur. When height of a member with rectangular cross section is not greater than three times of its width, stirrups of closed-type is recommended.

When design torsional moment of a member subjected to equilibrium torsional moment is smaller than design torsional strength of a member without torsion reinforcements, examination may be omitted because cracks due to torsion do not occur.

(2) In accordance with a rationale similar to that for shear cracking verification, in the case torsion cracking, too, crack width is not directly checked. Instead, it is verified indirectly that no adverse effects of cracking can be prevented by keeping the stress in the reinforcement to control torsion crack opening at or below a limit value. Precise examination may not be required, if it is confirmed that the stress of torsion reinforcement due to permanent loads is smaller than the values in Table C.8.3.1. The stress of torsion reinforcement may be obtained from Eq.7.4.3.

8.3.6 Examination for reinforcement corrosion due to carbonation

(1) The required performance of concrete structures shall not be impaired by the carbonation of concrete.

(2) Verification for carbonation should be carried out by ensuring that

$$\gamma_i \frac{y_d}{y_{\lim}} \leq 1.0 \qquad \textbf{(8.3.1)}$$

where, γ_i **: factor representing the importance of the structure. In general, it may be taken as 1.0, but may be increased to 1.1 for important structures.**

$y_{\lim}$ **: critical carbonation depth of steel corrosion initiation. In general, it may be obtained from Eq.(8.3.2).**

$$y_{\lim} = c_d - c_k \qquad \textbf{(8.3.2)}$$

where, c_d **: design value of concrete cover used for durability verification (mm); calculated, taking into account construction error, from Eq. (8.3.3).**

$$c_d = c - \Delta c_e \qquad \textbf{(8.3.3)}$$

c **: concrete cover (mm).**

Δc_e **: construction error (mm).**

c_k **: remaining non-carbonated cover thickness (mm). This may be taken as 10mm for structures in a normal environment, and between 10 and 25 mm for structures located in chloride rich environments.**

y_d **: design value of carbonation depth, In general, it may be obtained from Eq.(8.3.4).**

$$y_d = \gamma_{cb} \cdot \alpha_d \sqrt{t} \qquad \textbf{(8.3.4)}$$

where, α_d **: design carbonation rate (**$mm/\sqrt{year}$**), which is given as**

$$= \alpha_k \cdot \beta_c \cdot \gamma_c$$

α_k	:	**characteristic value of carbonation rate (** $mm/\sqrt{year}$ **).**
t	:	**designed service life of structure for carbonation (year). Equation (8.3.3) should be used to evaluate the carbonation depth only for service lives less than 100 years.**
β_e	:	**factor indicating the degree of environmental influence; usually, the values shown in Section 8.2 may be used.**
γ_{cb}	:	**safety factor to account for the variation in the design value of carbonation depth. Normally, it may be taken as 1.15. In the case of high fluidity concrete, it may be taken as 1.1.**
γ_c	:	**factor to account for the material properties of concrete. In general, it may be taken as 1.0, but should be taken as 1.3 for upper portions of the structure. However, if there is no difference in the quality of concrete in structure and that of laboratory-cured specimens, the value of 1.0 may be adopted for the whole structure.**

[Commentary] (1) The pH value in concrete reduces due to the carbonation of carbon dioxide penetrating from atmosphere into concrete. If such a zone of reduced pH reaches the location of reinforcing bars in concrete, these bars become vulnerable to corrosion. Once the corrosion is initiated, the formation and deposition of expansive corrosion products on the bars may cause the formation of longitudinal cracks and the spalling of cover concrete, which accelerate Furthermore corrosion and result in a significant reduction in the cross-section of reinforcing bars. It must be ensured that the performance of structure shall not fall below the required level due to the corrosion of reinforcing bars. According to previous reports, factors affecting steel corrosion due to carbonation include not only the quality of concrete and environmental conditions but also construction-related factors such as inadequate concrete cover, honeycombing, cracking and inadequate curing. It is therefore important to perform careful construction control.

In non-reinforced concrete structures, when nominal steel bars are not installed, there is no risk of degradation of structural performance due to steel corrosion, so the carbonation verification is not required. When nominal steel bars are installed, depending on the purpose of the installation, this verification may be required. In such a case, the verification will be carried out in accordance with the principles laid down in Clause (2).

(2) To ensure that reinforcing bars corrosion on account of carbonation in concrete does not occur, is quite simple just by verifying that the carbonation depth does not exceed the depth of reinforcing bars corrosion onset.

It has been learned from laboratory tests and investigations on actual structures that the corrosion of steel may begin before the carbonation depth actually exceed the cover thickness (i.e. before the carbonation front reaches the location of the reinforcing bars). In many cases, a so-called "remaining carbonation depth", i.e., the remaining part of cover thickness still not neutralized, is used to discuss the onset of corrosion. There have been very few cases where corrosion develops to the extent that impairs the performance of a structure once the remaining carbonation depth remains larger than 10mm.

The concrete cover value used in Eq. (8.3.2) is a value determined by allowing for construction error. The concrete cover value used for verification is a value obtained by subtracting construction error as in Eq. (8.3.3) in view of the fact that steel corrosion due to carbonation may be caused by inadequate concrete cover, and it may be difficult to accurately control concrete cover during construction. The amount of shortage in concrete cover caused by carbonation varies depending on the level of construction control. According to study results obtained by comparing design values of concrete cover with concrete cover measurement results, the maximum amount of shortage from the design value of concrete cover is about 10% of the design value. The amount of construction error in concrete cover may be set by referring to the Design: Standards, Part 5, of this Specification.

However, when salt permeated into concrete, chloride ions entrapped in hydration products of cement could become free during the carbonation process. These ions migrate towards un-neutralized regions and possibly accelerate the initiation of corrosion. Therefore, in such cases, a remaining carbonation depth higher than that of normal environment is recommended, taking into account the existence of salt. However, since the ingress of chloride ions into concrete occurs under wet condition and carbonation proceeds relatively fast in dry condition, it is unlikely that both phenomena proceed with high speed at the same time. From these reasons, remaining carbonation depths should be taken to be from 10 to 25mm. Since the practical value of the remaining carbonation depth depends on environmental conditions (such as temperature, humidity or salt supply etc.), or quality of concrete, it is not clarified at this time. The upper limit of the remaining carbonation depth of 25mm is used when chloride ions are present in concrete under salty environment. The value 25mm is also recommended for safety reason when reliable data is not available to estimate the remaining carbonation depth of corrosion initiation. The remaining carbonation depth can be reduced to less than 25mm if the safety of concrete structures is fully confirmed by inspections or experiments in the same condition.

The “square-root” law (i.e. the depth of carbonation varies linearly with the square-root of time) is selected for the estimation of the carbonation depth because it is the most general approximation based on the data of the past studies.

In Eq. (8.3.4), the safety factor for considering the variation of the designed carbonation depth, is determined by taking into consideration the accuracy of the equation itself, the assumptions that carbonation proceeds uniformly throughout the cross-section and that the effect of cracks can be neglected if the crack width is smaller than that stipulated in Section 8.3.2. In the case of high fluidity concrete, because its segregation resistance is high and the homogeneity is ensured, the safety factor for considering the variation of designed carbonation depth can be eliminated.

8.3.7 Examination for chloride attack

(1) Performance of a structure shall not be impaired by corrosion of reinforcement due to ingress of chloride ions.

(2) Examination for corrosion of reinforcement due to ingress of chloride ions may be carried out by ensuring that the criteria in Eq. (7.4.4) is satisfied:

$$\gamma_i \frac{C_d}{C_{\lim}} \leq 1.0 \tag{8.3.5}$$

where, γ_i **: structure factor. It may be taken as 1.0 for ordinary structures and 1.1 for important structures.**

$C_{\lim}$ **: threshold value of chloride concentration for onset of reinforcement corrosion. This may be determined by referring to measurement results or test results for similar structures. If those results are not relied upon, 1.2 kg/m^3 may be used. It may be preferably set at a value less than 1.2 kg/m^3 for structures subjected to freezing and thawing action.**

C_d **: design value of chloride concentration at location of reinforcement. It may, in general, be estimated using Eq.(8.3.6).**

$$C_d = \gamma_{cl} \cdot C_o \left\{ 1 - erf\left(\frac{0.1 \cdot c}{2\sqrt{D_d \cdot t}} \right) \right\} \tag{8.3.6}$$

where, C_o **: concentration of chloride ions at the concrete surface (kg/m^3). In general, it may be determined according to Section 8.2.**

γ_{cl} **: safety factor to take into account the scatter in** C_d**, the design value of chloride concentration at location of reinforcement. In general, it may be taken to be 1.3.**

c_d **: design value of concrete cover used for durability verification (mm); calculated, taking into account construction error, from Eq. (8.3.7).**

$$c_d = c - \Delta c_e \tag{8.3.7}$$

c **: concrete cover (mm).**

Δc_e **: construction error (mm)**

t **: design lifetime for chloride attack (year). In estimating the concentration of chloride ions at location of reinforcement using Eq.(8.3.6), an upper limit of 100 years for the design lifetime should be used.**

D_d **: design value of diffusion coefficient of chloride ions (cm^2/year) in concrete. In general, it may be estimated using Eq.(8.3.8).**

$$D_d = \gamma_c D_k + \left(\frac{w}{l}\right)\left(\frac{w}{w_a}\right)^2 D_o \qquad \textbf{(8.3.8)}$$

where, γ_c **: material factor for concrete. In general, it may taken to be 1.0. However, it should be taken to be 1.3 for the top face of a structure. In cases when the quality of concrete in a structure is the same as that of specimens cured under standard conditions, the material factor for all parts of the structure may be taken to be 1.0.**

D_k **: characteristic value of diffusion coefficient of chloride ions in concrete (cm^2/year).**

D_o **: a constant to represent the effect of cracks on transport of chloride ions in concrete (cm^2/year). It should be considered only in cases when flexural cracks are allowed. In general, it may taken to be 200 cm^2/year.**

w **: crack width (mm), calculated in accordance with 8.3.3.**

w_a **: limit value of crack width (mm), determined in accordance with 8.3.2.**

w/l **: ratio between crack width and crack spacing. In general, it may be evaluated using Eq.(8.3.9).**

$$\frac{w}{l} = 3\left(\frac{\sigma_{se}}{E_s}\left(or\ \frac{\sigma_{pe}}{E_s}\right) + \varepsilon'_{csd}\right) \qquad \textbf{(8.3.9)}$$

where, the definitions of σ_{se}**,** σ_{pe} **and** ε'_{csd} **are the same as in Eq.(8.2.1) and their values may be taken as those used in the computation of crack width.**

erf(s) is an error function defined as: $erf(s) = \dfrac{2}{\pi^{1/2}}\int_0^s e^{-\eta^2} d\eta$

(3) In cases it is difficult to satisfy the requirement prescribed in (2), reinforcement protected against rusting is preferably used or the surface of the concrete should be coated, etc. to protect the reinforcement from corrosion. In such cases, the effectiveness of these techniques should be evaluated using appropriate methods, keeping in mind the maintenance planning for the structure.

(4) In environments free from the effect of external chlorides, it may be assumed that provided the total chloride ion content at the stage of mixing is less than 0.30 kg/m^3, the performance of structures is not impaired due to presence of chloride ions. In the case of PC strands, which are more vulnerable to stress corrosion, this value is preferably reduced.

(5) For a concrete structure where the use of antifreeze agents is expected, particular attention shall be paid to the durability of concrete, and reliable waterproofing and drainage works should be carried out during construction in order to prevent chloride ion penetration into the concrete.

[Commentary] (1) Even when the corrosion develops in reinforcing bars, the structure is considered serviceable if all the required performances are not impaired by the corrosion. In other words, the performance of structure is considered to be satisfactory even when the corrosion has started so long as the corrosion-induced longitudinal cracks are not formed along the reinforcing bars.

In the case of plain concrete, there is no potential degradation of structural performance due to chloride-induced corrosion if nominal reinforcing bars are not installed, so the verification for chloride-induced corrosion is not necessary in this case. When nominal reinforcing bars are used, the verification for chloride-induced corrosion as stipulated in Clause (2) may be required in some cases depending upon the purpose of using nominal reinforcing bars.

(2) Regarding the verification of the performance of structure under chloride attack, the setting of the condition that chloride-induced corrosion of reinforcing bars does not occur during the service life of structure is relatively easy to understand from safety viewpoint. Therefore, it is important to verify that the concentration of chloride ion at the location of the reinforcing bars is below the critical concentration that could initiate the corrosion of reinforcing bars. However, if possible, the verification could also be carried out by checking the onset of corrosion-induced crack in concrete as a limit state, taking into account the required performances and the importance of the structure. It should be mentioned that the ‘chloride concentration’ mentioned in this specification refers to the total chloride content present per unit volume of concrete and not to the concentration of actual chloride ions of the liquid phase (or pore solution) in concrete.

The critical chloride concentration in the vicinity of reinforcing bars that could initiate the corrosion in reinforcing bars has been reported to be about 0.3~1.2 kg/m^3 of concrete. For example, values of about 0.3~0.6 kg/m^3 have been reported from accelerated tests carried out with chlorides added into fresh concrete, and from exposure tests carried out at the field, values of 1.2~2.4 kg/m^3 have been reported. Differences in the critical chloride concentrations reported in those accelerated and exposure tests, etc. can be attributed to factors such as differences in the water-cement ratio of the concrete, thickness of cover concrete, etc. High temperatures used in accelerated tests can also be an influential factor. Thus, since the critical chloride ion concentration for steel corrosion varies depending on various conditions, it is desirable, when checking on individual structures, that the critical concentration be determined by referring to measurement results or test results obtained under conditions, such as materials used and environmental conditions, similar to those for the structure under consideration. If those results are not relied upon, verification may be done by using a conservative limit value of 1.2 kg/m^3.

Mathematical formulations based on the diffusion theory are most commonly used to simulate the penetration of chloride ions into concrete. Since the diffusion is a process involving the movement of ions with the driving force due to the difference in the ion concentration of the liquid, it is desirable that the formulation used is based on the difference in the chloride ion concentration in the pore solution. The chloride ions in concrete are either entrapped or absorbed on the surface of hydration products of cement or constituent components of cement. Since these states of chloride within concrete also depend on chemical factors such as changes in the pH on account of carbonation, etc., it is desirable that a formulation that takes all these factors into account in a comprehensive manner.

The use of Eq. (8.3.6) in which the Fick’s law of diffusion is used to simulate the diffusion of chloride ions at the position of reinforcing bar corresponding to the number of years in service has been considered satisfactory. This equation is based on the concept that chloride ions diffuse equally toward steel. When bending crack is generated on concrete cover, the distribution of chloride ion concentration depends on the distance between crack and steel. However, when crack width is very narrow, the variation of concentration of chloride ion may trend to be small.

In the case of well-compacted concrete, the inconsistent distribution of chloride ion may occur due to the influence of crack, but steel is well protected in uncracked parts against the corrosion. From these reasons, the limit for the onset of steel corrosion can be estimated by averaging the influence of crack based on the concept that chloride ions diffuse equally toward steel. Therefore, when the bending crack width is narrower than the allowable one defined in Section 8.3.2, the diffusion coefficient can be estimated using Eq. (8.3.8), taking into consideration the influences of concrete quality and crack. In this estimation, the concentration of chloride ions at the position of reinforcing bars is evaluated using Eq. (8.3.6).

The Eq. (8.3.8) is to estimate the average diffusion coefficient of chloride ions in cover concrete considering the quality of non-cracked portions of concrete and the influence of crack opening. The term "ratio of crack width to crack interval", w/l, is introduced in order to express the influence of crack averagely. This averaging method is effective when crack width is smaller than the allowable crack. The resistance to the ingress of chloride ions is greatly affected if the crack width is large, so the term $(w/w_a)^2$ of the ratio between the crack width and the limit value of crack width is introduced to consider that influence.

In the case of early-time cracks such as thermal crack or shrinkage crack, the verification of steel corrosion due to the ingress of chloride ions must be carried out properly by measuring the crack width and interval. However, it may be difficult to measure accurately the crack width and interval. In these cases, the following formula can be used when the crack width is smaller than the limit value of crack width stipulated in Section 8.3.2.

$$D_d = D_\gamma \cdot \gamma_c \cdot \beta_{cl} \qquad \text{(C2.3.1)}$$

Where, β_{cl} is a factor representing the influence of early-time crack. It may be taken to be 1.5.

The safety factor to account for the variation in the design value of chloride concentration in the vicinity of reinforcing bars in Eq. (8.3.6) should take into account the accuracy of the prediction methodology adopted as well as localized effects of chloride ions on the corrosion of reinforcing bars. In the case of high fluidity concrete, the safety factor, which considers variations in the Design value, can be eliminated because high fluidity concrete has high segregation resistance and the assumed condition of homogeneity is highly satisfied. As far as the determination of the surface chloride concentration is concerned, it is important that the amount of airborne salt and the environmental condition are appropriately taken into account.

(3) If the environment of the structure is harsh or if corrosion cannot be allowed, it may be difficult to meet verification criteria even if concrete with a small diffusion coefficient is used and concrete cover is increased. In such cases, it may be economical to use corrosion resistant materials and methods, such as epoxy coated steel as reinforcing bars, durable coating materials for concrete surface, which could reduce the ingress of air-borne chloride ions, electro-chemical treatment (cathodic protection), to protect reinforcing bars against corrosion. When these methods are used, it is recommended that the following guidelines be followed: Recommendations for Design and Construction Using Electrochemical Corrosion Control Method (Concrete Library 107), Recommendations for Design and Construction of Reinforced Concrete Structures Using Epoxy-Coated Reinforcing Steel Bars (Concrete Library 112) and the Recommendations for Concrete Repair and Surface Protection of Concrete Structures (Concrete Library 119). The abovementioned recommendations describe a recommended verification method for steel corrosion due to chloride ion penetration occurring in cases where epoxy-coated reinforcing bars are used. Concrete calculation methods, however, for performance verification that can be used to evaluate the durability-improving effects of different construction methods have not yet been specified.

It is important that the corrosion prevention method is determined by considering the structural characteristics, environmental condition and the life cycle cost of concrete structure.

(4) Basically speaking, so long as the chloride ion concentration in the vicinity the reinforcing bars remains below a critical level for the onset of steel corrosion, during the designed service life of structure, it can be considered that the corrosion of reinforcing bars does not impair the performance of concrete structure. Therefore, even in cases where concrete is likely to contain chloride ions at the time of mixing, no problem is likely to arise provided the chloride level does not exceed the above mentioned critical level during the designed service life of the structure. However, it should be borne in mind that chloride ions present in concrete at the time of mixing may not be mixed uniformly within the matrix on the account of causes such as bleeding, etc. Furthermore, even after concrete has hardened, chloride ions could migrate and concentrate at some places due to several reasons. Taking the above into consideration, it is not advisable to use the same critical level of chloride ions (that cause the onset of reinforcing bars corrosion) for the permissible one at the time of mixing.

For this reason, to ensure that any degradation of the structure due to the steel corrosion is kept below an allowable level, the permissible level of chloride ions in concrete at the time of mixing has been set to be lower than 0.30 kg/m^3, the same as prescribed in the previous version of this document, and is considered to be realistically achievable. However, in cases that there is no likelihood of the ingress of chloride ions during the service-life of the structure, the water content and the water-cement ratio is set as low as possible, proper construction practices are adopted, segregation is avoided and the placed concrete is dense enough to prohibit the free movement of chloride ions in hardened concrete, the permissible level of chloride ions in concrete at the time of mixing can be raised to 0.6 kg/m^3.

(5) The use of deicing agent for concrete structures has become a popular practice in order to ensure the safety of road traffic in winter. However, the use of deicing agent facilitates the chloride-induced corrosion of steel. Structural damages have been observed in North-America. In recent years, damages due to the excess of critical chloride concentration for steel corrosion because of the widespread use of deicing agent have also been reported in Japan and this problem may spread in the future. Basically, the influence of antifreeze agents can also be verified by the method described in Item (2). Since, however, it is difficult to determine chloride ion concentration at the concrete surface, it is more realistic to take measures to prevent the water containing chloride ions from reaching the concrete such as adopting a type of structure without expansion devices as well as carrying out appropriate waterproofing and drainage works or to use antifreeze agents that do not contain chloride ions. Appropriate treatments are also expected for concrete structures currently in service if deicing agent is used.

8.4 Verification Related to Concrete Deterioration

8.4.1 Verification for freezing-thawing action

(1) The required performance of concrete structure shall not be impaired by freezing-thawing action.

(2) Verification related to freezing damage may be done by ascertaining that in the case where the concrete in a structure undergoes deterioration, the value obtained by multiplying the ratio between the minimum limit value E_{min} of the relative dynamic modulus of elasticity in a freeze–thaw test and its design value E_d by the structure factor γ_i is 1.0 or smaller.

$$\gamma_i \frac{E_{min}}{E_d} \leq 1.0 \qquad \textbf{(8.4.1)}$$

where

γ_i **: a factor representing the importance of the structure. In general, it may be taken as 1.0, but may be increased to 1.1 for important structures.**

E_d **: design value of relative dynamic modulus of elasticity in freeze–thaw test (**$= E_k/\gamma_c$**)**

E_k **: characteristic value of relative dynamic modulus of elasticity in freeze–thaw test.**

γ_c **: material factor. In general, it may be set at 1.0, but should be taken as 1.3 for upper positions of the structure. However, if there is no difference in the quality of concrete in the structure (*in situ*) and that of laboratory-cured specimens, this value may be set at 1.0 for all positions.**

$E_{\min}$ **: critical minimum value of relative dynamic modulus of elasticity to ensure satisfactory performance of the structure under freezing-thawing action. In general, it may be obtained from Table 8.4.1.**

Table 8.4.1 Minimum limit value E_{min} (%) of relative dynamic modulus of elasticity in freeze–thaw test to meet freeze–thaw resistance performance requirements for a concrete structure

Climate / Section / Exposure of structure	Severe weather conditions or frequent freezing-thawing action		Not so severe weather conditions, atmospheric temperature rarely drop to below 0°C	
	Thin 2)	General	Thin 2)	General
(1) Immersed in water or often saturated with water 1)	85	70	85	60
(2) Not covered in item (1) above and subjected to normal exposure conditions	70	60	70	60

1) Structures close to the water surface or in contact with water such as waterways, water-tanks, abutments of bridge, bridge piers, retaining walls, tunnel linings, etc. Besides, structures such as slabs, beams etc not close to the water surface but may be exposed to snow, water flow, spray, etc., also belong to this category.

2) Members with thickness less than 20cm may be considered 'thin'.

[Commentary] (1) There has been almost no quantitative information available to relate any change in the performance of structure to the manifestations of damage on account of cyclic exposure to freezing and thawing action, such as pop-outs, scaling, and formation of micro cracks in concrete. Therefore, at present it is difficult to establish criteria for the required performance of a structure subjected to cyclic freezing and thawing, and fix a limiting or maximum permissible level of 'allowable' deterioration, based on such criteria. The recommended method at present is to regard the level of deterioration at which a certain degree of concrete deterioration occurs because of freeze–thaw cycles but the structure does not lose its functionality as the critical state of performance of the structure under freeze–thaw cycles and use the verification of the performance of the concrete under freeze–thaw cycles in place of the verification of the performance of the structure under freeze–thaw cycles.

Furthermore, for concrete not likely to be exposed to cyclic freezing and thawing actions, the verification for freezing-thawing action is not necessary.

(2) The freeze–thaw resistance of concrete is difficult to evaluate accurately because it is affected by many factors such as the quality of concrete, lowest temperature, the number of freeze–thaw cycles and the degree of saturation. In many cases, freeze–thaw resistance requirements can be met by giving an appropriate level of freeze–thaw resistance to the concrete. On the other hand, the relationship between the results of accelerated tests and the extent of frost damage of actual structures, is somewhat better understood, thanks to past research works and field data. For this reason, it is recommended that parameters such as the relative dynamic modulus of elasticity and the weight loss in accelerated tests are used as indices for the verification of concrete resistance against frost damage. Accelerated freezing and thawing test must be carried out in accordance with the method specified in JIS A 1148 (A method): “The Freeze-thaw Test Method of Concrete (Freeze-thaw test method of concrete underwater)”. However, when the conditions of the freezing-thawing action are especially severe and the designed service life of the structure is relatively long, appropriate changes of testing conditions prescribed in JIS A 1148 (A method) should be made to bring the test closer to the actual conditions that the structure is likely to be subjected to. When a new structure is built in the same conditions with those of previously built structures, if the latter is proved to be strong against frost damage, the verification of frost resistance of the newly built structure can be omitted.

In the case of an ordinary structure that is not required to have a very high level of durability, freeze–thaw resistance verification may be omitted if the characteristic value of the relative dynamic modulus of elasticity in the freeze–thaw test of concrete is set at or above 90%. In the caser of a structure that is required to have a very high degree of durability such as the ability to maintain the initial structural soundness throughout the service life of the structure without affecting its appearance, it is necessary to prevent a decrease in the relative dynamic modulus of elasticity of test specimens or in the mass of specimens in freeze–thaw tests. In such cases, the relative dynamic modulus of elasticity may be kept at a level higher than 95%.

8.4.2 Verification related to chemical attack

(1) The required performance of a structure shall not be lost because of chemical attack.

(2) Verification related to chemical attack may be done by ascertaining that the value obtained by multiplying the ratio of the design value of chemical penetration depth (y_{ced}) to concrete cover c_d by the structure factor γ_i is 1.0 or smaller. If the concrete meets the chemical corrosion resistance requirements, it may be assumed that the required performance of the structure is not lost because of chemical attack, and this verification does not need to be done.

$$\gamma_i \frac{y_{ced}}{c_d} \leq 1.0 \qquad \textbf{(8.4.2)}$$

where

y_{ced}: **design value of chemical penetration depth (**$= \gamma_c y_{ce}$**)**

y_{ce} **: characteristic value of chemical penetration depth**

γ_c **: material factor for concrete, which may be assumed to be 1.0 but should be assumed to be 1.3 for the upper surface sections of a structure and may be assumed to be 1.0 for all parts of a structure if there is no difference in quality between the concrete in the structure and standard-cured specimens**

c_d **: design value of concrete cover used for durability verification (mm); to be calculated from Eq. (8.4.3).**

$$c_d = c - \Delta c_e \qquad \textbf{(8.4.3)}$$

c **:concrete cover (mm).**

Δc_e **:construction error (mm).**

(3) If chemical attack is severe, it is generally recommended that measures to control chemical attack such as covering concrete surfaces and using corrosion-proofed reinforcing materials be taken. In such cases, verification may be omitted if the control measures are evaluated by an appropriate method in accordance with the Maintenance section of this Specification.

[Commentary] (1) Chemically induced deterioration includes the dissolution and deterioration of concrete caused by contact with a corrosive substance, and cracking of concrete and spalling of cover concrete due to expansion that occur when a corrosive substance that penetrates concrete reacts with cement ingredients or steel. Current knowledge, however, is not necessarily good enough for quantitative evaluation the influence of concrete deterioration due to contact with or penetration of corrosive substances on the functional decline of structures. A realistic approach at present, therefore, is to define a limit state of a structure in terms of whether there is any visible deterioration of concrete due to contact with or penetration of corrosive substances or whether the influence of such deterioration reaches the steel location, taking into consideration such factors as the required performance, type of structure, degree of importance and maintainability of the structure and the severity of environmental conditions.

There is also deterioration due to biochemical activity besides deterioration due to chemical attack, but the durability of a structure against such deterioration may be verified in accordance with the method described in this section.

(2) It is difficult to evaluate the resistance of concrete to environmental deterioration factors of different types and degrees by a standard test method. When conducting tests for environmental deterioration factors of different types and intensities, therefore, it is necessary to determine different limit values associated with chemical corrosion resistance for different tests.

In cases where a structure is subjected to the deterioration activity of sulfates, verification may be done by ascertaining that the specified penetration depth will not be reached during the service life of the structure by using an appropriately determined characteristic value of sulfur penetration depth.

In a highly corrosive environment such as a hot spring or acid river where the deterioration of concrete cannot be controlled completely, a more reliable method of verification is to verify the performance of concrete specimens actually exposed to the environment. In such evaluation, the rate of corrosion of concrete is determined from the exposure period, and verification is done by ascertaining that concrete deterioration does not reach the critical depth for the structure during its service life. Deterioration in an environment where corrosive activity is severe and microorganisms such as sulfur-oxidizing bacteria are involved in corrosion such as a sewer environment occurs as the hydrogen sulfide gas generated by sewage sludge is oxidized by sulfur-oxidizing bacteria, and the sulfuric acid thus generated corrodes concrete. Consequently, the rate of corrosion of concrete varies depending on the type of sulfur-oxidizing bacteria involved and their habitat conditions. An appropriate method of verification, therefore, is to ascertain that deterioration will not reach the locations of steel in the structure during its service life by conducting exposure tests designed specifically for the deterioration situation for each facility and determining the rate of deterioration from the exposure period.

(3) If chemical corrosion is very severe as in a sewer or hot spring environment, it is generally difficult to meet chemical resistance requirements by means of cover concrete and the resistance of concrete alone. It can be said that sewerage facilities and structures in hot spring areas are in such environments. In such cases, it is often realistic and rational to take control measures such as coasting the concrete surface for protection from chemical attack and using corrosion-proofed reinforcing materials. If such control measures are taken, verification may be omitted by conducting exposure tests after actually taking control measures to verify the resistance to chemical attack.

CHAPTER 9 VERIFICATION OF STRUCTURAL SAFETY

9.1 General

(1) It shall be verified that the concrete structure meets the required safety performance during its design life.

(2) Safety of a structure shall be verified by confirming that (a) the ultimate limit state for failure of cross section for any of the members, and, (b) the ultimate limit state for rigid body stability for the structure, is not reached.

(3) Examination of the ultimate limit state for failure of cross section shall be carried out by confirming that Equation 9.1.1 is satisfied, i.e. the value obtained by multiplying the ratio of design member force (S_d) to design capacity of member cross section (R_d) by the structure factor (γ_i) is not greater than 1.0.

$$\gamma_i S_d / R_d \leq 1.0 \quad \textbf{(9.1.1)}$$

(i) The design capacity of cross section R_d shall be obtained from Eq. 9.1.2, by dividing computed member capacity (considering its cross section, design strength of materials, f_d, etc.), R (f_d), by the member factor, γ_b.

$$R_d = R(f_d)/\gamma_b \quad \textbf{(9.1.2)}$$

Generally, γ_b defined in Section 9.2 and below may be used.

(ii) The design member force S_d shall be obtained from Eq. 9.1.3 as the sum of member forces S, computed using the design loads, F_d, multiplied by a structural analysis factor γ_a.

$$S_d = \sum \gamma_a S(F_d) \quad \textbf{(9.1.3)}$$

(4) Examination of the ultimate limit state for fatigue failure shall be carried out in accordance with the followings:

(i) It shall be confirmed, that Eq. 9.1.4 is satisfied, i.e. the ratio of design variable stress, σ_{rd}, to design fatigue strength, f_{rd}, divided by the member factor, γ_b, multiplied by the structure factor, γ_i, is not greater than 1.0.

$$\gamma_i \sigma_{rd} /(f_{rd} / \gamma_b) \leq 1.0 \quad \textbf{(9.1.4)}$$

where f_{rd} is the characteristic value of fatigue strength for materials, f_{rk}, divided by the material factor, γ_m.

(ii) As an alternative to the above, examination for safety in fatigue can also be carried out by ensuring that Eq. 9.1.5 is satisfied.

$$\gamma_i S_{rd} / R_{rd} \leq 1.0 \quad \textbf{(9.1.5)}$$

where S_{rd}, is the fatigue variable member force, $S_r(F_{rd})$, obtained by multiplying the

design variable load, F_{rd}, by the structural analysis factor, γ_a; the design fatigue capacity of the member, R_{rd}, is the fatigue capacity of the member, $R_r(f_{rd})$, obtained by dividing the design fatigue strength of material, f_{rd}, by the member factor, γ_b.

(iii) The member factor, γ_b, may be taken to be between 1.0 and 1.1.

(5) Examination of ultimate limit states for displacement, deformation, formation of collapse mechanism and others, and, examination of the ultimate limit state of a member or structure using non-linear analysis without considering member forces, shall be carried out using methods whose applicability to the structure and accuracy have been previously established.

[Commentary] (1) and (2) This chapter describes standard methods for verifying the safety of structures under static loads without consideration of material deteriorations through design life, in cases where the provisions of Chapter 8, "Verification of Durability," Chapter 12, "Verification Related to Initial Cracking", and the constructability requirements described in the Standard Specifications for Concrete Structures "Materials and Construction", are satisfied. Further, safety against the effect of earthquakes should be verified in accordance with provisions of Chapter 11, "Verification of Earthquake Resistance."

A structure may be considered to be safe, i.e., its safety is maintained, as long as it or its members do not fail. In the case of a statically highly indeterminate structure, its safety may not be immediately lost even when some of the members reach the ultimate limit state and become incapable of carrying load. In cases where partial failure of members is permitted but the overall safety of the structure still needs to be maintained even after partial failure of some of the members, the nonlinear and the post-failure behavior of members should be appropriately taken into consideration at the time of safety verification as in seismic performance verification. In general, however, even if structures are ascertained to meet safety performance requirements under static loading beyond the limit state for failure of cross sections of members, the actual economic benefits are small. Also, verification of safety after failure of cross sections requires the use of larger factors of safety. The present specification, therefore, uses a standard approach: for the purposes of safety verification, it is simply required to verify that none of the members reaches the ultimate limit state. All units including the strength of materials used in verification methods described in this chapter are as described in Chapter 5, "Design Values for Materials."

(3) When a member is subjected to a combination of loads such as flexural moment and axial load, verification of safety against failure of member cross section shall be carried out by comparing the appropriate design member forces with design capacity of the member cross section, taking into account of the action of combined loads.

(i) The member factor (γ_b) is described in Section 9.2 and subsequent sections. When an empirical formula is used, calculated values of the load carrying capacity of the member are average values (see Section 4.4). For example, design equations for shear capacity of concrete members without shear reinforcement are in many cases based on experimental results obtained from testing small specimens. In view of considerable scale effects, the member factor may be taken as 1.3 even when precise empirical equations are used. If it is verified that an empirical equation is applicable to structures in realistic dimensions, the member factor may be taken as 1.15. Computation for torsional capacity of members without shear reinforcement is made using an elastic theory. According to experimental results of small-scale specimens, this formulation gives a conservative estimate of the torsional capacity of the member. However, the torsional failure of members without shear reinforcement occurs in tension, and in this case the ultimate strength is easily affected by non-structural cracks. Considering factors including the scale-effect, etc. the member factor may be taken as 1.3 as in the case of calculating the shear capacity as given above.

Computation for flexural and shear capacity of members with shear reinforcement, which yields according to the truss type mechanism is made using well-documented theoretical approaches. The present specification recommends use of a member factor of 1.1 to take into account variations in the geometry of members.

(4) Fatigue capacity of a member depends on the fatigue strength of the concrete and the reinforcement, which make up the member. If capacity of cross section of a member is proportional to the fatigue strength of the materials, same results will be obtained in the examination of safety for fatigue by either equation. However, since there are some cases that they are not proportional with each other, safety for fatigue shall be examined by using fatigue strength of materials in principle.

(5) In the examination of the ultimate limit state for deformation, it is examined whether plastic deformations, cracking, creep and others cause large deformations in the structure or its members to the extent that they lose their shapes or dimensions. Any loss of structural stability caused by large deformations should also be examined at this stage.

Examination of the ultimate limit state for mechanism formation is carried out by examining whether or not moment redistribution through plastic hinges causes formation of a collapse mechanism in statically indeterminate structures, on the basis of the rotational capacity for plastic hinges.

In cases where examination of the limit state for structural failure is carried out by non-linear analysis, the performance verification may be carried out using indices other than member force. Such indices include the average in-plane principal strain in the case of a shell structure subjected to shear, and, the average curvature and the rotation angle at the plastic hinge in the case of a member subjected to flexure.

9.2 Examination of safety for failure of cross section

9.2.1 Flexural moment and axial forces

9.2.1.1 Design capacity of member cross section

(1) The upper limit for axial capacity N'_{oud} in compression for members subjected to axial compressive force shall be calculated by Eq. (9.2.1) if ties are used, and by Eq. (9.2.1) or Eq. (9.2.2), whichever is greater, if spiral reinforcement is used.

$$N'_{oud} = \left(k_1 f'_{cd} A_c + f'_{yd} A_{st}\right)/\gamma_b \tag{9.2.1}$$

$$N'_{oud} = \left(k_1 f'_{cd} A_e + f'_{yd} A_{st} + 2.5 f_{pyd} A_{spe}\right)/\gamma_b \tag{9.2.2}$$

where, A_c **: area of concrete section**

A_e **: area of concrete section surrounded by spiral reinforcement**

A_{st} **: total area of axial reinforcement**

A_{spe} **: converted area of spiral reinforcement (=$\pi d_{sp} A_{sp}/s$)**

d_{sp} **: diameter of cross section surrounded by spiral reinforcement**

s : **pitch of spiral reinforcement**

A_{sp} : **area of spiral reinforcement**

f'_{cd} : **design compressive strength of concrete**

f'_{yd} : **design compressive yield strength of axial reinforcement**

f_{pyd} : **design tensile yield strength of spiral reinforcement**

k_1 : **strength reduction factor (**$=1-0.003 f'_{ck} \leq 0.85$**,**

where, f'_{ck}**: characteristic value for strength of concrete (N/mm²))**

γ_b : **member factor, which may generally be taken as 1.3**

(2) When the member is subjected to a flexure or combined flexure and axial loads, the member factor γ_b **may be taken as 1.10, and the design capacity may be computed in directions corresponding with member forces, keeping in mind the following assumptions:**

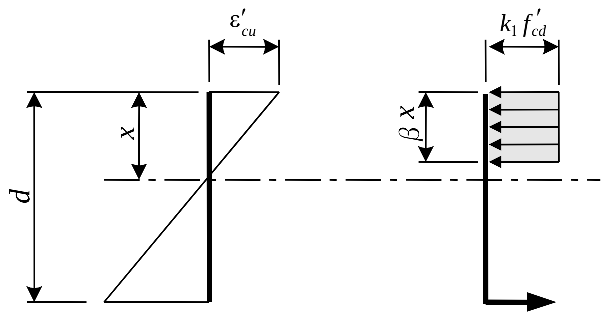
(i) Strain is proportional to the distance from neutral axis.

(ii) Tensile stress of concrete is neglected.

(iii) Stress-strain relationship of concrete follows that given in Fig. 5.2.1.

(iv) Stress-strain relationship of steel follows that given in Fig. 5.3.1.

(3) Except when the compressive strain is distributed over entire cross-section, the compressive stress distribution of concrete may be assumed as an equivalent stress block as shown in Fig. 9.2.1.



$$k_1 = 1-0.003 f'_{ck} \leq 0.85$$

$$\varepsilon'_{cu} = \frac{155 - f'_{ck}}{30000} \leq 0.0035$$

where $f'_{ck} \leq 80$ N/mm²

$$\beta = 0.52 + 80\varepsilon'_{cu}$$

Fig. 9.2.1 Equivalent stress block

(4) The design capacity of members subjected to biaxial flexural moments and axial loads simultaneously may be determined by the assumptions in (2) above.

(5) The effect of an axial force may be considered 'small' in cases when $e/h \geq 10$**, where** h **is the height of the cross section, and eccentricity** e **is the ratio of design flexural moment** M_d **to design axial compressive force** N'_d**. In such cases the design capacity of a member may be calculated as for a member subjected to flexure alone.**

[Commentary] (1) The coefficient k_1 has been conventionally taken as 0.85. In the case of high-strength concrete members, the design axial capacity of members may be smaller than that calculate on the basis of the strength of conventional cylindrical test specimens, with the difference growing larger as the strength of concrete increases. To take this effect into k_1, should be treated as a function of the compressive strength, in the case of high strength concrete.

A small increase in the flexural moment caused by some misalignment of members at the construction stage, for example, can significantly reduce the ultimate resistance of members subjected to axial compression with a small M_d / N'_d. To account for such cases, the present specification places an upper limit on the design load carrying capacity and suggests use of a higher member factor (=1.3).

(2) A general relationship between the design axial capacity and design flexural capacity of members subjected to combined axial load and flexural moment is described in Fig. C9.2.1. Therefore, safety under combined axial load and flexural moment should be examined by confirming that the point ($\gamma_i M_d , \gamma_i N'_d$) is located inside of ($M_{ud} , N'_{ud}$) curve, that is, at the side of the origin as shown in Fig. C9.2.1.

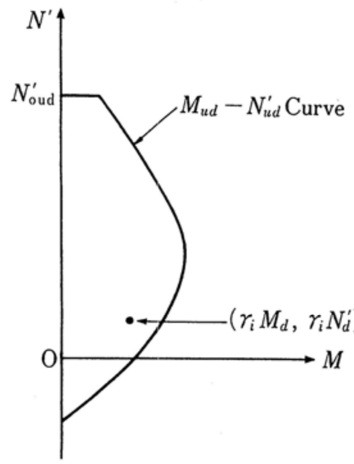


Fig. C9.2.1 Relationship between axial capacity and flexural capacity

Flexural moment from structural analysis is generally obtained using the centroid of member cross section. The flexural capacity should also be calculated using the same centroidal axis.

Assumption (i) addresses the strain distribution across a member cross-section, and, (ii) and (iii), deal with stress distribution in concrete.

Assumption (iv) is related to the stress-strain relationship of steel. Considering that, (a) the capacity of a member is hardly affected by the ultimate strain of reinforcing steel, and, (b) by specifying this value computation becomes more complicated, the Specification does not specifically define the value of the ultimate strain of steel.

Lateral confinement of concrete in beams and columns by spiral or tie reinforcement significantly improves the deformability of the member, and can be used to an advantage for moment redistribution and in seismic design. The effectiveness of the lateral reinforcement is, however, related to several factors, including properties of the materials used, shapes, interval and volume ratio of steel and the gradient and rate of strains of confined concrete, and thus, generalized design equations are yet not available. The load-deformation relationship may be determined using appropriate experimental results, if possible.

(3) The equivalent stress block shown in Fig. 9.2.1 has been determined on the basis of the stress-strain curve shown in Fig. 5.2.1. An exact correspondence between the magnitude and location of resultant in the equivalent stress block and those determined from the actual stress-strain relationship can be obtained only by varying the stresses in the stress block using the ultimate strain of concrete and the stress-strain relationship. For the sake of convenience, however, it has been assumed in the present specification that stress in the stress block is the same as the maximum stress, as given in the stress-strain relationship. Also, the height of the stress block is determined in a manner that the resultant lever arm is such that the axial force and flexural moment are roughly in agreement.

(4) The capacity of members subjected to biaxial flexure and axial loads simultaneously should be calculated in a manner similar to the members subjected to uniaxial flexure and axial loads. In such cases, all reinforcement used in the member may be taken into account. In the analysis, careful attention should be paid to the arrangement of reinforcement in the member, and it should be ensured that the direction of the neutral axis does not coincide with the direction of the principal axes of stresses, except in the case of uniaxial flexure.

Figure C9.2.2 shows an example of a rectangular cross section of a member under the action of a concentrated load acting at a location other than the centroid and causing biaxial flexure. The load-carrying capacity of such a member may be calculated through an iterative procedure to be repeated till the eccentricity in the vertical and horizontal directions, e_x and e_y, calculated by assuming the location of the neutral axis, y_o, and the angle of the neutral axis, θ, to the specified value.

In general, exact calculations of changes in concrete and steel stresses due to changes in y_o and θ is very tedious. Stresses, therefore, may be calculated from strains at the location of the centroid by dividing the cross section into small elements and assuming that stress in each element is constant.

(5) The present specification permits ignoring the effect of axial force when $e/h \geq 10$ to simplify the calculations and on account of the fact the results thus obtained are conservative because the value of $\gamma_i N'_d$ is considerably smaller than axial compressive capacity N'_{ud} at equilibrium. Tensile axial forces, however, should not be ignored.

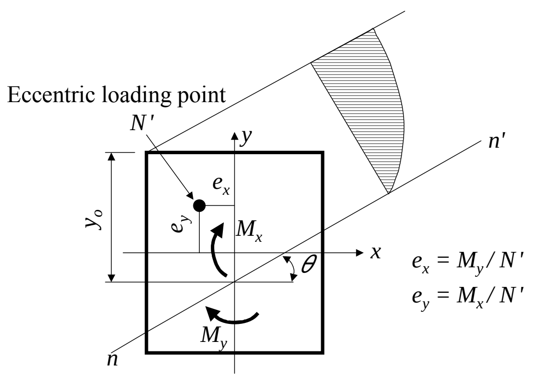


Fig. C9.2.2 Rectangular cross section subjected to biaxial flexure

9.2.2 Shear

9.2.2.1 General

(1) Design for shear shall take into consideration such factors as the type of member, e.g., linear, planar, and the direction of shear.

(2) For a linear member, the safety under the action of shear loads shall be examined on the basis design shear capacities V_{vd} and V_{wcd} as discussed in Section 9.2.2.2. If the shear span ratio is small, safety shall be checked with respect to the design shear compression capacity V_{dd} instead of these design shear capacities.

(3) For a planar member subjected to transverse shear, examination for transverse shear shall be made in a manner similar to that for a linear member. Examination for punching shear shall be carried out in accordance with the provisions of Section 9.2.2.3, when a concentrated load V_d is applied on the member.

(4) For a planar member subjected to in-plane shear, examination for in-plane shear shall be made in accordance with provisions of Section 9.2.2.4.

(5) If the shear force V_d must be transferred across construction joints or planes where the probability of crack occurrence is large, examination for direct shear transfer on shear planes shall be carried out in accordance with provisions of Section 9.2.2.5.

(6) For linear members subjected to reversed cyclic loads the member factor shall be appropriately increased to enhance the level of safety against shear failure.

(7) Strength verification tests or high-accuracy analyses should be conducted with respect to those parts of the structure where the flow of force changes considerably or the cross section changes abruptly, corners, opening zones, steel anchorage zones, or other regions to which the beam or plate theory cannot be applied, that is, so-called discontinuity regions. If such special verification is not used, verification may be done by using the strut-and-tie model.

[Commentary] (1) Behavior and failure mechanism of members subjected to shear forces depends on factors such as type of the member, e.g., linear or planar. In the case of planar members these are also affected by the direction of shear forces, e.g., transverse or in-plane. Design in shear shall be carried out taking account of these factors through appropriate methods.

This section explains the methods for examination of safety of commonly used structural members, such as linear members, e.g., beams and columns, and planar members, e.g., slabs, subjected to transverse shear forces, and, walls of cylindrical structures subjected to in-plane shear, in (2), (3) and (4) below. The method for examination of safety in direct shear transfer on planes such as those created by construction joints, and precast concrete joints, where the possibility of crack occurrence is large, is given in (5).

(2) For a linear member provided with shear reinforcement, the shear force is carried by shear reinforcement after the initiation of diagonal cracks and the load-carrying mechanism is transformed to a truss-type mechanism. Members lose their ability to resist further shear either by the yielding of the shear reinforcement, which act as tension chords of the truss, or, by the compressive failure of web concrete, which acts as the compression chords of the truss. Examination for safety in the cases of yielding of shear reinforcement and diagonal compressive

failure of web concrete is carried out using Eq. (9.2.3), and Eq. (9.2.8), respectively.

If, however, the shear span ratio (shear span/effective depth) is small (about 2.0 or smaller), the shear capacity may become greater than the shear capacity calculated from Eq. (9.2.3) because a deep-beam-like mechanism comes into play. Verification for the design shear compression strength, therefore, may be done by using Eq. (9.2.9).

(3) It is presented here that examination for shear of a planar member such as a one-way slab, which behaves like a beam, may be conducted in accordance with that of a linear member. The method to examine safety for punching shear failure caused by concentrated loads acting on a planar member is also described.

(4) On the basis of the several available methods for examination of safety of planar members subjected to in-plane shear, the present specification recommends use of a practical method to design an orthogonally reinforced planar member–design forces T_{xd} and T_{yd} in the reinforcement in the x and y directions and design diagonal compressive force of concrete (C'_d) is determined in accordance with provisions of Section 9.2.2.4(1) are compared with capacities T_{xyd}, T_{yyd} and C'_{ud} respectively computed using the provisions of Section 9.2.2.4(2).

(5) For a T-beam with a wide flange and a thin web, a larger shear force acts at the base of the flange which may sometimes cause shear failure. It is desirable to examine this phenomenon as the direct shear transfer.

(6) For a linear member whose safety is governed by reversed cyclic loading, it is necessary to enhance the level of safety against shear failure in order to prevent brittle failure. The member factor in such cases should, therefore, be increased by about 1.2.

(7) The design approach described here makes rational safety design possible for members, such as columns and beams, for which there is accumulated design and construction experience and there is extensive empirical knowledge about member cross sections and reinforcement methods needed to meet performance requirements. The load-carrying capacity, however, of abrupt-cross-sectional change regions, corners, opening zones, etc., is difficult to verify by the standard method described in Chapter 9, and past construction data on reinforcement methods in regions where the flow of force changes considerably are not necessarily available. One way to verify safety in such cases is to identify the load-carrying mechanism by conducting experiments or high-accuracy nonlinear analyses with respect to the assumed member configuration or material arrangements. Another method is to determine a load-carrying mechanism for design loads in advance and determine reinforcement patterns and material strength requirements so that the load-carrying mechanism can be achieved. As a method that is useful when employing the latter method of verification, the use of the strut-and-tie model is permitted. The strut-and-tie model is a structural model that clearly defines the load-carrying mechanism for design loads and the flow of force. In the strut-and-tie model method, a concrete structure or its members are discretized into one-dimensional struts and ties and nodes that connect together these elements, and the load-carrying capacity for the assumed flow of force is calculated from the equilibrium of forces and the strength of the struts and ties. Verification by the strut-and-tie model method is to be done in accordance with the Standard Specifications for Concrete Structures "Design: Design Methods, Part 6, Strut and Tie Model."

9.2.2.2 Design shear capacity of linear members

(1) The design shear capacity of a member V_{vd} may be obtained using Eq. (9.2.3). When both bent longitudinal bars and stirrups are arranged as shear reinforcement, it should be ensured that the stirrups provided carry at least 50% of the shear force provided by shear reinforcement.

$$V_{yd} = V_{cd} + V_{sd} + V_{ped} \tag{9.2.3}$$

where V_{cd} **: design shear capacity of linear members without shear reinforcing steel, obtained using Eq. (6.2.4).**

$$V_{cd} = \beta_d \cdot \beta_p \cdot \beta_n \cdot f_{vcd} \cdot b_w \cdot d / \gamma_b \tag{9.2.4}$$

$f_{vcd} = 0.20\sqrt[3]{f'cd}$ **(N/mm²) where** $f_{vcd} \le 0.72$ **(N/mm²)** **(9.2.5)**

$\beta_d = \sqrt[4]{1000/d}$ **(**d **: mm)** **when** β_d **> 1.5,** β_d **is taken as 1.5.**

$\beta_p = \sqrt[3]{100 p_v}$ **when** β_p **> 1.5,** β_p **is taken as 1.5.**

$\beta_n = 1 + 2M_o / M_{ud}$ $(N'_d \ge 0)$ **when** β_n **> 2,** β_n **is taken as 2.**

$= 1 + 4M_o / M_{ud}$ $(N'_d \langle 0)$ **when** β_n **< 0,** β_n **is taken as 0.**

N'_d **: design axial compressive force**

M_{ud} **: pure flexural capacity without consideration of axial force**

M_o **: flexural moment necessary to cancel stress due to axial force at extreme tension fiber corresponding to design flexural moment** M_d

b_w **: web width**

d **: effective depth**

$p_v = A_s / (b_w \cdot d)$

A_s **: area of tension reinforcement (mm²)**

f'_{cd} **: design compressive strength of concrete (N/mm²)**

γ_b **: 1.3 may be used in general**

V_{sd} **: design shear capacity of shear reinforcement, obtained using Eq. (6.3.5)**

$$V_{sd} = \left[A_w f_{wyd} (\sin\alpha_s + \cos\alpha_s) / S_s + A_{pw} \sigma_{pw} (\sin\alpha_p + \cos\alpha_p) / s_p \right] z / \gamma_b \tag{9.2.6}$$

A_w **: total area of shear reinforcement placed in** S_s

A_{pw} **: total area of prestressing steel expected to act as shear reinforcement placed in** S_p

σ_{pw} : tensile stress in prestressing steel acting as shear reinforcement when shear reinforcing steel yields

$$\sigma_{pw} = \sigma_{wpe} + f_{wyd} \leq f_{pyd}$$

σ_{wpe} : effective tensile stress in prestressing steel acting as shear reinforcement

f_{wyd} : design yield strength of shear reinforcement, and should not exceed 400 N/mm². However, if the characteristic compressive strength of concrete (f'_{ck}) is 60 N/mm^2 or more, a value of up to 800 N/mm^2 may be used.

f_{pyd} : design yield strength of prestressing steel expected to act as shear reinforcement

α_s : angle between shear reinforcement and member axis

α_p : angle between prestressing steel acting as shear reinforcement and member axis

S_s : spacing of shear reinforcement

S_p : spacing of prestressing steel expected to act as shear reinforcement

z : distance from location of compressive stress resultant to centroid of tension steel; may generally be taken as $d/1.15$.

γ_b : member factor. May generally be taken as 1.10

V_{ped} : component of effective tensile force in longitudinal tendon parallel to the shear force, obtained using Eq. (9.2.7).

$$V_{ped} = P_{ed} \cdot \sin\alpha_p / \gamma_b \quad (9.2.7)$$

P_{ed} : effective tensile force in longitudinal prestressing steel

α_p : angle between extreme compression fiber and member axis

α_t : angle between longitudinal prestressing steel and member axis

γ_b : 1.10 in general

(2) When linear members are supported directly, no further examination for V_{yd} may be required over a distance of one-half the total depth of members (h) from the face of the support. In this region, shear reinforcement not less than that required at the cross section located $h/2$ from the face of the support shall be provided. The depth at the face of the support may be used for members with varying depth, assuming that the haunch (whose slope does not exceed 1:3) is also effective in resisting shear forces.

Appropriate methods of design shall be used to estimate the design shear capacity in the neighborhood of supports, when planar members subjected to transverse shear are treated as linear members, and the provisions of Section 9.2.2.1(3) are used.

(3) The design diagonal compressive capacity V_{wcd} of web concrete in resisting applied shear forces may be calculated by Eq. (9.2.8).

$$V_{wcd} = f_{wcd} \cdot b_w \cdot d / \gamma_b \qquad \textbf{(9.2.8)}$$

where $f_{wcd} = 1.25\sqrt{f'_{cd}}$ **(N/mm²), when** $f_{wcd} \le 7.8$ **(N/mm²)**

γ_b **: member factor. May generally be taken as 1.3**

(4) Web width of members

(i) In cases when the diameter of a duct in prestressed concrete members is equal to or greater than 1/8 of the width of the web, the width used in Eq. (9.2.4) shall be appropriately reduced (from the actual width, b_w). It is recommended that the web width may be reduced to ($b_w - 1/2\sum\varphi$), i.e. by an amount equal to one-half the sum of all the diameters of the ducts 'φ' spaced in the cross section.

(ii) For members with variable web width in the direction of member depth except for circular cross sections, the web width b_w shall be taken as the minimum width within the effective depth d. For members with several webs, the total width of webs shall be taken as b_w. For members with solid or hollow circular cross sections, the web width shall be defined as the side length of the square with the same area as solid circular cross section or the total width of webs of the square box having the same area as hollow circular cross section. In these cases, the area of longitudinal tensile steel A_s may be defined as the area of steel being arranged in 1/4 (90°) portion of the cross tensile section. The effective depth d may be taken as the distance from the edges of the squares or the square boxes at the compression side to the centroid of the steel section accounted as A_s, as shown in Fig.9.2.2.

These definitions of area for longitudinal tensile steel shall not be applied for the computation of flexural capacity.

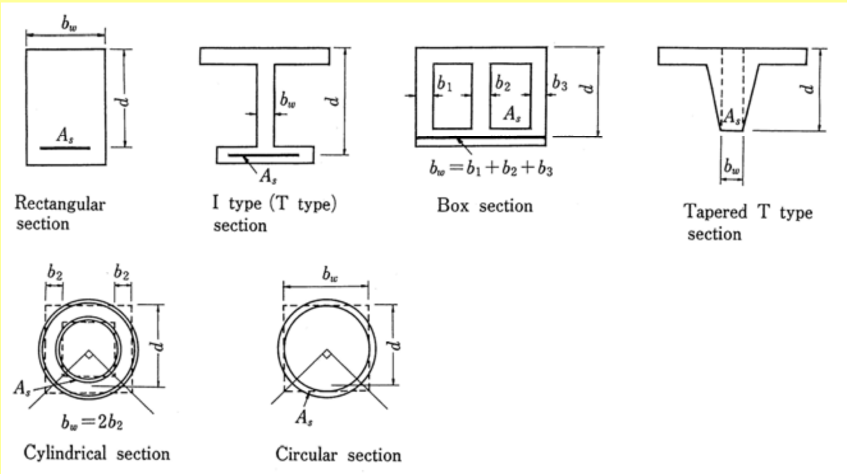


Fig. 9.2.2 Definitions of b_w and d concerning various shapes of cross sections

(5) The design shear compression capacity may be calculated from Eq. (9.2.9).

$$V_{dd} = \beta_d \cdot \beta_n \cdot \beta_p \cdot \beta_a \cdot f_{dd} \cdot b_w \cdot d / \gamma_b \qquad \textbf{(9.2.9)}$$

where V_{dd} **: design shear compression capacity (N)**

$$f_{dd} = 0.19\sqrt{f'_{cd}} \quad \text{(N/mm}^2\text{)} \qquad (9.2.10)$$

$\beta_d = \sqrt[4]{1000/d}$ **(** d **: mm)** **when** β_d **> 1.5,** β_d **is taken as 1.5.**

$\beta_n = 1 + 2M_o / M_{ud} \quad (N'_d \geq 0)$ **when** β_n **> 2,** β_n **is taken as 2.**

$\quad = 1 + 4M_o / M_{ud} \quad (N'_d \langle 0)$ **when** β_n **< 0,** β_n **is taken as 0.**

$\beta_p = \dfrac{1+\sqrt{100 p_v}}{2}$ **when** β_p **> 1.5,** β_p **is taken as 1.5.**

$$\beta_a = \frac{5}{1+(a/d)^2}$$

b_w **: web width**

d **: effective depth at the loading point in case of a simple beam or at the support front end in case of a cantilever beam (mm)**

a **: distance from the support front end to the loading point (mm)**

$p_v = A_s / (b_w \cdot d)$

A_s **: area of tension reinforcement (mm²)**

M_{ud} **: pure flexural capacity without consideration of axial force**

M_o **: flexural moment necessary to cancel stress due to axial force at extreme tension fiber corresponding to design flexural moment** M_d

N'_d **: design axial compressive force**

f'_{cd} **: design compressive strength of concrete (N/mm²)**

γ_b **: 1.3 may be used in general**

[Commentary] (1) The design shear capacity V_{yd} is basically the sum of the components carried by concrete V_{cd} and by shear reinforcement V_{sd}. The value of V_{sd} is calculated in accordance with the truss theory that assumes yielding of shear reinforcement and considers 45° compression diagonals. Consequently, V_{yd} gives the capacity corresponding to yielding of shear reinforcement. In reality, shear capacity may be considerably greater than V_{yd} if the amount of shear reinforcement is relatively small. However, no precise method to estimate the actual shear capacity is yet available, and therefore the present specification recommends use of Eq. (9.2.3) as a practical and conservative formula.

Eq. (9.2.4) is derived taking in to account the effect of concrete strength, member depth, reinforcement ratio and axial force on V_{cd}. Relationship between the shear strength by this equation and the characteristic compressive strength of concrete is shown in Fig. C9.2.3. f'_{cd} in

Eq. (9.2.5) is the design compressive strength of concrete, obtained by dividing the characteristic compressive strength of concrete, f'_{ck}, by the material factor γ_c. The effects of the effective depth, ratio of axial reinforcement and axial forces are shown in Fig. C9.2.4 (a), (b) and (c). When the characteristic compressive strength of concrete exceeds 80 N/mm^2, there may not be a significant increase in V_{cd} even when the concrete compressive strength increases.

The Standard Specifications for Concrete Structures "Design" adopted in 1996, therefore, had placed an upper limit of 0.72 N/mm^2 for f'_{vcd}. In preparation for the change in the value of the material factor γ_c for concrete having a characteristic compressive strength of 60 N/mm^2 or more from 1.5 to 1.3, this upper limit was reviewed taking into consideration recent research results including the latest experiment data. As a result, although V_{cd} for concrete having a compressive strength of 60 N/mm^2 or more tended to level off, the review confirmed that safety can be ensured for all types of test specimens considered, even in the formula proposed by Niwa et al. that takes into account the effect of a/d, from which Eq. (9.2.4) was derived, by setting an upper limit of 0.72 for f'_{vcd} and using a material factor of 1.3. In view of the fact that Eq. (9.2.4) gives more conservative values than the Niwa formula because the former neglects the effect of a/d, use of Eq. (9.2.4) with an upper limit of f'_{vcd} of 0.72 and a member factor γ_b of 1.3 was permitted.

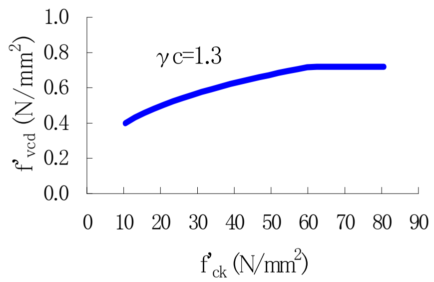


Fig.C9.2.3 Relationship between characteristic compressive strength of concrete f'_{ck} and f'_{vcd}

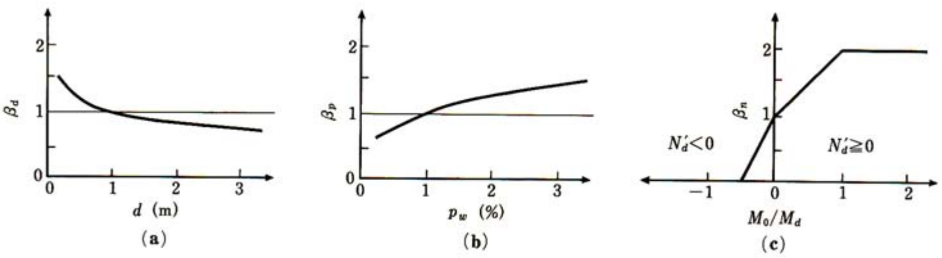


Fig. C9.2.4 Influencing factors on the shear capacity

Taking β_n as $(1+2M_o/M_u)$ may bring better applicability for the case of failure of members subjected to axial compression also, but for a conservative and safe estimate it may be assumed that $2M_o/M_u \approx M_o/M_d$, where M_d is the design flexural moment obtained based on the load for the ultimate limit state, and M_u is the design flexural capacity of members.

Only a few papers address the estimation of shear strength of members subjected to axial tension, and further research on the subject is required. Based on the experimental verification with previously reported test data, the value of β_n is found to be $1+4M_o/M_d$, where the definition of M_o is as shown in Fig. C9.2.5. For simplicity, the gross cross section of concrete may be considered to be effective when obtaining M_o.

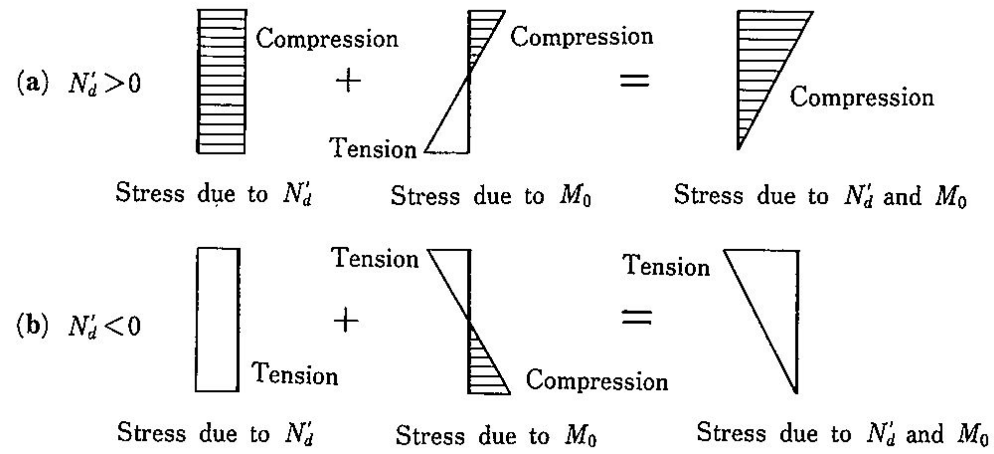


Fig. C9.2.5 Meaning of M_o (M_d>0)

Eq. (9.2.5) is applicable to normal concrete and the value in the case of lightweight aggregate concrete may be taken to be 70% of value calculated by this equation. The first term in the bracket in Eq. (9.2.6) represents the shear capacity of shear reinforcement including stirrups, hoops and bent bars, and the second one is the contribution to the capacity by diagonal or vertical prestressing steel.

When extremely high strength bars are used for shear reinforcement, the design yield strength of shear reinforcement f_{wyd} shall be limited to 400 N/mm², because an excessive width of diagonal crack at shear failure reduces the shear force resisted through aggregate interlock across crack planes, or dowel actions of the tensile reinforcement. However it has been confirmed experimentally that Eq. (9.2.3) may be used even when high strength reinforcement having yield strength of 800 N/mm^2 is used as shear reinforcement, if the concrete used has a characteristic compressive strength f'_{ck} of 60 N/mm^2 or more. Therefore, the upper limit of f_{wyd} when f'_{ck}>60 N/mm^2 is set at 800 N/mm^2.

It has been reported that in cases for self-compacting concrete Eq. (9.2.3) holds true for shear reinforcement having yield strength of around 800 N/mm^2 even when f'_{ck} is 50 N/mm^2. It has also been reported that bond characteristics, crack distribution and crack width of concrete vary with self-compactability even when compressive strength is the same. Since it has been pointed out that distribution of diagonal cracks is related to shear capacity, the upper limit of f_{wyd} of 800 N/mm^2 may be used for self-compacting concrete if the characteristic compressive strength of the concrete is not lower than 50 N/mm^2.

Stirrups resist shear force as tension web chord in the assumed truss system and furthermore, prevent diagonal cracks from propagating along tension reinforcement by their enclosure. The minimum amount of stirrup reinforcement required for this effect to take place has not yet been clearly established. Therefore, the present specification requires that at least 1/2 of the shear force to be carried by shear reinforcement is carried by stirrup.

When designing rectangular cross sections subjected to biaxial shear forces, it may be ensured that Eq. (C9.2.1) is satisfied.

$$(\gamma_i V_{dx} / V_{yx})^2 + (\gamma_i V_{dy} / V_{yy})^2 \leq 1.0 \qquad \text{(C9.2.1)}$$

where, V_{yx} : design shear capacity along x-axis

V_{yy} : design shear capacity along y-axis

V_{dx} : design shear force along x-axis under biaxial shear forces

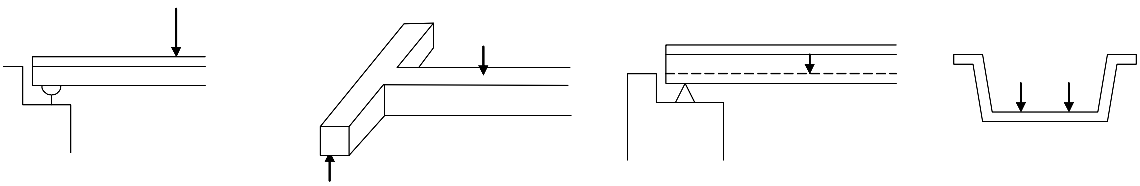
V_{dy} : design shear force along y-axis under biaxial shear forces

For circular cross sections, hoop reinforcement and spiral reinforcement may be regarded as shear reinforcement.

(2) The truss mechanism may develop in regions separated from the supports by some distance, but the so-called arch mechanism operates in the neighborhood of the supports, where compression is transferred directly to the supports. It is generally considered that failure is not initiated in regions close to the supports because of the enhanced shear capacity due to the arch action. It has also been confirmed that the effect of shear reinforcement arranged close to supports differs from that placed in the middle of beams, but the actual behavior at the edges of beams is not very well understood yet. Considering the need for simplicity of design procedures, the shear capacity (V_{yd}) based on the truss mechanism need not be examined within a distance of one half the member total depth from the faces of supports.

However, in special situations if large concentrated loads are applied at sections close to supports, appropriate examination for shear compression capacity, V_{dd}. As the shear capacity of linear members subjected to distributed loads in regions near supports and in corners is also larger than in other regions, use of a more appropriate review for design is recommended.

In a member supported directly compressive stresses are developed in the member web due to applied loads and support reactions as shown in Fig. C9.2.6 (a). This rule shall not be applied to the case shown in Figure C9.2.6 (b) where the support reaction is transferred to the main beam through the lateral beams and others, or, in the case shown in Figure C9.2.6 (c) where the applied loads do not produce any compressive stress in the member web.



(a) Direct loading, direct support (b) Indirect support (b) Indirect loading

Fig. C9.2.6 Direct and indirect supports

In many cases no shear reinforcement is arranged for planar members such as box culverts and retaining walls. Therefore, the design method regarding sections close to supports of linear members assuming that shear reinforcement is necessary is not always applicable. Planar members subjected to distributed loads and the corner regions of members may be designed using a more accurate and well-founded method.

For simply supported slabs subjected to uniform loads, for example, it is enough to check the safety of the section at distance d of the effective depth from the support. Further examination at other sections may not be required on the premise that the main tension reinforcement at this section is embedded beyond the support. Design for planar members with comparatively large thickness subjected to loads other than those that are uniformly distributed may be carried out in accordance with Section 12.7 "Footing".

(3) When excessive amounts of shear reinforcement is provided, there is a possibility that failure of member is caused by the compressive failure of web concrete and the shear reinforcement does not yield. This section is given to avoid the occurrence of this type of failure. Given the lack of experimental data, the present specification recommends use of Eq. (9.2.8) for a safe estimate. Relationship between f_{wcd} and the characteristic compressive strength of concrete f'_{ck} is shown in Fig. C9.2.7.

Since the indicated formula was derived for application to ordinary concrete, appropriate experiments should be carried out to verify its applicability to high-strength concrete, etc. In case the applicability is not verified through tests the equation may be adopted only for concretes not exceeding f'_{ck}=50 N/mm^2.

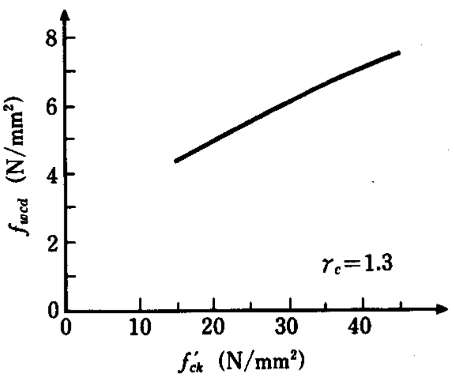


Fig. C9.2.7 Relationship between characteristic compressive strength of concrete f'_{ck} and f_{wcd}

(5) If the shear span ratio a/d is small, a directly supported bar member, even after the occurrence of diagonal cracking, continues to behave like a tied arch with the tension reinforcement acting as a tie. Failure results, therefore, from the yielding (flexural yielding) of the reinforcement acting like a tie or crushing failure (shear failure) of concrete. In this case, the tension reinforcement must not be anchored midspan because the tension reinforcement acts as a tie in a tied arch, and it is necessary to anchor all reinforcing bars needed to resist the maximum bending moment continuously beyond the support points.

Failure of the concrete in an arch rib is likely to occur as a shear failure. In this section, the shear compression strength equation has been formulated by conservatively simplifying a recently derived equation (C9.2.2) so that it asymptotically approaches the shear strength formula for ordinary bar members at a shear span ratio a/d of 2.5 or so. An arch rib considered here is formed by a cantilever beam or simple beam. If, therefore, a different type of arch rib formation is expected, detailed verification should be done separately on an as-needed basis.

$$Vc = \frac{0.24 \cdot k \cdot f_c'^{2/3} \cdot (1+\sqrt{100 p_v}) \cdot (1+3.33 r/d)}{1+(a/d)^2} b_w \cdot d \qquad \text{(C9.2.2)}$$

$$k = 1 + 7.4\sqrt[3]{100 p_w} \cdot (a/d - 0.75) / f_c'^{2/3}$$

where, r : length of bearing plate along the longitudinal axis

It has been confirmed experimentally that a beam with a small a/d value has a shear reinforcement effect if horizontal reinforcement is provided in the main section of the member. In this case, the tensile steel ratio p_v is a value obtained from Eq. C9.2.3, and the design shear compression strength V_{dd} can be calculated by substituting that value in Eq. 9.2.9.

$$p_v = p_{v1} + p_{v2} \cdot d_2 / d_1 \qquad \text{(C9.2.3)}$$

where, p_v : tension steel ratio

p_{v1} : tension steel ratio of tension steel

p_{v2} : tension steel ratio of horizontal reinforcement provided in the main section of the beam

d_2 : distance from the extreme compression fiber of tension reinforcement

d_1 : distance from the extreme compression fiber in horizontal reinforcement provided in the main section of the beam

It has been shown that shear reinforcement contributes to shear reinforcement by controlling local failure of arch rib concrete. If the effect of shear reinforcement is also taken into consideration, Eq. C9.2.4 may be used for calculation. There are cases, however, where there is little reinforcing effect if the amount of shear reinforcement is too small. When considering the effect of shear reinforcement, therefore, it is necessary to arrange shear reinforcing bars so that the shear reinforcement ratio becomes 0.2% or higher.

$$V_{dd} = (\beta_d \cdot \beta_n + \cdot \beta_w) \beta_p \cdot \beta_a \cdot f_{dd} \cdot b_w \cdot d / \gamma_b \qquad \text{(C9.2.4)}$$

where, V_{dd} : design shear compression capacity (N)

$\beta_d = \sqrt[4]{1000/d}$ (d : mm) when $\beta_d > 1.5$, β_d is taken as 1.5.

$\beta_w = 4.2\sqrt[3]{100 p_w} \cdot (a/d - 0.75) / \sqrt{f'_{cd}}$ when $\beta_w < 0$, β_d is taken as 0.

$\beta_n = 1 + 2M_o / M_{ud}$ $(N'_d \geq 0)$ when $\beta_n > 2$, β_n is taken as 2.

$= 1 + 4M_o / M_{ud}$ $(N'_d \langle 0)$ when $\beta_n < 0$, β_n is taken as 0.

$\beta_p = \dfrac{1 + \sqrt{100 p_v}}{2}$ when $\beta_p > 1.5$, β_p is taken as 1.5.

$$\beta_a = \frac{5}{1 + (a/d)^2}$$

b_w : web width

d : effective depth at the loading point in case of a simple beam or at the support front end in case of a cantilever beam (mm)

a : distance from the support front end to the loading point (mm)

$p_v = A_s /(b_w \cdot d)$

A_s : area of tension reinforcement (mm^2)

p_w : shear reinforcement ratio

A_w :total cross-sectional area of shear reinforcement perpendicular to the member axis in section s_s (mm2)

s_s : spacing of shear reinforcement perpendicular to the member axis (mm)

M_{ud} : pure flexural capacity without consideration of axial force

M_o : flexural moment necessary to cancel stress due to axial force at extreme tension fiber corresponding to design flexural moment M_d

N'_d : design axial compressive force

f'_{cd} : design compressive strength of concrete (N/mm^2)

γ_b : 1.3 may be used in general

If Eq. 9.2.9 and Eq. C9.2.4 are used to calculate the shear strength of a member with a large amount of longitudinal reinforcement its cross section such as a bridge pier with a small shear span ratio, shear strength tends to be underestimated. The design shear strength of such a member, therefore, should be calculated appropriately, taking into account the influence of the longitudinal reinforcement in the main section of the member. Eq. 9.2.9 can also be used in cases where concrete with a compressive strength of about 60 to 80 N/mm2.

The scope of application of Eq. 9.2.9 has been determined conservatively by referring to previous research results. If multiple loads or distributed loads act in the span, the distance from the center of gravity to the support front end may be taken as a. If a member is not supported directly or if the structural detail requirements are not met, Item (5) must not be applied because the load-carrying mechanism as described here may not be formed.

Eq. 9.2.9 is applicable to normal concrete. If the equation is to be applied to lightweight aggregate concrete, the value needs to be reduced. Generally, 70% of the value calculated from Eq. 9.2.9 may be used.

9.2.2.3 Design punching shear capacity of planar members

(1) When the loaded area is positioned far from free edges or openings and the eccentricity of the load is small, the design punching shear capacity V_{pcd} shall be determined using Eq. (9.2.11).

$$V_{pcd} = \beta_d \cdot \beta_p \cdot \beta_r \cdot f_{pcd} \cdot u_p \cdot d / \gamma_b \qquad \textbf{(9.2.11)}$$

where

$$f'_{pcd} = 0.20\sqrt{f'_{cd}} \text{ (N/mm}^2\text{), where } f'_{pcd} \le 1.2 \text{ (N/mm}^2\text{)} \qquad \textbf{(9.2.12)}$$

$\beta_d = \sqrt[4]{1000 / d}$ **(d : mm) when β_d > 1.5, β_d is taken as 1.5.**

$\beta_p = \sqrt[3]{100 p}$ **when β_p > 1.5, β_p is taken as 1.5.**

$\beta_r = 1 + 1/(1 + 0.25u / d)$

f'_{cd} **: design compressive strength of concrete(N/mm^2)**

u **: peripheral length of loaded area**

u_p **: peripheral length of the design cross section located at a distance d/2 from the loaded area**

d **and** p **: effective depth and reinforcement ratio defined as the average values for the reinforcement in two directions**

γ_b **: Member factor. May generally be taken as 1.3**

(2) When the loaded area is located in the vicinity of free edges or openings in members, the reduction of the punching shear capacity shall be appropriately taken into account.

(3) When the loads are applied eccentrically to the loaded area, the effects of flexure and torsion shall also be considered.

[Commentary] (1) When a local load is applied to a planar member such as a slab, the punching shear failure may occur in a manner that the concrete under the loaded area dents from the surrounding portion. For members subjected to the local loads such as column-slab connections and footings, the possibility of punching shear failure shall be examined.

The Specification assumes that the punching shear capacity of slabs can be estimated using the equation used for the estimation of the shear capacity of beams in view of the difficulty in establishing exact formulations to derive the punching shear capacity of slabs. The parameters used in Eq. (9.2.11) are determined by previously reported experimental results concerning punching shear for slabs and footings.

Relationship between the calculated value by Eq. (9.2.12) and the characteristic compressive strength of concrete is shown in Fig. C9.2.8. The factor β_r in Eq. (9.2.11) represents the effect of the loaded area and its value should be taken as shown in Fig. C9.2.9.

Since the formula was derived for application to ordinary concrete, appropriate experiments should be carried out to verify its applicability to high-strength concrete, etc. In case the applicability is not verified through tests, the equation may be adopted only for concretes not exceeding f'_{ck} =50 N/mm^2.

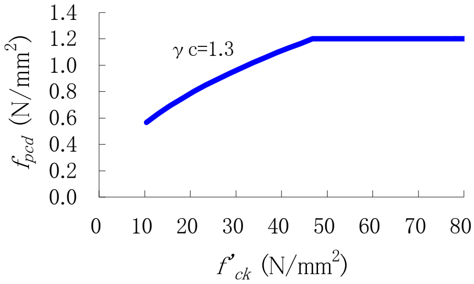


Fig. C9.2.8 Relationship between characteristic compressive strength of concrete f'_{ck} and f'_{pcd}

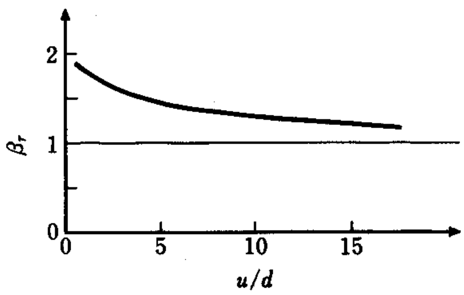


Fig. C9.2.9 Influence of loaded area on punching shear capacity

The failure mode of punching shear shall be such that the conical or pyramidal shape of fractured slope is created. Equation (9.2.11) for the shear capacity is derived on the assumption that the design cross-section is located d /2 from the loaded area as shown in Fig. C9.2.10.

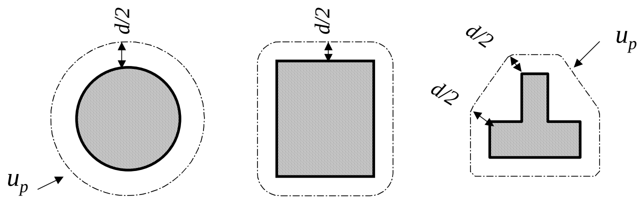


Fig. C9.2.10 Design cross section

The failure mode of members subjected to pull-out loads, such as steel tower footings, is similar to the failure mode of punching shear. The design capacity shall be calculated by the followings:

i) When the design for pull-out shear is conducted in accordance with the case for punching shear, the peripheral length u of loading face shall be computed by Eq. (C9.2.5) (See Fig. C9.2.11).

$$u = 4 \cdot \sqrt{2} \cdot r \text{ : with attachment}$$

$$= 2 \cdot \pi \cdot r \text{ : without attachment} \qquad \text{(C9.2.5)}$$

where, r : distance between center of pier and the edge of attachment for the case with attachment and distance between center of pier and the connection of anchor for the case without attachment.

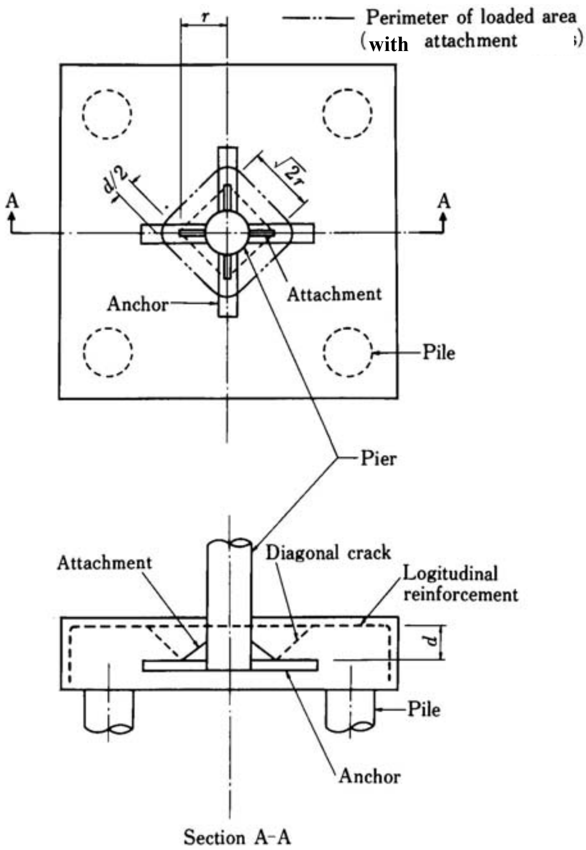


Fig. C9.2.11 Critical cross section for shear of footing with pull-out force

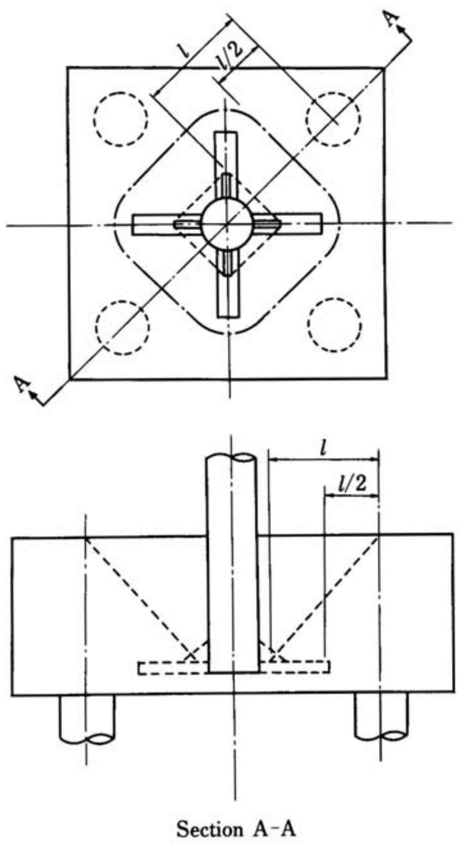


Fig. C9.2.12 Critical cross section for the case with closely spaced piles

ii) When the distance l between the edge of loading face and the center of the piles is less than the effective depth d, the critical cross section shall be, in general, at 1/2 of distance between the edge of loading face and the center of the piles (See Fig. C9.2.12).

iii) When pull - out force is applied to footing, the mechanism of failure is clearly similar to the case of punching shear. Some experimental results with shear reinforcements show them to be similar to the cases of directly transmitted shear. However, there are not many experimental data for cases with shear reinforcement and formula has not yet been established to-date. Therefore, when the effect of shear reinforcement is confirmed by an experiment or other method, pull-out shear strength may include this effect.

It has been confirmed that the shear failure of simply supported circular slabs (See Fig. C9.2.13), such as bottom slabs of tanks, on their peripheral sides subjected to uniformly distributed or line loads is not similar to the punching shear failure specified in this section but to the shear failure of linear members. Hence, it is appropriate that this type of shear capacity is determined according to Eq. (C9.2.6).

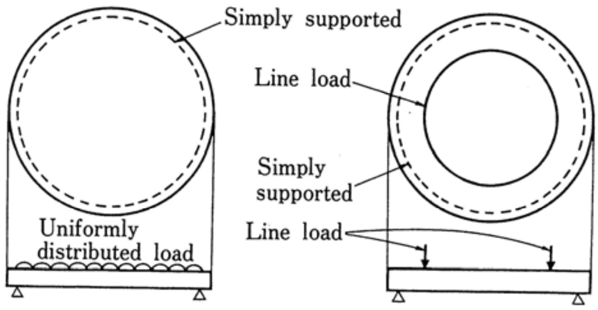


Fig. C9.2.13 Circular slabs subjected to uniformly distributed load or line load

As provided above, when designing a slab section, examination should be carried out for moments and shear forces assuming the slab to act as a beam. Further, in slabs subjected to a concentrated load, the examination for punching shear should be carried out.

However, available researches have shown that slabs such as an underground LNG tank, whose width is larger than their thickness, have greater shear resistance than that of ordinary slender beams. For example, experiments have shown that shear resistance of circular slabs is larger than that of ordinary beams because of the restraint to radial expansion provided by the reinforcing bars arranged in the circumferential direction. Experiments have also shown that the shear strength of simply supported and uniformly loaded circular slabs, whose diameter is larger than ten times the effective depth, may be computed by Eq.(C9.2.6), which has been derived from Eq.(9.2.4) and modified to account for the resistant due to circumferential reinforcements, which results in radial compressive forces.

$$V_{cdp} = \beta_d \cdot \beta_p \cdot \beta_\theta \cdot \beta_{a/d} \cdot f_{vcd} \cdot b_w \cdot d/\gamma_b \tag{C9.2.6}$$

where, $\beta_\theta = 1 + 2M_0 / M_u$

$$M_0 = P_n(d - h/3)\,,\ \ M_u = p_r f_y d^2\left(1 - \frac{p_r f_y}{1.7 f'_c}\right),\ \ P_n = \frac{f_y A_{s\theta}}{r} n$$

d : effective depth

h : overall height

p_r : reinforcement ratio in radial direction

$A_{s\theta}$: cross sectional area of one circumferential reinforcement

r : radius of slab at critical section

n : number of tensile reinforcements in circumferential direction installed outside of the critical section

$$\beta_{a/d} = 0.75 + \frac{1.4}{a/d}$$

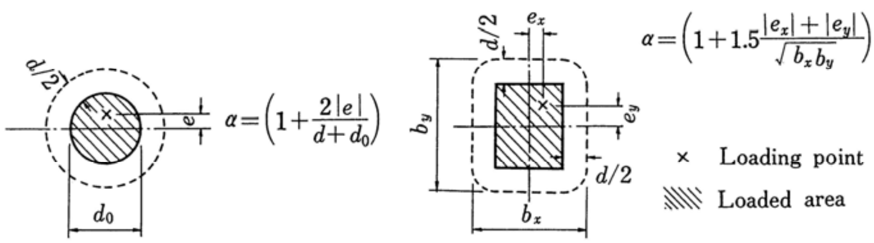
a/d : shear span to depth ratio

$\beta_0 \cdot \beta_{a/d} \leq 1.5$

Other notations have same definitions with those in Eqs.(6.3.4) and (6.3.5).

(2) If a free edge is located near the loaded area of a one-way slab, the punching shear capacity decreases. In such cases, it is advisable to regard the slab as a linear member having a width equal to the effective width of the slab and calculate the design shear capacity of a linear member as described in Section 9.2.2.2.

(3) Eq. (9.2.11) cannot be used when combined flexural and torsional moments are applied in a manner that corner columns support the slabs or when the applied loads are eccentric to the loaded area. Though sufficient experimental data is not available concerning the combined flexural and torsional moments in the presence of axial loads, it is proposed to reduce the punching shear capacity by multiplying the obtained capacity under axial loads by 1/α (See Fig. C9.2.14).



(1) For circular loaded area (2) For rectangular loaded area

Fig. C9.2.14 Strength reduction rate 1/α for eccentric loading

9.2.2.4 Design member forces in planar members subjected to in-plane forces

(1) For reinforced concrete planar members reinforced in two perpendicular directions, subjected to in-plane member forces, tensile forces T_{xd} and T_{yd} in each of the reinforcing directions and diagonal compressive force C'_d may be calculated as design in-plane forces using Eqs. (9.2.13), (9.2.14) and (9.2.15), respectively.

$$T_{xd} = N_1 \cos^2 \alpha + N_2 \sin^2 \alpha + (N_1 - N_2)\sin\alpha\cos\alpha \qquad \textbf{(9.2.13)}$$

$$T_{yd} = N_1 \sin^2 \alpha + N_2 \cos^2 \alpha + (N_1 - N_2)\sin\alpha\cos\alpha \qquad \textbf{(9.2.14)}$$

$$C'_d = 2(N_1 - N_2)\sin\alpha\cos\alpha \qquad \textbf{(9.2.15)}$$

where, T_{xd}, T_{yd} :design tensile force per unit width in the reinforcement along the x-axis and y-axis directions, respectively

α : angle between the in-plane force N_1 and the reinforcement along the x-axis (≤ 45°)

C'_d : design diagonal compressive force per unit width

N_1, N_2 : design value of principal in-plane forces; $N_1 \geq N_2$; N_1 being tensile

(2) For the examination of in-plane member forces for design member forces in accordance with (1), the design yield capacity of the reinforcement, T_{xyd}, T_{yyd} and the design capacity of concrete in compression, C'_{ud}, may be obtained using Eqs. (9.2.16), (9.2.17) and (9.2.18), respectively.

(i) Design yield capacity of reinforcement

$$T_{xyd} = p_x \cdot f_{yd} \cdot b \cdot t / \gamma_b \qquad \textbf{(9.2.16)}$$

$$T_{yyd} = p_y \cdot f_{yd} \cdot b \cdot t / \gamma_b \qquad \textbf{(9.2.17)}$$

where, p_x and p_y: ratio of reinforcement in x–axis and y–axis directions (A_s / bt)

b : member width: unit width in general

t : member thickness

γ_b : may be generally taken as 1.15

(ii) Design capacity of concrete in compression

$$C'_{ud} = f'_{ucd} \cdot b \cdot t / \gamma_b \qquad \textbf{(9.2.18)}$$

where,

$f'_{ucd} = 2.8\sqrt{f'cd}$ **(N/mm^2), where** $f'_{ucd} \leq 17$ **N/mm^2** **(9.2.19)**

γ_b : may be generally taken as 1.3

[Commentary] (1) When in-plane normal forces in the two perpendicular directions of reinforcement, and in-plane shear forces act together in reinforced concrete plate members reinforced in two perpendicular directions, the resultant state of stress can be represented by principal in-plane forces N_1 and N_2 having the angle α with reinforcement direction.

Equations (9.2.13) − (9.2.15) have been derived assuming that compressive forces develop in a direction diagonal to the orthogonal reinforcement and cracks form at an angle of 45° to the direction of reinforcement. These equations further assume that the shear force in the plane of the crack can be separated into two components – that parallel to the reinforcement and that parallel to the direction of the diagonal of reinforcement in the two directions, and, that the shear force is carried by the axial tension component of the reinforcement and the compressive stress component in the concrete.

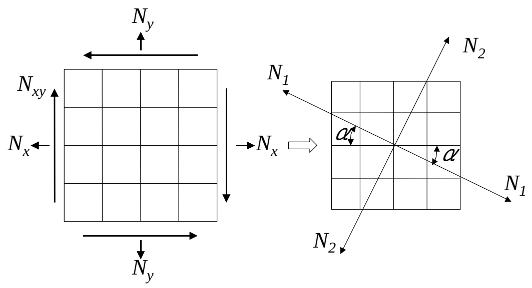


Fig. C9.2.15 Orthogonally reinforced concrete plate subjected to biaxial tension

(2) After cracking, the combined in-plane normal and shear forces are carried as axial tensile forces in reinforcement and compressive forces in concrete. Equations (9.2.16) and (9.2.17) for estimating the yield capacity of reinforcement are based on the assumption that the distribution of in-plane normal and shear forces over the depth of members is uniform.

In the estimation of the capacity of cracked concrete in compression, there are many unknown factors and Eq. (9.2.18) has been suggested for its simplicity to give a conservative estimate of the degree of reduction in compressive strength. The relationship between f'_{ucd} in this equation and the characteristic compressive strength of concrete is shown in Fig. (C9.2.16).

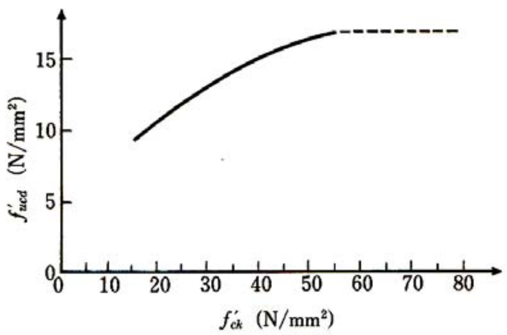


Fig. C9.2.16 Relationship between characteristic compressive strength of concrete f'_{ck} and f'_{ucd}

Recent research has shown that the reduction factor can be determined so that the reduction rate does not become extremely low even in cases where concrete strength is relatively high. On the basis of the results of recent studies on the reduction factor for compressive strength of cracked concrete, the reduction factor for the compressive capacity for in-plane forces of heavily cracked reinforced concrete members can be calculated from the expected maximum average tensile lateral strain (See Table C9.2.1). Therefore, f'_{ucd} may be calculated using Eq. (C9.2.7).

$$f'_{ucd} = \lambda \cdot f'_{cd} \tag{C9.2.7}$$

where, λ is a compressive strength reduction factor, whose value is given in Table C9.2.1 depending on the lateral strain.

Table C9.2.1 Reduction factor λ for lateral strain ε_t

Lateral strain ε_t * (x10^{-6})	2400 or less	3600	4800 or more
λ	0.8	0.7	0.6

* Average tensile strain in the direction perpendicular to crack

For conditions other than those shown in Table C9.2.1, linear interpolation may be used for intermediate lateral strains.

The values of the reduction factor given above, however, are based on the experimental results using test specimens comprising of ordinary concrete having a compressive strength of not greater than 50 N/mm^2) ordinary deformed steel bars, and, having a reinforcement ratio of up to about 2%. In the experiments, the specimens were subjected to compression and tension in orthogonal directions or pure shear. The values for the reduction factor given above are, therefore, valid only under these conditions, with an upper limit of lateral tensile strain as 5000x10^{-6}. When the reduction factor is used for cases where reversed cyclic loading could be repeated until a larger lateral tensile strain occurs, or, in cases where the effect of cyclic loading is large, 0.2 should be subtracted from the value given above to obtain the new reduction factor.

9.2.2.5 The design capacity for shear transfer

(1) When reinforcement is placed in a shear plane, the design capacity for shear transfer V_{cwd} under axial force may be computed using Eq. (9.2.20).

$$V_{cwd} = \{(\tau_c + p \cdot \tau_s \cdot \sin^2\theta - \alpha \cdot p \cdot f_{yd} \cdot \sin\theta \cdot \cos\theta)A_c + V_k\} / \gamma_b \tag{9.2.20}$$

where, $\tau_c = \mu \cdot f_{cd}{}^b (\alpha \cdot pf_{yd} - \sigma_{nd})^{1-b}$

$\tau_s = 0.08 f_{yd} / \alpha$

$\alpha = 0.75\{1 - 10(p - 1.7\sigma_{nd} / f_{yd})\}$

where, $0.08\sqrt{3} \le \alpha \le 0.75$ **(for deformed bars)**

σ_{nd} : **average normal stress acting on the shear plane;**

for compression, σ_{nd} may be taken as equal to $-\sigma'_{nd}/2$ In all cases, ($\alpha \cdot p \cdot f_{yd} - \sigma_{nd}$) must be positive.

σ'_{nd} : **average normal compressive stress acting on the shear plane**

α_t : **angle between tensile reinforcement and member axis**

p : **reinforcement ratio along the shear plane, where reinforcing steel with sufficient development length in both directions from the shear plane is taken into account**

A_c : **area of shear plane**

θ : **angle between shear plane and reinforcement**

b : **coefficient representing configuration of planes and ranging between 0 and 1; in general the values may be taken as follows:**

2/3 for a cracked surface (in the case of normal strength concrete), and, 1/2 in the case of a treated construction joint, cracked surface in the case of high-strength concrete, or, in the case of an adhesive-bonded joint between precast concrete members

u : **average coefficient of friction for solid-to-solid contact and may be taken as 0.45**

V_k : **shear capacity of the shear key**

$V_k = 0.1 A_k \cdot f'_{cd}$**, where** A_k **is the cross-sectional area of the shear key at the shear plane**

γ_b : **may be generally taken as 1.3**

(2) Design shear transfer capacity V_{cwd} **in cases when flexural moment and axial force act on the shear plane may be determined by finding the neutral axis under the action of the flexural moment and axial force to define tension and compression zones divided by the neutral axis, following the steps i) through iii) described below and then substituting the values of** $V_{cwd,t}$ **and** $V_{cwd,c}$ **calculated using Eq. (9.2.20) in Eq. (9.2.21).**

$$V_{cwd} = \beta_M \cdot V_{cwd,t} + V_{cwd,c} \quad \textbf{(9.2.21)}$$

where, $V_{cwd,t}$: **shear transfer capacity of the tension side of the shear plane**

$V_{cwd,c}$: **shear transfer capacity of the compression side of the shear plane**

β_M : **reduction factor to allow for the effect of flexural moment. In general, it's values may be taken as follows:**

$\beta_M = 4(1 - M_d / M_y)$ **where** $\beta_M \le 1$ **: for construction joints and cracks**

= 0 : for joints of precast concrete members

M_d : **design flexural moment acting on the shear plane**

M_y : **moment at yielding of the outermost reinforcement on the tension side**

i) Calculation of axial forces carried by reinforcement and concrete under flexural moment and axial force

In cases when flexural moment, M_d, and axial compressive force, N'_d, act on the shear plane, P_{st}, P'_{sc} and P'_c are calculated on the basis that assumptions (i) through (iv) of Section 9.2.1.1 (2) hold. Eq. (9.2.22) and Eq. (9.2.23) may be assumed for the stress-strain relationship for concrete and reinforcing steel, respectively.

$$\sigma'_c = E_s \varepsilon'_c \text{(where } \varepsilon'_c \geq 0\text{)} \quad (9.2.22)$$

$$\sigma = E_s \cdot \varepsilon \quad (9.2.23)$$

where, N'_d **: design axial compressive force acting on the shear plane**

P_{st} **: Total axial tensile force carried by the reinforcement on the tension side**

P'_{sc} **: Total axial compressive force carried by the reinforcement on the compression side**

P'_c **: Total axial compressive force carried by the concrete on the compression side**

ii) Calculation of yield moment M_y

The moment M_y at which the stress in the outermost reinforcing steel on the tension side reaches its design tensile yield strength, f_{yd}, should be calculated by a method similar to the one described in i) above.

iii) Calculation of shear transfer capacities $V_{cwd,t}$ and $V_{cwd,c}$

a) Assuming $\sigma_{nd} = P_{st} / A_{ct}$, $V_{cwd.t}$ is calculated using Eq. (9.2.20). In such cases, shear capacity V_k due to shear keys on the tension side should be ignored.

b) Assuming $\sigma_{nd} = -(1/2)(P'_{sc} + P'_c)/ A_{cc}$, $V_{cwd.c}$ is calculated using Eq. (9.2.20), where A_{ct} and A_{cc} are the shear area on the tension and the compression side, respectively.

(3) When the entire shear plane is in compression, design shear transfer capacity V_{cwd} may be calculated using Eq. (9.2.20), assuming $\sigma'_{nd} = N'_d / A_c$ and ignoring the effect of flexural moment.

[Commentary] (1) Shear transfer capacity is the sum of the transferred force on the concrete shear plane and the shear plane components of axial force and shear force acting on the intersecting reinforcing steel. Transferred force to be carried by the concrete depends on the compressive force acting on the concrete shear plane, surface roughness of the concrete, and the compressive strength of the concrete. The shear reinforcement also assumes the role of exerting vertical confining force on the concrete shear plane. Shear transfer capacity, therefore, is the sum of the forces transferred by the concrete and the reinforcement when the reinforcement loses confining force as it yields.

With respect to confining force exerted by reinforcement, yielding of the entire cross section can be assumed in the longitudinal direction if the reinforcement ratio is not greater than 1%. It is generally known, however, that in cases where shear displacement becomes large because of an increase in reinforcement ratio or loads acting perpendicularly on the shear plane, flexural moment and shear force as well as axial force act on the reinforcement cross section so that confining force is lost before the entire cross section of the reinforcement yields in the longitudinal direction. The reduction factor α in Eq. (9.2.20) expresses decreases in axial capacity of shear reinforcement and increases in shear capacity. The transfer capacity formula was derived by adding up the components in the direction of shear in the reinforcement and the concrete.

The shear transfer capacity of concrete is dependent on two factors: the interlocking effect due to the roughness of shear surfaces, and friction at the points of contact. In the case of shear along smooth construction joint surfaces, the interlocking effect decreases while friction becomes dominant. As a result, the influence of confining force becomes stronger as the concrete strength becomes lower. The coefficient b in Eq. (9.2.20) represents the effect of surface roughness. For cracks in high-strength concrete, the value of the coefficient b need to be made smaller because surface roughness decreases as the coarse aggregate particles fracture. If construction joint surface is not properly treated, 2/5 may be used for coefficient b. For safe and conservative evaluation, $\sigma_{nd} = -\sigma_{nd}/2$ is to be used if σ_{nd} is compressive. When σ_{nd} is tensile, the equation is to be used as it is.

Since Eq. (9.2.20) is applicable to a shear plane with reinforcement, $p = 0$ may be used when it is applied to a plane without reinforcement. "Reinforcement with sufficient development length" refers to reinforcement having a development length equal to at least 10 times the reinforcing bar diameter in both directions from the shear plane.

Precast concrete member joints are usually bonded with adhesives or mortar. The formula shown in this specification is applicable to adhesive-bonded joints for which sufficient experimental data is available as well as implementation examples. If mortar or other materials are to be used, a separate study needs to be conducted. The reason why shear capacity is given as the sum of friction due to axial force and the capacity of shear keys is that shear keys are often used with precast concrete.

(2) When flexural moment acts on a shear plane, the amount of shear stress that can be transferred varies within the shear plane depending on such factors as crack width. It is not possible, therefore, to apply Eq. (9.2.20), which assumes uniform shear stresses, as is in this case. For example, crack width is smaller where compression is greater in the cross-section, so greater shear stresses are transferred in areas where compression is greater. An exact calculation requires dividing the shear plane into small elements, calculating the shear transfer capacity of each element and summing the shear transfer capacities thus calculated, but performing such computations is an onerous task. Comparison was made of the method of defining the shear plane into a compression zone and a tension zone divided by the neutral axis and evaluating the shear capacity of each zone, and the method of rigorous analysis that divides the shear plane into smaller areas. The comparison showed that the former method is sufficiently accurate for practical purposes. For this reason, Eq. (9.2.21) is adopted as the formula for calculating shear transfer capacity under flexural moment and axial force.

If the shear surfaces are rough like crack surfaces, shear force is transferred to some degree because of the interlocking effect of the rough shear surfaces even if they are open. If, however, the shear surfaces cannot be reliably expected to become sufficiently rough, the shear transfer capacity of the tension zone should not be counted on. This is why a coefficient, β_M, by which to allow for the influence of flexural moment was introduced so that shear transfer capacity of the tension zone can be evaluated according to the acting flexural moment and the degree of roughness of the shear surfaces. If an adhesive is used at precast concrete member joints, a certain degree of surface roughness is likely. Therefore, if the value of β_M can be set so as to allow for the influence of such surface roughness, the shear transfer capacity of the tension zone may be taken into consideration.

Linear relationship is used when axial forces to be carried by the reinforcement and the concrete under flexural moment and axial force were calculated. The reason for this is that in cases where flexural moment is so large as to cause the outermost reinforcement in the tension zone to yield, the shear transfer capacity of the tension zone is ignored in evaluating β_M in order to be on the safe side. In general, in the design of cross sections before the yielding of steel reinforcement, the stress-strain relationship for reinforcement and concrete can be assumed to be elastic without causing significant errors.

In cases where the entire cross section is in tension, the shear transfer capacity of the tension zone alone needs to be evaluated because the neutral axis is out of the shear plane.

(3) The results of the rigorous analysis mentioned in (2) indicate that when the entire cross section is in compression, the influence of flexural moment on shear transfer capacity is relatively small. This is the reason that Eq. (9.2.20) is specified when the entire cross section is in compression.

9.2.3 Torsion

9.2.3.1 General

(1) For structural members not significantly affected by torsional moment and those subject to only compatibility torsional moment, the entire examination specified in Section 9.2.3 regarding safety against torsion may not be carried out. Here, 'structural members not significantly affected by torsional moment' refers to those structures where the value obtained by multiplying the ratio of the design torsional moment, M_{td}, to the design pure torsional capacity, M_{tcd}, as specified in Section 9.2.3.2 for no torsion reinforcement by a structure factor, γ_i, is less than 0.2 for all cross-sections

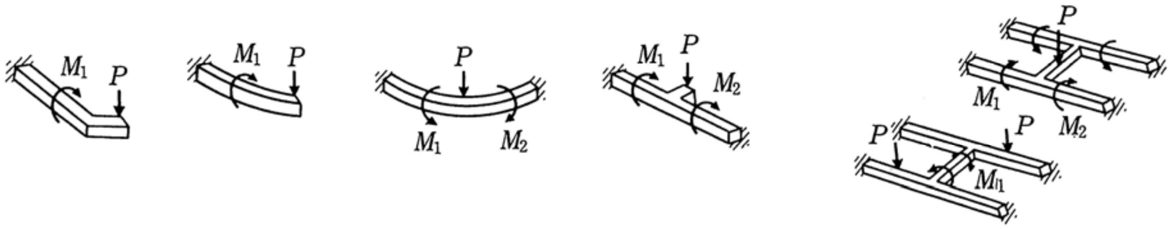
(2) When the design torsional moment, M_{td}, and the design pure torsional capacity, M_{tcd}, calculated using Eq. (9.2.26) satisfy Eq. (9.2.24) for all cross sections, examination specified in Section 9.2.3.2 may be omitted provided that the minimum torsional reinforcement is provided in accordance with Section 13.4.3.

$$\gamma_i M_{td} / M_{tcd} \leq 0.5 \qquad \textbf{(9.2.24)}$$

(3) When the design torsional moment, M_{td}, does not satisfy Eq. (9.2.24), torsion reinforcement shall be provided in accordance with Section 9.2.3.3.

(4) When torsional moment and flexural moment, or torsional moment and shear force act simultaneously, the examination for safety shall be carried out in a manner to take their interactions into consideration.

[Commentary] (1) From a design point of view, torsional moments are classified into equilibrium torsional moments and compatibility torsional moments (See Fig. C9.2.17).



(a) Equilibrium torsional moment (Statically determinate structure) (b) Equilibrium torsional moment (Statically indeterminate structure) (c) Compatibility torsional moment (Statically indeterminate structure)

Fig. C9.2.17 Equilibrium torsion and compatibility torsion

An equilibrium torsional moment is a torsional moment against which a structure must resist in order to keep the force equilibrium in the overall structure system. If this torsional moment is neglected in the calculation of the force equilibrium of the structure, the stability of the overall structure is destroyed. Namely, the capacity of the overall structure is completely dependent on the capacity of the members subject to torsion.

Meanwhile, a compatibility torsional moment is a torsional moment caused by the compatibility between the members composing a statically indeterminate structure, and provides an influence mainly on the elastic deformation of the structure. In general, the torsional rigidity of a concrete member is greatly reduced after diagonal cracking or plastic deformation by torsion. Hence, the torsional moment acting on the member becomes very small when a concrete member of a statically indeterminate structure reaches such a state.

Therefore, in the ultimate limit state, compatibility torsion may be neglected in the force equilibrium calculation, and it is clearly stated that the safety against a failure of a cross section under torsional moments is examined only for equilibrium torsion. Also, when a member is subject to compatibility torsion, it is necessary to assume that the torsional rigidity of the member cross-section is zero.

In an actual design, the examination for torsional moments may be deleted if a condition, $\gamma_i M_{td} / M_{tcd} < 0.2$, is satisfied. This condition is defined for the case where the influence of torsional moments is not significant. This is because examination for safety against torsion in accordance with Section 9.2.3 is cumbersome even when only small equilibrium torsional moments are acting.

(2) In the safety examination for torsion, torsion reinforcement is not required in general, when the design torsional moment is not greater than the design torsional capacity with the resistance of concrete only. However, even in such a case, the suitable minimum reinforcement is required to avoid a sudden failure, because cracking may occur due to the constraint at the member ends, stress concentration, drying shrinkage, temperature differences in concrete, and others.

The minimum torsional reinforcement is determined so that the torsional capacity is at least one-half of that of the member without torsion reinforcement.

9.2.3.2 Design torsional capacity for members without torsion reinforcement

(1) The design torsional capacity, M_{tud}, of a linear member without torsion reinforcement, subject to torsional moments only, shall be obtained using Eq. (9.2.25).

$$M_{tud} = M_{tcd} \qquad \textbf{(9.2.25)}$$

where, M_{tcd} **: design pure torsional capacity**

$$M_{tcd} = \beta_{nt} \cdot K_t \cdot f_{td} / \gamma_b \qquad \textbf{(9.2.26)}$$

K_t **: See Table 9.2.1**

β_{nt} **: coefficient for axial compressive force such as prestress**

$$\beta_{nt} = \sqrt{1 + \sigma'_{nd} / (1.5 f_{td})}$$

f_{td} **: design tensile strength of concrete**

σ'_{nd} **: average working compressive stress due to axial forces**

$\sigma'_{nd} \leq 7 f_{td}$

γ_b **= 1.3 (in general)**

(2) When flexural moment, M_d, and torsional moment, M_{td}, act simultaneously, examination for safety may be carried out by verifying that Eq. (9.2.27) is satisfied.

$$\gamma_i \left[\left\{ \left(M_{td} / M_{tcd} - 0.2 \right) / 0.7 \right\}^2 + M_d / M_{ud} \right] \leq 1.0 \qquad \textbf{(9.2.27)}$$

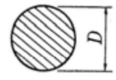
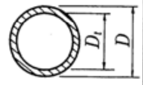
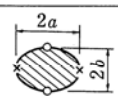
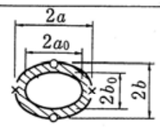
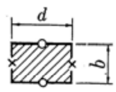
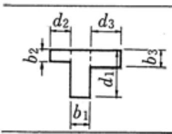
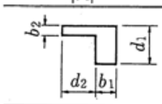
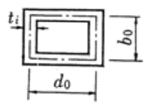
where, M_{ud} **: design flexural capacity calculated in Section 9.2.1.1**

(3) When shear force, V_d, and torsional moment, M_{td}, act simultaneously, examination for safety may be carried out by verifying that Eq. (9.2.28) is satisfied

$$\gamma_i \left(M_{td} / M_{tcd} + 0.8 V_d / V_{yd} \right) \leq 1.0 \qquad \textbf{(9.2.28)}$$

where, V_{yd} **: design shear capacity calculated from Eq. (9.2.3)**

Table 9.2.1 Factors for torsion

Shape of cross section	K_t	Remark
	$\dfrac{\pi D^3}{16}$	
	$\dfrac{\pi(D^4-D_i^4)}{16D}$	
	○: $\pi ab^2/2$ ×: $\pi a^2 b/2$	
	○: $\pi ab^2(1-q^4)/2$ ×: $\pi a^2 b(1-q^4)/2$	○: $q=a_0/a$ ×: $q=b_0/b$
	○: b^2d/η_1 ×: $b^2d/(\eta_1\eta_2)$	$\eta_1=3.1+\dfrac{1.8}{d/b}$ $\eta_2=0.7+\dfrac{0.3}{d/b}$
 	$\Sigma\dfrac{b_i^2 d_i}{\eta_{1i}}$ b_i and d_i are the length of shoter and longer sides of each of the component rectangles, respectively.	It is appropriate to subdivide the cross section into component rectangles in such a way as to result in the highest possible torsional rigidity.
	$2A_m t_i$ K_t of a box section shall be calculated as a hollow section. In case the ratio of the member thickness to the entire width of the box section in the direction of thickness exceeds 0.15, K_t may be calculated by taking the section as a solid one.	A_m: Area enclosed by the centerline of wall thickness. t_i: Web thickness

[Commentary] (1) The mechanical behaviors of a concrete member before torsional cracking can be estimated by a formula based on the elasticity theory, assuming that the entire concrete cross section is effective. Since the strain is small at this stage, the torsional resistance provided by reinforcement is, in general, neglected when compared with that of concrete. Therefore, before the cracking, the formula based on the elasticity theory is applied to a concrete cross section neglecting the effect of reinforcement, and the factors shown in Table 9.2.1 are used. Incidentally, K_t for a box section is obtained the same as for a hollow section. The specifications for a box section in Table 9.2.1 are determined by calculating with the ratio between member thickness and the overall width of the box section in the direction of the thickness as the parameter.

The factor, β_{nt}, in Eq. (9.2.26) for the axial compressive force such as prestressing forces corresponds to β_n in the equation to calculate the design shear capacity. This factor is derived from the maximum principal tensile stress theory that a failure occurs when the principal tensile stress due to the combination of the axial stress and the torsional shear stress reaches the tensile strength of concrete.

The term, $1.5 f_{td}$, in the equation for β_{nt} is an approximate value of the average tensile strength of concrete. Also, the axial compressive stress, σ'_{nd}, is the average value in the cross section.

When a member cross section is subjected to axial compressive stress with a triangular distribution such as prestress, but not to uniform axial compressive stress, the theoretical torsional capacity based on the elasticity, generally, does not coincide with the value calculated by using β_{nt} obtained from the average stress. However, it was decided that the average stress may be used in the calculation of β_{nt}, considering the fact that the theoretical values based on the elasticity are considerably on the safety side according to experimental results, and that, in actual cases, the ultimate axial stress distribution is different from that due to axial forces only, because of the flexural effect, and others.

When σ'_{nd} exceeds $7 f_{td}$, due to the combination of the compressive stress by torsional moments and the axial compressive stress, a member is in the domain of compressive failure and not in the domain of failure by the maximum principal tensile stress. In this case, even a small torsional moment may cause a sudden failure, which should be avoided. Therefore, the foregoing limit is specified.

For a member subject to axial tensile forces, β_{nt} given by Eq. (C9.2.8) may be used when the axial tensile force is small and does not cause cracking $(\sigma_{td} < f_{td})$. Also, it is assumed that $\beta_{nt} = 0$ when cracking occurs only due to axial tensile forces $(\sigma_{td} \geq f_{td})$.

$$\beta_{nt} = \sqrt{1 - \sigma_{td} / f_{td}} \quad (0 \leq \beta_{nt} \leq 1) \qquad \text{(C9.2.8)}$$

where, σ_{td} : average working tensile stress due to axial tensile force

However, designing in such a manner that the overall member is subject to axial tensile forces, and considering the combined load of torsional moments and axial tensile forces may result in the reduction of the torsional capacity of the member, so that considerable torsion reinforcement may be required. Therefore, it is desirable in such a case to consider means to avoid a design where a member is subject to combined load of torsional moments, and axial tensile forces by, for example, applying prestress.

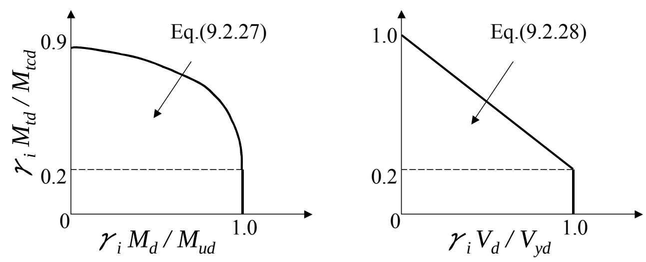
(2) The load capacity of a member subject to combined load of torsional moments and flexural moments is calculated by using the interaction relationship obtained from the maximum principal stress theory.

For a member without torsion reinforcement, and in the range where torsional moments are dominant, it is the principle of design that a member develops no cracking. However, in the range where flexural moments are dominant, the above design will present a large difference from the design method for flexural moment, unless cracking due to flexure is allowed. Therefore, herein, Eq. (9.2.27) that assumes occurrence of cracking is specified for the tensile region due to flexure. Eq. (9.2.27) integrates structure factors according to the interaction relationship indicated in the Standard Specification for Concrete Structures-1991 "Design". When the structure factor is 1.15, there is little difference from the conventional equation. When γ_i is smaller than 1.15, Eq.(9.2.27) gives conservative values.

(3) For the load capacity of a member subject to combined torsional moments and shear forces as well as that of a member subject to combined torsional and flexural moments, no cracking is allowed, in principle, in design for a member without torsion reinforcement when torsional moments are dominant. When shearing forces are dominant, Eq. (9.2.28) which assumes occurrence

of cracking due to shear forces is specified.

When torsional moment and flexural moment, or torsional moment and shear force are applied simultaneously, the interaction relationships can be expressed, using the design member force, the design capacity of member cross section and the structure factor, as shown in Fig. C9.2.18 (a) and (b). Safety verification can be made, therefore, by confirming that the values obtained from a given set of member forces are located inside the curve, that is, on the origin side. The reason for the fact that these interaction relationships are shifted to $\gamma_i M_{td} / M_{tcd} = 0.2$ is that the examination for torsion may be completely omitted for $\gamma_i M_{td} / M_{tcd} < 0.2$.



(a) Flexural moment and torsional moment (b) Shear and torsional moment

Fig. C9.2.18 Interaction relationship

9.2.3.3 Design torsional capacity for members with torsion reinforcement

(1) The design capacity for diagonal compression failure of web concrete against torsion, M_{tcud}, may be obtained using Eq. (9.2.29).

$$M_{tcud} = K_t \cdot f_{wcd} / \gamma_b \quad \textbf{(9.2.29)}$$

where, $f_{wcd} = 1.25\sqrt{f'_{cd}}$ **(N/mm²), where** $f_{wcd} \leq$ **7.8 N/mm²** **(9.2.30)**

K_t : **See Table9.2.1**

γ_b : **1.3 (in general)**

(2) The design torsional capacity of rectangular, circular, and annular cross sections, M_{tyd}, may be obtained using Eq. (9.2.31).

$$M_{tyd} = 2A_m\sqrt{q_w \cdot q_l} / \gamma_b \quad \textbf{(9.2.31)}$$

where, A_m : **effective area for torsion ($b_0 d_0$: rectangular, $\pi d_0^2 / 4$: circular, annular)**

b_o : **length of the shorter side of transverse reinforcement**

d_o : **length of the longer side of transverse reinforcement for a rectangular cross section and the diameter of concrete cross section enclosed by transverse reinforcement for circular and annular cross sections**

$$q_w = A_{tw} \cdot f_{wd} / s$$

$$q_l = \sum A_{tl} \cdot f_{ld} / u$$

$\sum A_{tl}$: **area of the longitudinal reinforcement that works effectively as torsion reinforcement**

A_{tw} : **area of a single reinforcing bar that works effectively as torsion reinforcement**

f_{ld}, f_{wd} : **design yield stresses of longitudinal and transverse reinforcement, respectively**

s : **longitudinal spacing of transverse reinforcement that works effectively as torsion reinforcement**

u : **length of the centerline of transverse reinforcement ($2(b_0+d_0)$: rectangular, πd_0: circular, annular)**

γ_b : **1.3 (in general)**

When $q_w \geq 1.25q_l, q_w = 1.25q_l$, **and when** $q_l \geq 1.25q_w, q_l = 1.25q_w$

(3) The design torsional capacity for T-, L-, and I-shaped sections may be obtained by considering the cross-section to be made up of rectangular sections and adding the torsional capacities of individual rectangular components , M_{tvdi}**, given by Eq. (9.2.31) in accordance with the following (i)−(iv). However, each** M_{tydi} **shall not exceed** $\xi \cdot A_{mi}$**.**

where, A_{mi} : **effective area for torsion of each component rectangle**

ξ : M_{tydi}/A_{mi} **of the component rectangle with maximum torsional effective cross section**

(i) A_{mi} **may be taken as the area enclosed by the transverse reinforcement, as shown in Fig.6.4.1.**

(ii) In cases when a longitudinal reinforcing bar for torsion belongs to more than one component rectangles, it shall not be taken into consideration more than once in this calculation.

(iii) For a T-shaped section with a continuous flange, the transverse reinforcement in the flange may be considered effective, although it does not enclose the longitudinal reinforcement. However, when the amount of the upper and lower reinforcement in the flange is different, the smaller value shall be taken as the limit.

(iv) The one-side effective flange width for torsion, λ_t**, may be computed using Eq. (9.2.32).**

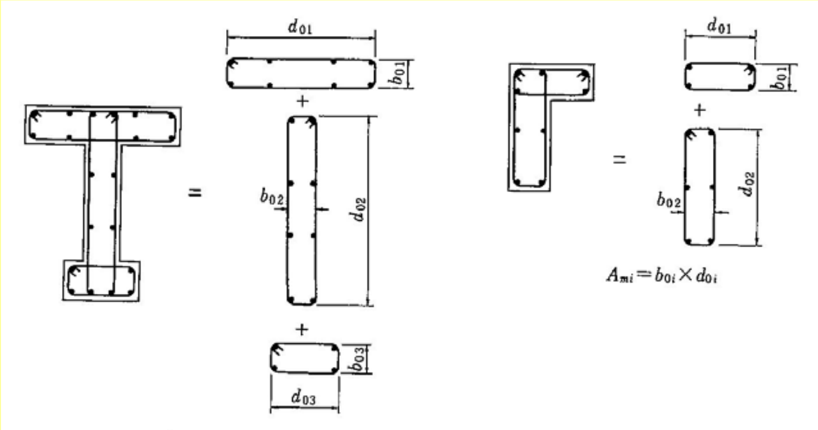
$$\lambda_t = 3t_i \qquad \textbf{(9.2.32)}$$

where, $\lambda_t \leq l_c$ **(for the cantilever part)**

$\lambda_t \leq l_b/2$ **(for the middle part)**

t_i : **average thickness of flange**

l_c, l_b : **lengths of cantilever slab and net spacing of beam, respectively**



Fig,9.2.3 Calculation scheme of A_{mi} for T and L sections

(4) A box section for which the ratio of the wall thickness to the overall width of the box section in the direction of thickness is not less than 1/4 shall be designed as a solid section. However, when the minimum ratio of the wall thickness to the overall width of the box section in the direction of thickness is less than 1/4 the provisions of (7) below shall apply.

(5) When a member is subject to flexural moment M_d and torsional moment M_{td} simultaneously, examination for safety of rectangular, circular and annular sections may be carried out by verifying that Eqs. (9.2.33) through (9.2.35) are satisfied.

$$\gamma_i M_{td} / M_{tu\min} \leq 1.0$$

(for $M_{ud} \geq M'_{ud}$ and $\gamma_i[M_d] \leq M_{ud} - M'_{ud}$) (9.2.33)

$$\gamma_i \left[\left(\frac{1.3(M_{td} - 0.2M_{tcd})}{M_{tu\min} - 0.2M_{tcd}} \right)^2 + \frac{|M_d| - M_{ud} + M'_{ud}}{M'_{ud}} \right] \leq 1.0$$

(for $M_{ud} \geq M'_{ud}$ and $M_{ud} - M'_{ud} \leq \gamma_i[M_d] \leq M_{ud}$ (9.2.34)

$$\gamma_i \left\{ \left(\frac{1.15(M_{td} - 0.2M_{tcd})}{M_{tu\min} - 0.2M_{tcd}} \right)^2 + \frac{|M_d|}{M_{ud}} \right\} \leq 1.0$$

(for $M_{ud} < M'_{ud}$ and $\gamma_i[M_d] \leq M_{ud}$) (9.2.35)

where, $M_{tu\min}$: smaller value of M_{tucd} and M_{tyd}

M_d : design flexural moment

M_{ud} : absolute value of the design flexural capacity when the longitudinal reinforcement located on the tension side under M_d is considered as tension reinforcement

M'_{ud} : absolute value of the design flexural capacity when the longitudinal reinforcement located on the compression side under M_d is considered as tension reinforcement

(6) When a member is subject to shear force V_d and torsional moment M_{td} simultaneously, examination for safety of rectangular, circular and annular sections may be carried out by verifying that Eq. (9.2.36) is satisfied.

$$\gamma_i \lfloor M_{td} / M_{tu\min} + (1 - 0.2 M_{tcd} / M_{tu\min})(V_d / V_{yd}) \rfloor \leq 1.0 \qquad \textbf{(9.2.36)}$$

where, $M_{tu\min}$: smaller value of M_{tucd} and M_{tyd}

V_{yd} : design shear capacity calculated from Eq. (9.2.3)

(7) For a box section for which the ratio of the wall thickness to the overall width of the box section in the direction of thickness is less than 1/4, the design torsional capacity, M_{tyd}, may be obtained using Eq. (9.2.37).

$$M_{tyd} = 2A_m (V_{odi})_{\min} \qquad \textbf{(9.2.37)}$$

where, $(V_{odi})_{\min}$: minimum value of the in-plane shear capacity per unit length of each wall

In this case, the provisions of Section 13.4.3 shall be adopted for the joint part of each wall and the anchoring of the reinforcement in concrete.

When a member is subject to flexural moments or shear forces together with torsional moments, examination for safety may be carried out in a manner similar to that used for a rectangular cross section

[Commentary] (1) When a large amount of torsion reinforcement is arranged in such a manner that they do not yield in both longitudinal and transverse directions, concrete will crush due to the diagonal compression in concrete before the torsion reinforcement yields. Since such a brittle failure should be avoided, the failure capacity for this case is taken by Eq. (9.2.29). This equation is an approximate one proposed as a substitution for limiting the quantity of torsion reinforcement to be not greater than that of balanced strain reinforcement.

(2) The resistance of a concrete member against torsional moment after the occurrence of torsional cracking may be considered as mainly due to the tensile force of reinforcement and the compressive force of concrete. However, according to experimental data, it is deduced that a part of the resistance is apportioned by the tensile force in concrete, the dowel action of reinforcement, the aggregate interlock, and others.

As the modeling methods for the analysis of the load bearing mechanism in this case, there are two methods of which one is the space truss theory and the other the diagonal flexure theory. Between them, the method based on the space truss theory is easier to calculate the torsional capacity due to yielding of torsion reinforcement. Therefore, the design method based on the space truss analogy is specified herein.

Equation (9.2.31) is an equation for the design torsional capacity derived from the force equilibrium when reinforcement is composed by reinforcing bars in the longitudinal direction and transverse reinforcing bars perpendicular to those, and when they both yield. However, when one of the reinforcement is dominant, the reinforcement up to 1.25 times the lesser is considered as torsion reinforcement.

Equation (9.2.31) may be considered as an equation for torsional capacity, replacing the solid cross section by a hypothetical thin-wall box section and assuming that $q = \sqrt{q_w \cdot q_l}$ is the ultimate shear flow. In this case, the area enclosed by the centerline of the thin-wall section (the centerline of the shear flow) is the torsional effective cross-sectional area. Here, this area is assumed to be the area enclosed by the centerline of the transverse reinforcement. There are various methods to define the effective cross-sectional area, and one of them is to take the area enclosed by the line connecting the center of each longitudinal reinforcing bar. At the present, as the conclusion to which of the methods are the best has not been obtained, the torsional capacity computed by Eq. (9.2.31) may be overestimated. Therefore, 1.3 is selected for the value of γ_b. The difference in the effective cross-sectional area by each method becomes larger for a smaller section and smaller for a larger section. Therefore, γ_b may be reduced up to about 1.15 when the precision of Eq. (9.2.31) is confirmed.

(3) For T, L, or I section, information for the torsional capacity, M_{tyd} is not sufficient at the present. Herein, the torsional capacity is basically considered as the sum of the torsional capacity of each component rectangle, based on a similar idea for the case of a member without torsion reinforcement. The method to obtain A_{mi} used in Eq. (9.2.31) is determined by considering the actual configuration of transverse reinforcement. According to this method, a part of concrete is double counted. However, it seems that there is no problem because the safety for concrete crushing is examined by Eq. (9.2.29) for the entire cross section.

(4) For a box section, it has been confirmed from experimental data that it may be treated in approximation as a solid cross section when the wall thickness is more than a certain value. Therefore, this rule is proposed as a simple method for a box section.

(5) The interaction relationship of the capacity of a cross section of a member subject to flexural and torsional moments simultaneously is influenced by the arrangement of longitudinal reinforcement. For symmetric reinforcement arrangement with respect to the horizontal axis (torsion type), the interaction curve becomes parabolic. A torsional capacity, however, increases by flexural effects when the tensile flexural reinforcement is provided more than the compressive reinforcement.

Various interaction relationships of the member capacity have been proposed. Herein, the following two equations which can express the effect of reinforcement allocation by a simple form are modified and shown below.

$$\frac{F_{ty}}{F_{by}}\left(\frac{M_{td}}{M_{tyd}}\right)^2 + \frac{M_d}{M_{ud}} = 1 \qquad \text{(C9.2.9)}$$

$$\left(\frac{M_{td}}{M_{tyd}}\right)^2 - \frac{F_{by}}{F_{ty}}\left(\frac{M_d}{M_{ud}}\right) = 1 \qquad \text{(C9.2.10)}$$

where, F_{ty}, F_{by} : tensile forces at yielding of upper and lower longitudinal reinforcement, respectively

Using M_{ud} and M'_{ud} instead of F_{by} and F_{ty}, respectively, and neglecting the increase of torsional capacity due to the flexural effect from the safety point of view, and further considering the consistency with the negligible torsion limit under which the effect of torsion is negligible ($\gamma_i M_{ty} < 0.2 M_{tcd}$), Eqs. (9.2.33), (9.2.34) and (9.2.35) were derived. Eq. (9.2.34) integrates structure factors according to the interaction relationship indicated in the Standard Specification for Concrete Structures-1991 “Design”. When the structure factor γ_i is 1.15, if M_d / M_{ud} is small and the torsional effect is dominant, the equation is more conservative than the conventional formula. If M_d / M_{ud} is large and the flexural effect is dominant, the equation becomes less conservative than the conventional formula. Considering the fact, however, that the conventional interaction relationship equation gives highly conservative approximations, and that design torsional moment are relatively small, it was decided to make modifications mentioned above and use a single structure factor. When γ_i becomes smaller than 1.15, the degree of safety gradually rises even in the range where the flexural effect is dominant.

Like Eq. (9.2.34), Eq. (9.2.35) integrates structure factors according to the interaction relationship indicated in the Standard Specification for Concrete Structures-1991 “Design”. In this case, differences from the conventional formula are small even when γ_i is 1.15. As γ_i becomes smaller than 1.15, the new formula becomes even more conservative than the conventional formula.

Although these equations are for the case where reinforcement yields, they can be applied to the case where concrete crushes before the reinforcement yields.

(6) Eq. (9.2.36) expresses the linear interaction relationship between torsion and shear, and also is consistent with the concept of the negligible torsion limit.

(7) If the wall thickness of a box section becomes smaller, the torsional deformation and capacity becomes different from those of a thick wall. If the wall thickness is thin, the shear flow theory which assumes uniform torsional shear stresses in the direction of wall thickness can be applied. In this case, in-plane shear forces due to torsional shear flow act in each wall composing the box section. Therefore, calculating the in-plane shear capacity of each wall, the torsional capacity of a box section is obtained. When the in-plane shear capacity is obtained in accordance with Section 9.2.2.4, the in-plane shear capacity, V_{od}, is the smaller value of T_{xyd} and T_{yyd}. The value of γ_b in the calculation of T_{xyd} and T_{yyd} may be chosen as 1.30. If the precision of Eq. (9.2.37) is sufficiently confirmed from the torsional loading tests of large-scale box sections, and others, the value of γ_b may be reduced to about 1.15.

The condition of applying the shear flow theory is that there is no out-of-plane deformation in each wall surface. Therefore, when the wall is thin, such measures as installing more than one inner partition wall may be necessary so that out-of-plane deformation is constrained.

When a box section is subject to shear forces and torsional moments at the same time, or is subject to flexure and torsional moments at the same time, safety examination may be made in a similar manner as with a rectangular section.

9.3 Examination of Safety for Fatigue

9.3.1 General

(1) Examination for performance of a structure in fatigue shall be carried out when the ratio of variable loads to total loads is large, or the structure is subjected to a large number of loading cycles.

(2) Safety of a beam under fatigue load shall be examined for flexural moment and shear force.

(3) Safety of a slab under fatigue load shall be examined for flexural moment and punching shear.

(4) For a column, examination of safety under fatigue loading is not required in general. However, when the applied flexural moment or axial tensile force is large, examination for fatigue shall be carried out in a manner similar to that for beams.

[Commentary] (1) Fatigue failure of a material in a structure will directly affect the safety of the structure. In Section 9.3, standards of verification of safety of a structure relating to fatigue failure of the material in the structure are provided, among the performance requirements of a structure for fatigue. Since, in most cases, serviceability of a structure is assured until the fatigue failure, safety of the structure for fatigue load and environmental attack during the design life has only to be examined in general. When the examination of serviceability for fatigue is required, an appropriate method based on experimental results, etc. should be employed.

(2) and (3) Examination of the limit state of failure of section in fatigue is required, in general, primarily for fatigue failure of main and shear reinforcement subjected to repeated tensile force. However, in special cases examination of concrete under fatigue may be necessary. Such cases include those where lightweight aggregate concrete is used or the concrete is wet, as described in the Commentary of Section 5.2.2. Such concretes may be relatively weak in fatigue compared with normal concrete and dry concrete, and shall, therefore, be examined for fatigue. Further, shear capacity of a member in water may be lower. Therefore, for a member submerged in water, in addition to stress in shear reinforcement, examination of its fatigue capacity as a member without shear reinforcement shall also be carried, even though shear reinforcement may actually be provided in the member.

9.3.2 Design variable force and equivalent number of cycles

Irregular variable member forces may be divided into sets of individual variable member forces and substituted for actions having an equivalent number of cycles, N, on the basis of Minor's hypothesis, to determine the design variable member force, S_{rd}.

[Commentary] The variable force acting on the cross-section of a member on account of the application of different variable loads varies in a complex manner. This force could be considered as a sequence of individual variable member forces $(S_{r1}, S_{r2}, \cdots, S_{rm})$ acting for different number of cycles $(n_1, n_2, \cdots, n_m)$, respectively. Various methods have been proposed for this purpose, such as the range-pair counting method. Depending on the characteristics of the variable loads applied, an appropriate method should be chosen.

The equivalent number of cycles, N, for the design variable member force, S_{rd}, can be obtained using Minor's hypothesis for the sets of the individual variable member force. This hypothesis, also denoted as a direct damage rule, is based on cumulative damage and assumes that fatigue failure occurs on the basis of the sum of the damage caused by individual loads as described in Eq. (C.9.3.1).

$$\sum_{i=1}^{m} \frac{n_i}{N_i} = 1.0 \tag{C9.3.1}$$

For a member whose capacity of cross section is governed by fatigue strength of the steel reinforcement, the equivalent number of cycles, N, for the design variable member force, S_{rd}, can be evaluated using Minor's hypothesis if the gradient of the $S-N$ line is given by such as Eq. (5.3.2). The equivalent number of cycles, N, for flexural moment (M_{rd}, M_{ri}) can be calculated by Eq. (C.9.3.2). The equivalent number of cycles, N, for shear force (V_{rd}, V_{ri}) can be calculated by Eq. (C.9.3.3).

$$N = \sum_{i=1}^{m} n_i (M_{ri} / M_{rd})^{1/k} \tag{C9.3.2}$$

$$N = \sum_{i=1}^{m} n_i \left[\frac{V_{ri}}{V_{rd}} \cdot \frac{V_{ri} + V_{pd} - k_2 V_{cd}}{V_{rd} + V_{pd} - k_2 V_{cd}} \right]^{1/k} \tag{C9.3.3}$$

where, k : a constant representing gradient of $S-N$ line of steel reinforcement, may be estimated in accordance with such as Section 5.3.2 (2),

V_{pd} : design shear force due to permanent load,

V_{cd} : design shear capacity without shear reinforcement

k_2 : a coefficient representing the influences of variable loads, which may be set to 0.5 in general.

For a member whose capacity of cross section is governed by the fatigue strength of concrete, the equivalent number of cycles, N, for the design variable member force, S_{rd}, can be evaluated by Eq.(C9.3.4) using Minor's hypothesis if the design fatigue strength of concrete is given by Eq.(5.2.7).

$$N = \sum_{t=1}^{m} n_i \cdot 10^{\frac{K}{k_{1f} S_d}(S_{ri} - S_{rd})} \tag{C9.3.4}$$

where S_d : member force when stress reaches f_d

σ_p : stress due to permanent load,

k_{1f}, f_d, K : defined in Section 5.2.2 (3).

9.3.3 Fatigue strength of concrete members without shear reinforcement

(1) Design shear fatigue capacity V_{rcd}, of reinforced concrete beam members without shear reinforcement, may be computed using Eq. (9.3.1).

$$V_{rcd} = V_{cd}(1 - V_{pd}/V_{cd})\left(1 - \frac{\log N}{11}\right) \tag{9.3.1}$$

where N : fatigue life,

V_{cd} is given by Eq. (9.2.4).

(2) Design punching shear fatigue capacity, V_{rpd}, of reinforced concrete slabs as plane member may be computed using Eq. (9.3.2).

$$V_{rpd} = V_{pcd}(1 - V_{pd}/V_{pcd})\left(1 - \frac{\log N}{14}\right) \tag{9.3.2}$$

where, V_{pcd} is derived using Eq. (9.2.11).

[Commentary] Although shear reinforcement is not usually provided for a footing and a retaining wall, these structures rarely fail in shear fatigue. However, in cases when fatigue does become an issue, examination shall be performed using Eq. (9.3.1) or Eq. (9.3.2). When applying these equations, it is recommended that the number of cycles is less than approximately 2×10^6.

Fatigue capacity may be computed using Eq. (9.3.1) or Eq. (9.3.2) even in cases when a member is subjected to fatigue load underwater. In this case, however, the design compressive strength of concrete, f'_{cd}, necessary to calculate V_{cd} or V_{pcd}, shall be determined from results of experiments carried out in water.

The mechanism of contribution of shear reinforcement to shear capacity of a member subjected to fatigue load underwater is not sufficiently clear. Thus, in such cases, it is recommended that to be on the safe side, the fatigue shear capacity without shear reinforcement be adopted as the design value.

Recent research has revealed that punching shear fatigue capacity is considerably reduced under the action of repeated moving loads, such as a slab of highway bridges. In such cases, fatigue capacity shall be estimated by appropriate methods, including laboratory tests. It has also been found that the fatigue capacity of the plane member is reduced to a greater extent if its surface is wet. Equation (9.3.2) in the Specification shall not be applied in cases of moving loads, since it was derived from experimental results in which loading point was fixed.

CHAPTER 10 VERIFICATION OF SERVICEABILITY

10.1 General

(1) It shall be verified that concrete structures satisfy the required serviceability during the design life.

(2) As a general rule, in the verification of the serviceability of a structure, it shall be verified that neither the structure nor any of its members reaches the serviceability limit state under design loads.

(3) In general the serviceability limit states should be represented by indices such as stress, crack, displacement, deformation and vibration, and the examination for the limit states shall be carried out using an appropriate method.

(4) In limit state verification, compressive stress in the concrete and tensile stress in the reinforcement due to bending moment and axial force shall be limited appropriately, and the comfortability and other serviceability-related functions shall be verified by an appropriate method.

[Commentary] Structures or members are required to preserve sufficient functions suitable for the purpose of their usage during their design life. Serviceability limit states are required to be determined to suite the purpose of use of structures and shall be checked by appropriate methods of which accuracy and applicable range are clarified. In Chapter 10, standard methods to verify serviceability of structures are presented based on the assumption that they satisfy required the durability verification specified in Chapter 8, the verification of early-age cracking specified in Chapter 12, and constructability specified in the Standard Specifications for Concrete Structures, “Materials and Construction”, and therefore material deterioration during the design life is negligible.

As far as the serviceability, its limit states, and verification index are concerned, various limit states may be considered. In general, however, only the serviceability limit states listed in Table C4.1.1 may be examined. In this chapter, verification for appearance, vibration, displacement, and deformation associated serviceability, water resistance, and damage due to fire is explained. Other functions, which are not mentioned in this chapter, may be verified by appropriate method, if necessary.

Although the relationship of specific serviceability-related performance items with compressive stress in the concrete and tensile stress in the reinforcement due to bending moment and axial force is not indicated clearly, this chapter examines them as a basis for serviceability verification, taking past design examples, etc., into consideration.

10.2 Limit Value of Stresses

Compressive stress in concrete and tensile stress in reinforcement due to flexural moment(s) and axial force(s) shall not exceed the limiting values given in (1) and (2), respectively.

(1) Limiting value of flexural compressive stress and axial compressive stress in concrete under permanent load shall be 0.4 f'_{ck}, where f'_{ck} is the characteristic compressive strength of concrete. In cases when concrete is permanently subjected to multi-dimensional restraint, the limiting value of stress may be increased in accordance with the state of stress in multi-dimensional compression. However, such increment shall not exceed 10% and 20% in the case of two-dimensional and three-dimensional restraint, respectively.

(2) Limiting value of tensile stress in reinforcement shall be f_{yk}, where f_{yk} is characteristic yield strength of reinforcement.

[Commentary] (1) Maximum limit on the compressive stress of concrete is introduced in order to avoid excessive creep strains and cracks in the longitudinal direction due to compressive forces. These limiting values have been determined considering the modulus of elasticity of concrete given in Table 5.2.1 in Section 5.2.5 and the conditions in which the provisions for the creep coefficient of concrete described in Section 5.2.9 can be adopted. However, in cases when the modulus of elasticity and creep coefficient of concrete are determined differently, the limiting value of compressive stress of concrete may be determined in a manner independent of the provisions of this section. The standard methods to examine safety, serviceability and seismic performance of structures that are shown in the Specification are based on the provisions in this clause. In case that these limit values are not adopted, safety and seismic performance shall be examined taking the effect of nonlinear creep strain into account.

The limiting value of compressive stress in concrete may be increased for concrete confined three-dimensionally, such as concrete in steel-concrete composite columns, and concrete subjected to permanent earth pressure or water pressure, such as concrete in underground structures. This is because compressive strength of the concrete increases and creep decreases depending upon the degree of confinement. It has been reported so far that static strength of concrete increases by 20-30% under two-dimensional confinement and 30-40% under three-dimensional confinement by steel. However, from safety considerations, the permitted increment in strength should be limited as shown in this section.

(2) No limits may be imposed on the tensile stress in the reinforcement when carrying out the examination for the cracking in concrete and the fatigue of reinforcement. However, in cases when the tensile stress exceeds the elastic limit, some difficulties, such as the assumptions made during structural or stress analysis no longer holding, may be encountered. Therefore, the tensile stress in the reinforcement shall be smaller than the yield stress. This is why the maximum values of permanent loads and variable loads under service conditions must be used as design loads.

10.3 Verification Related to Appearance

10.3.1 General

(1) In the verification of the appearance-related serviceability of a structure, design response values shall be calculated in accordance with Chapter 7, and design limit values shall be determined according to the environment of the structure or its members.

(2) The appearance-related serviceability of a reinforced concrete structure or prestressed and reinforced concrete structure is verified by using crack width as a verification index may be done in accordance with Sections 10.3.2 and 10.3.3. The verification of the appearance-related serviceability of prestressed concrete structures using crack width as a verification index may be omitted.

(3) When appearance-related serviceability is verified by using displacement or deformation as an index, Section 10.5 shall be followed.

[Commentary] (1) Concrete structures must not make people uneasy or uncomfortable or obstruct the use of the structures with cracks, dirt, etc., on their surfaces.

This specification describes a verification method to make sure cracks do not mar the appearance of concrete structures. Verification with respect to other factors contributing to appearance degradation such as dirt, etc., must be done separately by an appropriate method.

(2) In the verification related to cracks from the viewpoint of appearance, crack width or stress may be used as a verification index.

There are many kinds of cracks in concrete structures. This clause deals with cracks due to structural loads, namely flexural moment, shear force, torsional moment and axial load.

The verification related to cracks due to loading needs to be performed under service condition. If the structural system changes during construction, member forces need be calculated taking into account the effect of these changes.

Since cracks may be also induced on account of the materials used, methods of construction, etc. the effect of these on cracking should be accounted for during design. For example, it is noted that thermal cracks due to heat generated during hydration of cement in concrete, or, cracks formed due to shrinkage of concrete during construction, may greatly affect the structure. Concerning this, Chapter 12 must be followed. Crack control measures to be taken at the design stage include crack width control reinforcement, crack inducing joints and crack control prestressing. When designing crack control measures, it is good practice to refer to previous studies and past examples. For a prestressed concrete structure in which cracking is not permissible under normal service conditions, verification concerning flexural crack width may be omitted if the extreme fiber tensile stress in the concrete under permanent loads does not become a tensile stress and if the extreme fiber stress in the concrete under a combination of permanent loads and variable loads is not greater than the design flexural crack strength and the required tension reinforcement is in place.

10.3.2 Flexural cracks

In flexural crack verification, the flexural crack width at the concrete surface shall be taken as a design response value, and the limit value of the maximum crack width at the concrete surface from the viewpoint of appearance shall be taken as a design limit value.

[Commentary] (1) In crack width verification for reinforced concrete structures and prestressed and reinforced concrete structures that permit the occurrence of cracking under normal service conditions, the basic rule is to use crack width as a verification index. In this case, in view of the fact that appearance evaluation is affected by psychological factors, the limit value of crack width needs to be a value that does not pose any appearance-related problem. According to past examples and experience, usually the limit value may be set at about 0.3 mm. In this case, the maximum value of flexural crack width at the concrete surface may be calculated from Eq. 7.4.4. The earlier the age at which concrete cracking occurs, the greater the crack width at the concrete surface becomes. The recommended method when verifying the appearance-related performance of a structure located in a dry environment by calculating crack width from Eq. 7.4.4 is to determine the value of ε'_{csd} in accordance with Section 5.2.8, assuming construction steps such as placing the concrete for the structure and removing the formwork and false work and taking into account measured values of shrinkage strain and the ages at which cracking occurs. In general, in the case of concrete with a shrinkage strain as determined by the JIS test method of 1000×10^{-6} or smaller, if cracking occurs at ages of 30 to 200 days, ε'_{csd} may be assumed to be about 450 to 300×10^{-6}. If shrinkage-compensating concrete is used, ε'_{csd} may be reduced in view of the steel stress reduction due to the crack control effect and expansion of the shrinkage-compensating concrete.

In the case of a reinforced concrete structure, if the extreme fiber tensile stress in the concrete with a fully effective cross section is not greater than flexural crack strength, verification concerning flexural crack width may be omitted by making the stress in the tension reinforcement under permanent loads not greater than the limit value of steel stress, s_{l1}, in accordance with Chapter 8. In this case, to apply the limit value of steel stress under permanent loads, the values for normal environmental conditions may be used.

10.3.3 Shear cracks and torsion cracks

(1) As a general rule, verification for shear cracking and torsion cracking shall be done by using shear or torsion crack width at the concrete surface as a design response value and the limit value of the maximum crack width at the concrete surface determined from the viewpoint of appearance as a design limit value.

(2) Verification for shear cracking and torsion cracking may be omitted if the requirements described in Chapter 8 are met.

[Commentary] The mechanism of initiation and propagation of shear cracks and torsion cracks is different from that of flexural cracks, and is not clear enough. Therefore, it is difficult to calculate crack width in the present technical level. By limiting the stress in shear reinforcement and torsion reinforcement, precise examinations may not be required.

10.4 Verification for Vibration

When vibration is considered in the verification of serviceability, examination for vibration caused by variable loads shall be carried out using appropriate methods to ensure that functions and serviceability of the structure are not impaired.

[Commentary] Vibration are rarely cause of problems in concrete structures. However, in cases when the time period of variable loading is close to the natural period of members, resonance may result. This may bring lead to an uncomfortable environment during use and cause cracks in the structure. In such cases, it is advisable to take some countermeasure, such as altering the natural period of the member by changing the dimension of members, etc.

10.5 Examination for Displacement and Deformation

(1) When displacement and deformation are considered in the verification of serviceability, it is required to follow the method in this section.

(2) Short-term displacement and deformation, and long-term displacement and deformation shall be considered separately. Short-term displacement and deformation refer to instantaneous displacement and deformation of the structure or member upon application of load(s). Long-term displacement and deformation include short-term displacement and deformation and additional displacement and deformation under sustaining loads.

(3) Short-term displacement and deformation, and long-term displacement and deformation of the structure or member shall be smaller than the permissible displacement and deformation.

(4) Permissible displacement and deformation of the structure or member shall be determined considering the type and the purpose of use of the structure, the type of load(s), etc.

[Commentary] (1) Displacements and deformations, in general, are related to maintaining functions and serviceability for safety and comfort with moving traffic, preventing damages due to excessive displacements and deformations, and maintaining esthetics of structures. Considering the purpose of use of a structure, enough stiffness and appropriate camber should be provided, and support need to be selected adequately. It is advisable to examine the influences of gasp kinks between members and expansions/shortenings of members if necessary. For structures resting directly on the ground, it is advisable to examine the serviceability limit state of vertical support using an appropriate method, in cases when such an examination is specially required.

(2) There are two types of displacements and deformations. One is short-term displacements and deformations caused instantaneously at the time of application of load. The other is additional displacements and deformations caused by shrinkage and creep of concrete due to permanent loads. Long-term displacements and deformations are defined as the sum of the short-term and the additional displacements and deformations.

(3) When the span length of a member is much shorter than the height of its cross section, examination of displacements and deformations may be omitted. However, if long-term displacements and deformations due to shrinkage or creep of concrete cannot be neglected, there is a need to examine the long-term displacements and deformations.

(4) Excessive displacements and deformations of structures or members cause an uncomfortable feeling when using structures and reduce its functionality and good appearance. In order that such a situation is avoided and the structures are used in a good condition, it is necessary to determine the permissible displacements and deformations, considering the type and planned use of the structure, and the type of loads.

Permissible displacements and deformations, in general, are determined for the ordinary serviceability limit states. However, if necessary, they may be determined for limit states in extraordinary cases such as earthquakes.

10.6 Verification for Water-Tightness

(1) In the verification for water tightness, the performance of concrete structure shall not be impaired by water or moisture permeation.

(2) The verification of water-tightness must be carried out on the each part of structure with the verification index to be the transmissibility. The performance of the concrete structure in terms of water tightness may be secured using appropriate surface treatment. In such cases, the effectiveness of the treatment shall be evaluated using appropriate methods.

(3) In the case of the verification for water tightness by water flow, the verification should be carried out by ensuring that:

$$\gamma_i \frac{Q_d}{Q_{\max}} \le 1.0 \qquad \textbf{(10.6.1)}$$

where,

γ_i **: a constant, representing the importance of the structure. In general, it may be taken as 1.0, but may be increased to 1.1 for important structures.**

$Q_{\max}$ **: allowable flow rate per unit time (m^3/s).**

Q_d **: designed flow rate per unit time (m^3/s).**

$$Q_d = \gamma_{pn}\left(K_d \cdot A \cdot \frac{h}{L} + Q_{cjd}\right) \qquad \textbf{(10.6.2)}$$

where,

K_d **: design value of permeability coefficient of concrete in structure (m/s).**

$$= K_k \cdot \gamma_c$$

K_k **: characteristic value of the permeability coefficient of concrete (m/s).**

A **: total surface area of concrete through which water may penetrate (m^2)**

h **: difference in the water levels between the interior and exterior surfaces of the structure (m).**

L : **expected length of water permeation. In general, L may be taken as the designed thickness of member.**

Q_{jcd} : **design value of the flow rate from crack or joint (m^3/s)**

γ_{pn} : **safety factor to account for the variation in the designed transmissibility,** Q_d **. Normally, it should be taken as 1.15.**

γ_c : **concrete material factor. In general it may be taken as 1.0.**

[Commentary] (1) Concrete structures requiring water tightness is the structures greatly affected by the permeation of water on their structure safety, durability, functionality, maintenance, appearance, etc. The examples are storage facilities, underground structures, hydraulic structures, water tanks, sewage facilities, tunnels, etc. It is also known that the leaching of calcium from concrete over a sustained period of time can also impair the required performance of concrete structure. The provisions of this Section may be ignored if water-tightness is not one of the required performance parameters.

(2) The verification of water-tightness must be carried out not on the structure in a whole but on each part, with the only verification index to be the transmissibility. In order to secure the water tightness of a concrete structure, measures such as using a waterproofing sheet especially in part(s) requiring water tightness, inducing crack at pre-designed locations (joints, and then taking suitable water-proofing measures for such locations may also be considered. It should, however, be noted that since the effectiveness of such measures could vary largely depending on the materials used (during construction), the methods of construction adopted, etc., it is important that the details of such measures are worked out only after appropriate evaluation and consideration of various factors, including the structure maintenance plan.

(3) In the case of tank structures, the reduction rate of stored liquid in the tank can be treated as the verification index aiming at the whole structure. For example, the water level in the PC egg-shape digester (sewage digestion tank) must vary just within 5mm from the specified water level in 24 hours. Permeation amount within 0.1% of water in water tank during circulation can be disregarded. In addition, the allowable water permeation is governed by the structure function, the processing capacity of drainage system and the evaporation of water from structure surface.

The water-tightness of a concrete structure is related to not only the water-tightness of the normal concrete used in the structure, but also the discontinuity arising at particular positions such as cracks, joints etc.. In general, the extent of water permeation through a crack or vertical construction joint is much larger than that of sound concrete. Therefore, it is desirable to avoid the occurrence of cracks when the water tightness of structure is required. It should be noted that the provision of adequate and appropriate steel reinforcing bars is an effective means of controlling crack with and preventing cracking in concrete. The use of expansive material serves a similar purpose. It may be noted that carrying out an appropriate treatment for construction joints and installing water-sealing plates in the direction perpendicular to the joint are very effective in controlling water permeation through these surfaces.

The transmissibility through sound parts shall be evaluated based on the permeability coefficient using Darcy's law. The water permeability coefficient is a constant of proportionality to vary linearly with the hydraulic gradient. This coefficient may be obtained using the method given by the United States Reclamation Bureau (referred to as the "Output Method"), or, the method given in DIN1048 (referred to here as the "Input Method"). The permeability coefficient can be estimated indirectly from its empirical relationships with water-cement ratio or with compressive strength, or from characteristic values of micro-pore structure such as the coefficient of air permeability or the total porosity of concrete. In such a case, a suitable factor of safety should be used, depending upon the accuracy of the method(s) used. The following equation which expresses relationship between water-cement ratio and permeability coefficient is obtained from previous research results. The safety factor of 1.0 may be selected for this equation.

$$\log K_p = 4.3 \cdot W/C - 12.5 \qquad \text{(C 10.6.1)}$$

Where, w/c is water-cement (binder) ratio.

In the event that water penetration through discontinuous surfaces such as cracks and joints cannot be prevented, the verification for transmissibility shall be carried out with duel consideration on the influence of crack or joint as main items of verification. When the water permeation through structure members is considered a one-dimensional process, Eq. (10.6.2) shall be used to evaluate the total transmissibility through sound parts and from cracks or joints. Appropriate permeability tests shall be carried out for obtaining the transmissibility through cracks and joints.

Prevention of cracking or permissible crack width for water-tightness shall meet the requirements of examination of water-tightness. In general, Table C10.6.1 may be used as a reference depending upon the required level of water-tightness and dominant force acting on the cross-section.

Table C10.6.1 Reference values for permissible crack width for water-tightness (mm)

Required level of water-tightness		High	Normal
Dominant force acting on the cross-section	Axial tension	----1)	0.1
	Flexural moment 2)	0.1	0.2

1) Concrete stresses due to stress resultant should be in compression at whole sectional area. Minimum compressive stress should be greater than 0.5N/mm^2. In cases when detailed analysis is carried out the value may be determined differently.

2) Under the action of reversed cyclic loading, the permissible crack width should be determined in a manner similar to that under axial tension.

Here, past test data used to set the permissible crack width have all been obtained from members with fully penetrating cracks. In these tests, it has been reported that the amount of permeating water is very little when the applied water pressure is below 0.9N/mm^2 and the crack width is less than 0.1mm. In Table C10.6.1, it has been assumed that that water-tightness can be easily achieved under flexural moment because part of the section is in compression.

10.7 Verification for Fire Resistance

(1) The required performance of a concrete structure shall not be impaired by the fire.

(2) In case when the concrete meets criteria for fire resistance, the structure may be assumed that its performance will not be impaired on account of fire.

[Commentary] (1) It is important to ensure that the performance of normal civil engineering structures during their designed service life will not be affected by the deterioration of concrete or the quality degradation of reinforcing materials on account of fire.

(2) Since the fire resistance of concrete structures is basically determined by that of the cover concrete, it is considered sufficient by ensuring that the requirements for the fire resistance of the cover concrete are satisfied. The verification for the fire resistance of cover concrete shall be carried out at the mix design stage as stipulated in “Construction”. Prestressed concrete structures and members may degrade rapidly, so the investigation on the fire resistance must be carried carefully once the rapid degradation under fire is anticipated.

In general, when the minimum cover depth is 20mm larger than the cover depth required for normal environment in 8.3.6., the verification for fire resistance can be omitted.

CHAPTER 11 SEISMIC DESIGN

11.0 Notation

θ_m	:	rotation angle of member at the maximum strength
M_m	:	maximum moment
M_n	:	yield moment of tensile reinforcement
η	:	coefficient of decreasing gradient

11.1 General

(1) The verification of earthquake resistance shall be carried out not only to ensure the safety of a structure but also to prevent fatal structural damage that has adverse effects on the daily civil life.

(2) In the verification of earthquake resistance, seismic performance shall be defined as verification limit values to ensure comprehensively for safety against an earthquake and usability and restorability after the earthquake.

(3) In principle, the seismic performance shall be defined not only to ensure the safety of structures for ground motion but also to consider such factors as the ensuring human life, the evacuation, rescue activities and secondary disaster prevention activities, the basic infrastructure and economic activities, the degree of difficulty in restoration and construction cost. Generally, the seismic performance is classified as follows:

(i) Seismic Performance Grade 1: The structure is able to keep the function for an earthquake without restoration.

(ii) Seismic Performance Grade 2: The functions of the structure can be restored in a short period of time after an earthquake.

(iii) Seismic Performance Grade 3: The structural system does not collapse for an earthquake.

(4) The verification of the earthquake resistance of a structure shall be conducted in order to ensure the required seismic performance for the assumed ground motion. In general, verification shall ascertain the follows:

(i) Seismic Performance Grade 1 is applied to Level 1 ground motion.

(ii) Seismic Performance Grade 2 or Seismic Performance Grade 3 is applied to Level 2 ground motion.

(5) Limit values for the seismic design of a structure shall be determined appropriately. In principle, limit values for Seismic Performance Grades 1 and 2 shall be determined taking into consideration the influence of the damage state to structural members on the entire structure, and limit values for Seismic Performance Grade 3 shall be determined taking into consideration the relationship between the stability of the structure and the

load-carrying capacity of members. In general, it may be assumed that the structure satisfies seismic performance requirements if the limit values for members listed below are satisfied.

(i) Seismic Performance Grade 1: yield displacement or yield rotation angle of members

(ii) Seismic Performance Grade 2: shear strength and torsion strength and ultimate displacement or ultimate rotation angle of members

(iii) Seismic Performance Grade 3: shear strength of vertical members and self-weight bearing capacity of a structure

(6) Structural planning shall take into consideration the topographical, geological and geotechnical characteristics of the construction site and the influence on adjacent structures so that the structural system can have the required seismic performance. In general, the following should be taken into consideration in structural planning:

(i) Coupling with adjacent structures should be taken into consideration.

(ii) Attention should be given to the center of stiffness of the structure and the center of loads so that they coincide as much as possible.

(iii) The plastic hinge regions in members should be designed to have sufficient ductility.

(iv) Seismic isolation and structural response control technologies are taken into consideration.

(v) Soil flow resulting from liquefaction should be taken consideration in design not to have adverse effects on the structure.

(vi) In the case of an underground structure, consideration should be given to prevent water infiltration into the structure and joints, leakage of the content of the structure, uplift due to soil liquefaction, and so on.

(vii) If the location of an active fault is clearly known and a structure is constructed across the fault, not only the structure should have safety for ground acceleration but also system redundancy should be taken into consideration.

(viii) The safety of the people around the structure during an earthquake should be taken into consideration.

[Commentary] (1) In general, infrastructures are base of life of people and social and production activities. When deciding on seismic performance for a structure, it should be considered not only the dynamic response of the structure according to the magnitude of assumed earthquakes, but also the post-quake restorability. In connection with a strong earthquake whose probability of occurrence for the service life time is very low, it is necessary to prevent fatal structural damage, but it is also necessary, from the view of social and economic activities, to minimize functional degradation of the structure so that the daily civil life, social and production activities can be restored as soon as possible.

(2), (3) Seismic Performance Grade 1 refers to the performance that ensures residual deformation of the structure is kept within a small deformation range. It may be thought that this seismic performance requirement is satisfied if the dynamic response of the members is before their yielding.

Seismic Performance Grade 2 refers to the performance ensuring that load-carrying capacity does not decrease. In general, it may be thought that this seismic performance requirement is satisfied if none of the structural members fails in shear and the response displacement of the members does not reach the ultimate displacement. In special cases, residual displacement may be specified. In such cases, it is necessary to calculate residual displacement by an appropriate method and verify that the calculated values are within tolerance. In general, the residual displacement of the foundation including the ground tends to be larger than the residual deformation of the structure. An appropriate evaluation method of the residual displacement of the foundation is necessary. Structures in which seismic force acts mainly as in-plane force such as wall structures cannot be expected to have large deformation capacity. In seismic design of such structures, it is necessary to have sufficient shear capacity in order to prevent brittle failure.

Seismic Performance Grade 3 refers to the performance that ensure that even if the structure is difficult to repair, the structural system does not collapse because of the self-weight of the structure, soil pressure and water (fluid) pressure, and so on. Generally, a concrete structure satisfies Seismic Performance Grade 3 if its members are sufficiently safe from shear failure. Depending on the type of structure, however, the displacement of the structural system may become excessively large and the deformation of the members due to the self-weight and additional moment increase until collapse. It is necessary to verify by an appropriate method for such case.

The terms "repair" and "strengthening" used in this chapter indicates the purpose of restoration work carried out after a structure is damaged. "Repair" means restoration work carried out to restore the original state, and "strengthening" means restoration work carried out so that structural characteristics such as strength and deformation capacity satisfies seismic performance requirements.

(4) When verifying the earthquake resistance of a structure, it is necessary to define seismic performance requirements for a structure corresponding to the magnitude of ground motion and its return period. Such requirements must be determined in view of such factors as the purpose and functions of the structure, the degree of contribution to or influence on the local community and society, asset value, the degree of influence of damage on the surroundings, the availability of alternatives and the degree of difficulty in restoration.

The verification, with respect to the combinations mentioned above, is applicable for ordinary civil engineering structures. For special structures, for example, LNG in-ground storage tanks, it is recommended that Level 2 ground motion is further classified into two levels as shown below for verification purposes:

i) Strong ground motion the probability of occurrence for the service life time is relatively low

ii) Extremely strong ground motion the probability of occurrence for the service life time is very low

(5) In order to verify the seismic performance of a structure, it is necessary to determine the limit values of response that ensure the predetermined seismic performance. Although the seismic performance of a structure depends on the structural characteristics of its members, the seismic performance requirements specified for the structure do not need to be applied as are to structural members. A two-layer rigid-frame pier, for example, does not collapse even if the intermediate level beams have failed in shear. Since the relationship between the seismic performance of a structure and the damage level of its members is usually unknown, limit values are set for structural

members described above.

(i) If the reinforcement does not yield during an earthquake, usually Seismic Performance Grade 1 is satisfied.　On the assumption that shear failure does not occur, therefore, it is required that the yield displacement or yield rotation angle of structural members is used as a limit value for Seismic Performance Grade 1.　Usually, the concrete stress does not yet reach the compressive strength before the reinforcement yielding.　If, however, the amount of reinforcement is large, it may be necessary to use the compressive strength of concrete as a limit value.　If two structures are located close together so that they might crash with each other or if it is necessary to check whether passing vehicles are safe, it is recommended to use the limit displacement determined from the viewpoint of functional protection.　In general, in cases where a structure is required to have Seismic Performance Grade 2 or Seismic Performance Grade 3 against Level 2 ground motion, if the structure has the safety requirements with respect to shear force, then it is safe against shear force even under the Seismic Performance Grade 1 criteria.　Usually, therefore, the verification of safety against shear force under the Seismic Performance Grade 1 criteria is not necessary.　However, satisfying Seismic Performance Grade 2 against Level 2 ground motion, it is necessary to verify the shear strength for some of members if these are to satisfy the Seismic Performance Grade 1 criteria.

(ii) Concerning Seismic Performance Grade 2, it is required that limit values can be set appropriately within the range beyond the yield displacement until reaching the ultimate displacement while preventing shear failure and torsion failure, in view of the degree of difficulty in repairing the structure and the influence of the damage to the structural members, in order to restore the functions of the structure in a short period of time after an earthquake.

This means that in order to achieve Seismic Performance Grade 2 (the functions of the structure can be restored in a short period of time after an earthquake), the designer must determine limit values appropriately within the range beyond the yield displacement until reaching the ultimate displacement, taking into consideration the damage conditions of the structures and the difficulty of repair.

Many experimental tests have shown that the load-carrying capacity decrease clearly corresponding to the spalling of cover concrete.　Moreover, the damaged members can be repaired by such as crack grouting in many cased before the spalling of cover concrete.　However, it is difficult to repair the damaged member after the spalling of cover concrete.　In the case of an ordinary column, spalling of cover concrete tends to occur at a displacement close to the ultimate displacement.　This indicates that repair can be done in a short period of time, if the limit value is defined as the displacement related to near the maximum strength.

For the members, such as the bottom of bridge piers, underground beams and piles, members supporting overburden loads such as upper beams of rigid-frame viaducts and girders of prestressed concrete bridges, cannot be inspected and repaired in a short period of time after an earthquake.　In this case, the limit values for Seismic Performance Grade 2 shall be defined before the maximum strength point.

(iii) In the case of a concrete structure, in general, if vertical members are sufficiently safe from shear failure, the structure satisfies Seismic Performance Grade 3.　However, there are cases that the displacement of the structural system becomes so large because of the deformation of members due to their weights and additional moment, and structure system fails.　Using dynamic analysis is necessary to verify that the structure does not collapse because of its own weight.

(6) In order to satisfy the required seismic performance as a structural system, seismic design of a structure should generally take the following into consideration:

(i) If the dynamic characteristics, type of foundation structure, ground conditions, and so on are different between adjoining structures, the response of one structure may affect that of the other structures and unexpected damage may be caused. Structural planning needs to take this into consideration.

(ii) It is important that the center of stiffness of the structure coincide with the center of load action as much as possible to minimize the horizontal torsion of the structure. It is necessary to enhance the seismic performance, if the both of center cannot coincide.

(iii) In order to prevent excessive deformation and stress concentration and prevent the collapsing of the structure, it is important to give the sufficient ductility by reinforcing with a sufficient amount of hoop reinforcement in plastic hinge regions. To be more specific, it is recommended to have two times shear capacity to flexural capacity in plastic hinge regions. From the viewpoint of seismic performance, statistically indeterminate structures are superior in terms of deformation capacity to statically determinate structures because forces can be redistributed in statically indeterminate structures.

(iv) In order to enhance the seismic performance of a structure, it is recommended to consider the use of seismic isolation or structural response control technologies. Seismic isolation enhances the damping of a structure with a relatively short period of vibration. A structural response control system with an energy absorption mechanism enhances the damping of a structure with a long period of vibration.

(v) It is important to consider that soil flow caused by liquefaction does not subject to the structure. Soil liquefaction and the resultant lateral flow of soil greatly affect the earthquake resistance of underground structures. In seismic design, it is necessary to carefully examine the stability of soil.

(vi) The displacement and deformation behavior and stability of the surrounding ground are important in the seismic design of underground structures. For a structure with a large cross section, the three-dimensional displacement distribution, that is the distribution not only in a plane but also in the direction of depth, is important. For a buried pipeline with a small cross section, the displacement distribution along the pipeline is important. It is necessary, therefore, to fully understand the seismic response of the surface layer of the ground. In order for an underground structure to maintain the required seismic performance, it is recommended to consider the use of structures and materials designed for enhancing flexibility.

(vii) For the surface structure, if the location of an active fault is clearly known, constructing continuous structures and increasing the strength may be effective for forced displacement caused by the fault. For the underground structures, large cross sections, double-wall structures, flexible structures, and isolating structures from internal facilities may be effective. If the forced displacement is so large as to introduce these technologies, it is necessary to consider the system planning such as using alternative structures in order to minimize functional degradation in civil life.

(viii) Depending on the dynamic behavior of a structure, the safety of the people using the structure and the people living near the structure may be threatened. For example, even if damage to a bridge structure is not serious, large deformation or displacement of the structure may cause such problems as derailment of trains or falling of vehicles. If vehicles fall, people living near the structure may be endangered. In structural planning, it is necessary to take into consideration methods for preventing such consequences.

11.2 Definition of Limit Values

(1) Yield displacement of member can be given at the yielding of center of tensile reinforcement.

(2) Ultimate displacement of member can be given at the maximum displacement which is greater than yield strength.

(3) Shear strength and torsion strength are given by referring to Chapter 9.

[Commentary] (1), (2) Capacity of displacement and strength are given by referring to Chapter 9. For the multi arrangement of reinforcement, the reinforcement yielding from the outside sequentially, and the clear yielding of member does not appear on the load-displacement relationship. The yield displacement can be given as described above.

The load-carrying capacity of flexural member decreases after the spalling of cover concrete and buckling of longitudinal bars. As the load-carrying capacity undergo the yield strength, serious damage as crush of the core concrete and fracture of longitudinal bars occur. The ultimate displacement is defined as the maximum displacement greater than yield strength. Typical load-displacement relationship of member is shown in Fig. C11.2.1. If the load-carrying capacity has sustainable performance in the cyclic large deformation under the yield strength, the ultimate displacement can be defined as the other limit value.

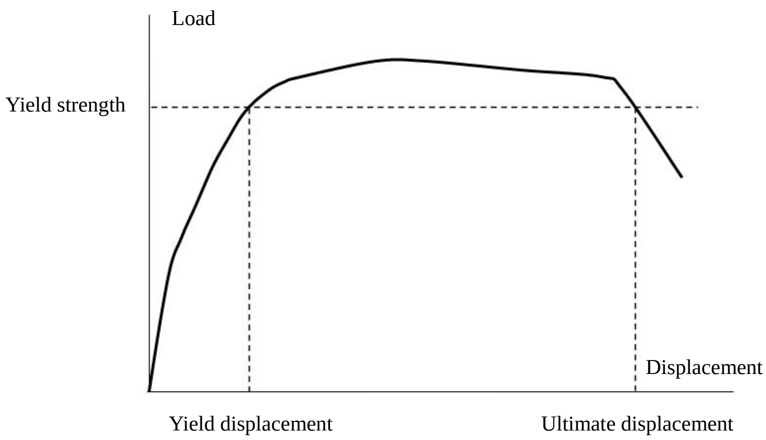


Fig. C11.2.1 Limit values of member

Calculating by beam element method, response displacement is related to the rotation angle. The yield rotation angle of member can be given by Eq. (C7.2.1). The ultimate rotation angle has to be evaluated for each member by appropriate method. In general, the ultimate rotation angle can be given by Eq. (C11.2.1).

$$\theta_n = \theta_m + \eta\{1-(M_n/M_m)\} \tag{11.2.1}$$

where, θ_m : rotation angle of member at the maximum strength given in Eq. (C7.2.5)

M_m : maximum moment given by referring to Section 9.2.1, where member factor of 1.0 is used.

M_n : yield moment of tensile reinforcement

η : coefficient of decreasing gradient, where 0.1 is used in general.

It is necessary to model the post-peak behavior shown in load-displacement relationship in verification of Seismic Performance Grade 2 and 3. It is difficult to evaluate the post-peak behavior of concrete members, then, the decreasing gradient of skeleton of load-displacement relationship is assumed to be constant. In the case of high axial loading or the case that shear capacity is similar to flexural capacity, it is recommended that coefficient (η) is determined as small value.

(3) It is known that the shear strength and torsion strength decrease under cyclic loading in large deformation. However, it is difficult to evaluate the decreasing gradient of these strengths, and the decreasing gradient is considered by increasing member factor as described in Section 11.4.

11.3 Verification

(1) For structural members not significantly affected by torsional moment and those subject to only compatibility torsional moment, the verification for torsion may not be carried out if the member satisfies the requirement of shear capacity.

(2) For the plate members significantly affected by out-plane shear, the verification for in-plane shear, in Seismic Performance Grade 3, may not be carried out.

(3) Verification may be referring to limit values described in Section 11.2.

(4) The verification for self-weight of a structure, in Seismic Performance Grade 3, may not be carried out if the dynamic analysis indicates that a structure does not fail against the earthquake.

[Commentary] (1) Generally, the torsion stiffness of member decreases clearly after the yielding of reinforcement by shear force and flexural moment. If the member has sufficient ductility after yielding of reinforcement, compatibility torsional moment also decreases as well as torsion stiffness. In this case, the verification for torsion may not be carried out.

(2) For members significantly affected by in-plane shear force, for example, the underground storage tank, the out-plane shear action is not corresponding to the main shear resistance region even if the local out-plane shear force is caused. In this case, out-plane shear behavior is usually not brittle, and the verification may not be carried out.

(3) For the analysis of linear members by using finite element method, the verification of that the dynamic response displacement is not exceed the ultimate displacement can be replaced with the verification of that the equivalent sectional stiffness is not exceed the value at the ultimate displacement. In general, it is related to the ultimate displacement when the equivalent sectional stiffness ratio becomes 50% of the initial property. Where, the equivalent sectional stiffness ratio is defined as the average of stiffness ratio of gross concrete section for each member. The equivalent sectional ratio is described in Section 5.2.3.

For the analysis of plate members by using finite element method, the verification of that the dynamic response displacement is not exceed the ultimate displacement can be replaced with the verification of that the compressive strain of plate surface is not exceed the two times strain at the maximum compressive strength. In addition, the verification of that vertical member is not failed in shear can be replaced with the verification of that the compressive strain of plate surface is not exceed the tree times strain at the maximum compressive strength.　Though large strain causes in local region in linear members, the strain in plane and shell members is distributed constantly, moreover, the strain is distributed constantly in thickness of plate.　The strain of plate surface can be used as limit value appropriately.

(4) For the verification of Seismic Performance Grade 3, it is necessary to prevent not only shear failure of member but also structural collapse.　Such verifications may be replaced with the dynamic analysis, if the calculated results show that the structure is not failed and the dynamic response converges after ground motion.

11.4　Safety Factors

Safety factors used for verification related to earthquake resistance shall be determined appropriately according to verification methods, taking into consideration the purpose of safety factors described in Section 4.5.

[Commentary]　Safety factors used for verification related to earthquake resistance may be treated as follows:

For Seismic Performance Grade 1, the yield displacement or the yield rotation angle of a member is used as a limit value.　Because the member receives little damage even if this limit value is exceeded slightly, a safety factor of 1.0 may be used.

For Seismic Performance Grade 2 and Seismic Performance Grade 3, it is necessary to verify that the load-carrying capacity of the structure does not decrease clearly.　In general, if the members of a structure are designed to prevent shear failure and torsional failure, the structure loses very little of its load-carrying capacity even after the members fail in bending, and shows highly ductile behavior.

The dynamic response of a structure is depends on the characteristics of its members as well as ground motion.　The stiffness of members greatly affects response displacement, but when response displacement reaches the yield displacement or flexural strength of a member, shear force corresponding to the yield load or flexural strength of the member acts.　The purpose of the material characteristic values, material factors and member factors used for the verification of safety from member failure described in Chapter 9 is to calculate the lower limit of flexural strength.　If this is used for earthquake resistance verification, the stiffness, yield load and flexural strength of members are underestimated.　Consequently, response displacement is overestimated in response analysis, but shear force is underestimated.　It is generally known that the shear capacity of a member decreases when the member is subjected to reversal cyclic loading in large displacement. In order to verify safety from earthquake-induced shear force appropriately, it is necessary to calculate the working shear force by using design values for the materials used based on the actual strength of the materials and evaluate shear capacity reduction due to reversal cyclic loading.

The principle is to conduct more than one dynamic response analysis using different material characteristic values, materials factors and member factors for the case where response displacement is to be verified and the case where it is to be verified that shear failure does not occur. In this Specification, safety factors are determined as shown in Table C11.4.1.

(i) A material factor of 1.0 is used to calculate all response values for structures.

(ii) In the verification related to the displacement and deformation of a structure, characteristic values assuming lower limits are used as the design strength of the materials used, and design values of the tensile yield strength of steel are calculated by using lower limit values of

JIS-specified yield strength as characteristic values.

Among the member factors used for limit value calculation, the member factor for the limit value of deformation is 1.0. In the case of a member whose behavior is significantly affected by bending, because a clear decrease in load-carrying capacity does not occur even if the ultimate displacement as defined in this Specification is slightly exceeded, a member factor for displacement of 1.0 may be used. For members whose ultimate state is caused by shear failure after flexural yielding, a member factor for displacement of 1.0 may be used because increasing the member factor for shear capacity as described in Item (iv) below has the effect of increasing the amount of transverse reinforcement and preventing a clear decrease in load-carrying capacity.

(iii) If it is to be verified that shear failure of a member does not occur, the actual strength of steel is used as design strength. If JIS-conforming steel is used, the value obtained by multiplying the lower limit value of JIS-specified yield strength used as a characteristic value by the material modification factor ρ_m may be used as the design value of tensile yield strength. In general, the material modification factor ρ_m of 1.2 may be used. The characteristic strength value and the material factor for concrete may be similar to the values used for the verification of the displacement and deformation of a structure. This is based on the understanding that characteristics of concrete such as strength do not affect the response values of the structure as much as those of steel.

The member factor used for shear capacity calculation is to be in accordance with Section 9.2.2.2.

(iv) In view of safety from shear capacity reduction due to reversed cyclic loading and from excessively large earthquake, when calculating shear capacity with large deformation cycles exceeding the maximum strength point, it is necessary to use member factor values greater than the member factors indicated in Section 9.2.2.2. In general, the member factor should be increased by a factor of 1.2. The effect of reversal cyclic loading on plastic hinge regions also needs to be taken into consideration. In this connection, Section 11.1 (6)(iii) should be followed. If, however, it has been verified by an appropriate method such as an experiment that a clear decrease in strength does not occur even if the ultimate displacement of a member is exceeded, the member factor does not need to be increased.

When trying to make verification through a single response analysis using a material modification factor (ρ_m) of 1.0 in order to allow for an increase in response shear force due to the difference between the characteristic value and the actual value of reinforcing steel strength, the conventional method of calculation is to make corrections as mentioned above and then evaluate the value obtained by multiplying the response shear force by 1.2 as design shear force. In this revised Specification, in principle, in order to allow for the influence of the actual strength of reinforcing steel, verification based on an analysis using a material modification factor (ρ_m) of 1.2 be conducted separately from deformation verification using ρ_m=1.0.

(v) Uncertainty associated with structural analysis includes the uncertainty of ground characteristics, ground motion, and accuracy of analysis methods themselves. When using a standard analysis method described in this Specification, usually a structural analysis factor of 1.0 may be used.

These are summarized in Table C11.4.1.

Table C11.4.1 Standard values of safety factors and modification factors

<table>
<tr><th colspan="2" rowspan="2">Safety factor / Modification factor / Seismic performance</th><th colspan="2">Material factor γ_m</th><th rowspan="2">Member factor
γ_b</th><th rowspan="2">Structural analysis factor
γ_a</th><th rowspan="2">Load factor
γ_f</th><th rowspan="2">Structure factor
γ_i</th><th rowspan="2">Material modification factor for steel strength
ρ_m</th></tr>
<tr><th>Concrete
γ_c</th><th>Steel
γ_s</th></tr>
<tr><td>Seismic Performance Grade 1</td><td>Response valued and limit value</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td></tr>
<tr><td rowspan="2">Seismic Performance Grades 2 and 3</td><td>Response value</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0-1.2</td><td>1.0-1.2</td><td rowspan="2">1.0-1.2</td><td>Displacement : 1.0
Shear force : 1.2</td></tr>
<tr><td>Limit value</td><td>1.3</td><td>1.0 or 1.05</td><td>1.0 *
1.1-1.3 **</td><td></td><td></td><td>1.0</td></tr>
</table>

* Limit value of displacement

** The shear capacity of beam and column subjected to reversed cyclic loading is calculated by multiplying by about 1.2; hence, 1.3×1.2=1.56 for the part to be carried by concrete (V_{cd}) and 1.1×1.2=1.32 for the part to be carried shear reinforcement (V_{sd}). Shear capacity, however, in the evaluation of the flexural shear capacity ratio in accordance with Section 11.1 (6) (iii) does not need to be increased.

11.5 Verification by Experimental Tests

When structural details are determined by the experimental trials, it is necessary to satisfy the better performance for the specimens than those made according to Chapter 13. The tests of performance confirmation are carried out as follows:

(1) Shape of member, the size of cross section, the size and interval of the steel, and so on shall be corresponded to the real member.

(2) In principle, reversed cyclic loading tests with displacement control system are carried out, taking into consideration loading to imitate the real member.

[Commentary] Structural details are base of the verification in design, it is important the structural details shall be satisfied.

It is natural that the technological improvement makes it to be able to verify the required performance by alternative method. It may be determined the structural detail, if it is ensured to have better performance according to Chapter 13 by experimental tests.

(1), (2) When the structural details are determined by experimental trials, it is necessary to ensure that the specimens have better performance than the standard specimen. Also, it is necessary to use the specimens, whose size of cross section is corresponding to real member, and the loading tests imitate the real stress distribution. In principle, reversed cyclic loading tests shall be carried, because the positive and negative loading subjects to the structural members during an earthquake. In addition, it is necessary to imitate the loading pattern and stress distribution caused in real structures. To determine the structural details, it is recommended in cyclic loading tests to set the upper displacement between maximum strength point and ultimate displacement, and to set the loading cycles for each displacement from 5 to 10 times.

When it is difficult to prepare the real scale specimens and to imitate the load pattern and stress distribution, it may be determined the structural details from the experimental results under management of responsible engineer.

CHAPTER 12 VERIFICATION RELATED TO INITIAL CRACKING

12.1 General

(1) It shall be verified that initial cracks in a structure do not affect the required performance of the structure.

(2) In general, verification related to settlement cracking and plastic shrinkage cracking may be omitted. For structures that are known to be free from problems from past construction experience, verification related to cracking due to heat generated by hydration and autogenous shrinkage may be omitted.

(3) If a structure is provided with crack-inducing joints for the purpose of crack control, their structure and locations shall be determined so that the functions of the structure are not impaired.

[Commentary] The influence of cracks that occur in a structure at the construction stage on various performance attributes of the structure during its design service life has not been fully explained. It goes without saying, however, that the verification of durability, safety, serviceability and earthquake resistance described in the previous chapters is based on the assumption that initial cracks that affect the required performance of the structure do not occur at the construction stage. It is true that ascertaining that initial cracks occurring at the construction stage do not affect the required performance of the structure is a good way to make us assured that the required performance can be maintained during the design service life of the structure. Cracks caused by volume changes occurring at the construction stage can be controlled variously, and it is possible to control them by using certain construction procedures or curing methods even after mix proportions and structural details have been finalized. Another characteristic of cracks occurring at the construction stage is that they, unlike cracks that occur after the structure goes into service, can be easily detected in the acceptance inspection of the structure. A method for evaluating the influence of cracks resulting from the hydration of cement on the performance of the structure is described in the Design: Standards, Part 4, of this Specification (Fig. C12.1.1).

As mentioned above, the verification of the durability, safety, serviceability and earthquake resistance of a structure is based on the assumption that initial cracks that affect the required performance of the structure do not occur. The verification related to initial cracking described in this chapter, therefore, is meant to be conducted at the design stage. There may be cases, however, where it is more rational to conduct verification related to initial cracking at the construction stage. There may also be cases where verification related to initial cracking needs to be done at both design and construction stages. This chapter has been written assuming that it can be consulted in connection with verification related to initial cracking either at the construction stage or at both design and construction stages.

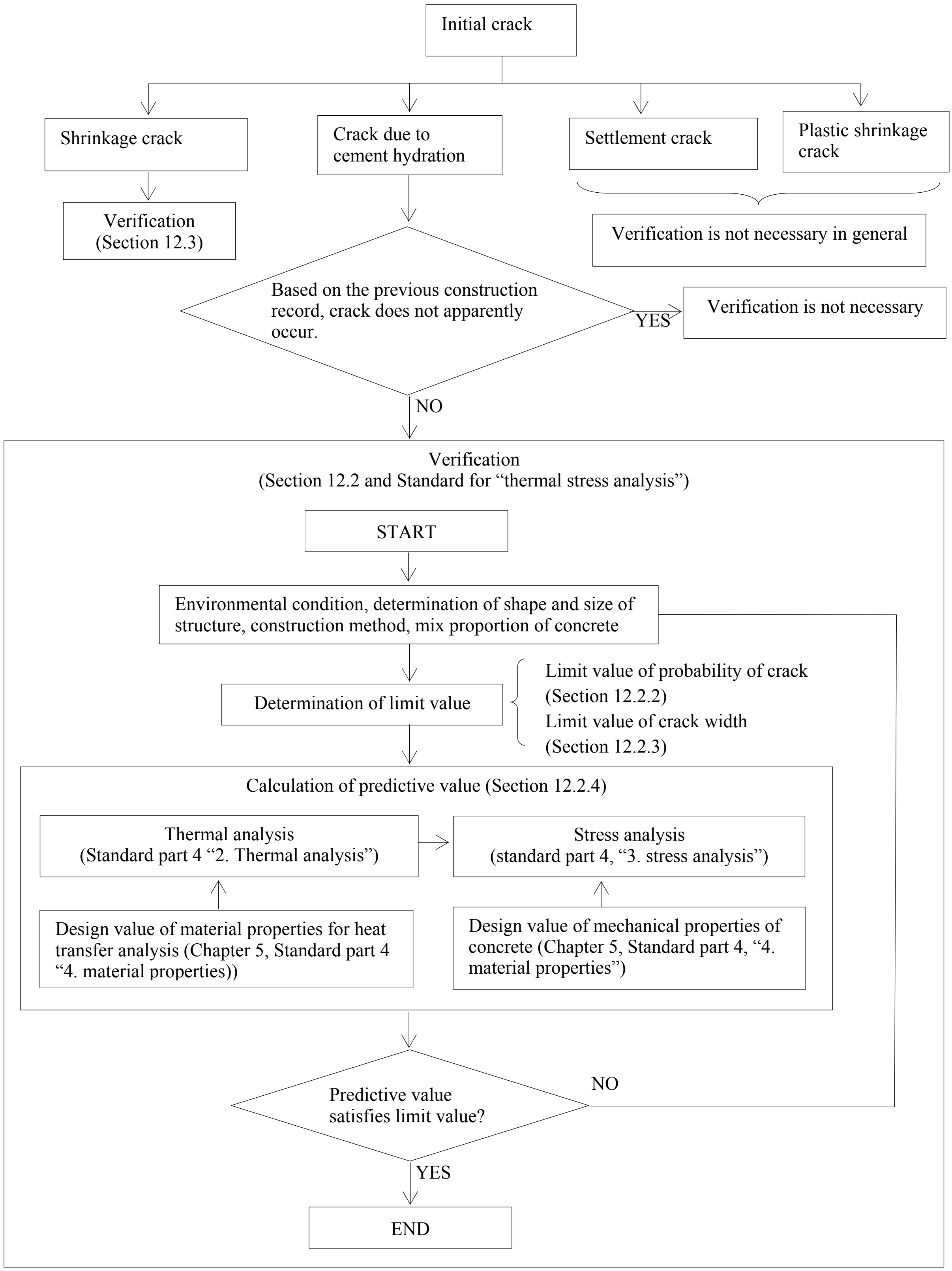


Fig. C12.1.1 Flowchart for verification related to initial cracking

This chapter deals with main types of cracks that occur at the construction stage, namely, cracks caused mainly by the segregation of ingredients before hardening or by rapid drying and cracks caused mainly by changes in volume of concrete resulting from hydration or shrinkage. Settlement cracks may occur in regions over reinforcing bars or regions with non-uniform cross sections because of the settlement of aggregates or the segregation of ingredients, but these cracks can usually be prevented by carrying out tamping at appropriate timing. Plastic shrinkage cracks may occur when the rate of evaporation of water from the surface is higher than the rate of ascent of bleeding water, but usually these cracks can also be prevented by preventing rapid drying of the surface of newly placed concrete. In short, because problematic settlement cracks and plastic shrinkage cracks can be prevented by placing concrete in accordance with the Construction section of this Specification, verification related to those types of cracks may be omitted. Even verification related to cracks caused by the hydration of cement may also be omitted if such cracks are so small that they are thought to pose no problem as a result of careful evaluation from the viewpoints of safety, serviceability, durability, appearance, etc. Construction is carried out without following the Construction section of this Specification, it is necessary to verify by an appropriate method that these types of cracks do not have adverse effects on the structure.

In cases where, as in the construction of, for example, a reinforced concrete viaduct, many structures of the same type are constructed, verification as described in this chapter may be omitted if it is known from past construction experience that initial cracks occurring at the construction stage do not affect the required performance of the structures.

The dimensions of structural members to be treated as mass concrete structures because of concern about the possibility of cracking due to the hydration of cement are difficult to determine uniformly because they vary depending on the type of structure, materials used and construction conditions. As a rule of thumb, a relatively large slab having a thickness of 80 to 100 cm or more and a wall with a restrained bottom having a thickness of 50 cm or more may be thought of as mass concrete structures. If, however, rich mix concrete is used as in the construction of prestressed concrete structure, a thinner member may need to be treated as a mass concrete structure depending on the restraint conditions. Dam concrete, however, is outside the scope of this chapter because it is dealt with in the Dam Concrete section of this Specification.

In general, it is often difficult to control thermal cracks occurring in a massive wall or wall-like structure by means of material- and mix-proportion-related control measures alone. For concrete that is required to be watertight, cracks hampers the achievement of the purpose of the structure. One method that can be used in such cases is to provide reduced-cross-section regions at certain intervals in the longitudinal direction of the structure in order to induce cracks in those regions, thereby preventing cracking in other regions and facilitating remedial measures taken in crack regions. In order to induce cracking at the intended locations, the cross-sectional reduction ratio at crack-inducing joints needs to be 30% or higher. Since the required spacing of crack-inducing joints is greatly affected by such factors as the size of the structure, the amount of reinforcing steel, placing temperature and the placing method, spacing needs to be determined in view of those factors. It is also necessary to carefully consider details such as methods for preventing steel corrosion at joints, methods for maintaining the required concrete cover and the selection of joint filler materials. By providing crack-inducing joints, cracking can be controlled with relative ease in walls and wall-like structures. Since, however, crack-inducing joints might become structural weak spots, it is necessary to determine their structure, locations, etc., appropriately by referring to relevant data such as past construction records.

12.2 Verification Related to Cracking Caused by Hydration of Cement

12.2.1 General

(1) In the verification related to cracking, it shall be judged, by verifying that cracking does not occur in a structure or crack width is not larger than the limit value, that the required performance of the structure is not lost because of cracking.

(2) If cracking is not permitted, a limit value for the probability of occurrence of cracking shall be set and verification shall be made.

(3) If cracking is permitted but restricted so that the required performance of the structure is not lost because of cracking, a limit value for crack width shall be set and verification shall be made.

(4) Verification related to cracking shall be made on the basis of concrete stress calculated through stress analysis incorporating the temperature change from the initial state calculated through thermal analysis and the volume change of concrete due to autogenous shrinkage, both of which are determined in advance.

[Commentary] (1) Cracking due to the hydration of cement is affected by a combination of various interacting factors such as the environmental conditions, the dimensions and shape of the structure, thermal and mechanical properties of materials and construction methods. It must be verified, therefore, that cracks impairing the required performance of the structure do not occur by evaluating those factors appropriately. Permitting the occurrence of cracking is not synonymous with permitting initial cracking unlimitedly. This is based on the philosophy that trying to prevent very fine cracks that are judged, as a result of careful evaluation from the viewpoints of safety, serviceability, durability, appearance, etc., to pose no problem is an unreasonably inefficient approach to the design and construction of a structure.

(2) and (3) If cracking due to the hydration of cement is permitted, verification is made by ascertaining that crack width is not greater than the limit value of crack width. If the occurrence of cracking is not permitted, it must be verified on the basis of the probability of occurrence of cracking that cracking does not occur. In this case, the upper limit value of the probability of occurrence of cracking is used as the limit value for verification. If crack width is thought to difficult to calculate, verification may be made on the basis of the probability of occurrence of cracking.

When making verification related to steel corrosion due to chlorides, it is necessary to verify in accordance with Section 8.3 that crack width is not greater than the limit value and that chloride ion concentration at the steel location is not higher than the critical concentration for steel corrosion.

(4) In the verification related to cracking due to the hydration of cement, usually a thermal analysis incorporating the influence of the environmental conditions, the dimensions and shape of the structure, thermal characteristics of the materials used, construction method details, etc., is conducted as the first step. Then, the volume change based on the temperature change from the initial state calculated through thermal analysis and the volume change due to autogenous shrinkage is determined. After that, a stress analysis modeling the restraint conditions for the placed concrete

and the mechanical properties of the materials used is conducted, and concrete stress is calculated. The strength/stress ratio (cracking index) is determined through comparison between the actual strength of concrete and the calculated stress. Whether cracking occurs or not is checked in terms of the cracking index determined from the limit value of the probability of occurrence of cracking. If crack width is to be restricted, it is verified that the crack width calculated on the basis of stress analysis is not greater than the limit value of crack width or that the calculated cracking index is not greater than the cracking index corresponding to the limit value of crack width determined on the basis of the probability of occurrence of cracking. These methods are used to ascertain that initial cracks occurring at the construction stage do not affect the required performance of the structure.

One method each of thermal analysis and stress analysis is described in Chapters 2 and 3 of the Design: Standards, Part 4, of this Specification, and standard values of thermal properties and mechanical properties of materials are indicated in Chapter 4 of the Design: Standards, Part 4, of this Specification.

12.2.2 Verification related to occurrence/nonoccurrence of cracking

(1) The occurrence or nonoccurrence of cracking shall be verified in terms of the cracking index determined from the limit value of the probability of occurrence of cracking.

(2) If measures are taken to prevent cracking, the limit value of the probability of occurrence of cracking shall be set taking into consideration the environmental conditions, the dimensions and shape of the structure, construction methods and concrete mix proportions.

[Commentary] (1) The ratio of the tensile strength (characteristic value) of concrete specimens to the principal tensile stress (calculated value) in the structure is defined as a cracking index, and the occurrence or nonoccurrence of cracking is checked by using the cracking index determined from the limit value of the probability of occurrence of cracking. If Eq. (C.12.2.1) is satisfied, structures may be deemed to have been passed:

$$I_{cr}(t) \geq \gamma_{cr} \tag{C.12.2.1}$$

where,

$I_{cr}(t)$: cracking index

$$I_{cr}(t) = f_{tk}(t)/\sigma_t(t)$$

$f_{tk}(t)$: tensile strength of concrete at age of t days

$\sigma t(t)$: maximum principal tensile stress in concrete at age of t days

γ_{cr}: safety factor for probability of occurrence of cracks. The factor maybe 1.0 to 1.8

The thermal cracking index has been conventionally used in the verification of cracking due to volume changes caused by heat generated by the hydration of cement for mass concrete. The thermal cracking index is defined as the ratio of the tensile strength of the concrete to the tensile stress in the concrete generated by heat generated by the hydration of cement. The larger the thermal cracking index, the lesser the possibility of crack occurrence and vice versa. Generally, as the index becomes smaller, the number of cracks increases and the crack width tends to grow larger. In cases where the distribution of generated stresses is more or less uniform, penetrating cracks are often formed. In cases where stresses due to volume changes have a considerable gradient in the member cross section, surface cracks do not develop into penetrating cracks immediately and, if the concrete has high fracture toughness, performance can be retained. Crack patterns in this case vary with the dimensions of the structure. Neither the fracture toughness of the materials nor stability of the cracking process is considered in the thermal cracking index. Since, however, a relatively strong correlation has been found between the thermal cracking index and the probability of occurrence of cracking, the index may be used for verification associated with the occurrence of cracking.

In this specification, the cracking index is defined as the ratio between the tensile strength (characteristic value) of concrete specimens and the principal tensile stress (calculated value) in the structure. In this case, it should be noted that the probability of occurrence of cracks corresponding to the cracking index of 1.0 may not be always 50% because the probability of occurrence of cracks is influenced by the accuracy of estimation methods and the difference between the tensile strength of test specimens and that of the structure.

(2) For use when the calculation methods described in this chapter and Chapter 3 of the Design: Standards, Part 4, of this Specification are used, the relationship between the factor of safety γ_{cr} and the probability of occurrence of cracking shown in Fig. C12.2.1 is available. For reference, typical values of γcr and the probability of occurrence of cracking are shown in Table C12.2.1.

Table C12.2.1 Typical values (for reference) of the probability of occurrence of cracking and the factor of safety γ_{cr} for ordinarily reinforced concrete structures

Table C12.2.1 Probability of occurrence of cracking and factor of safety

	probability of occurrence of cracking	factor of safety γ_{cr}
When cracking is to be prevented	5%	1.75 or greater
When cracking is to be restricted as much as possible	25%	1.45 or greater
When cracking is permitted but excessively large crack width is to be prevented	85%	1.0 or greater

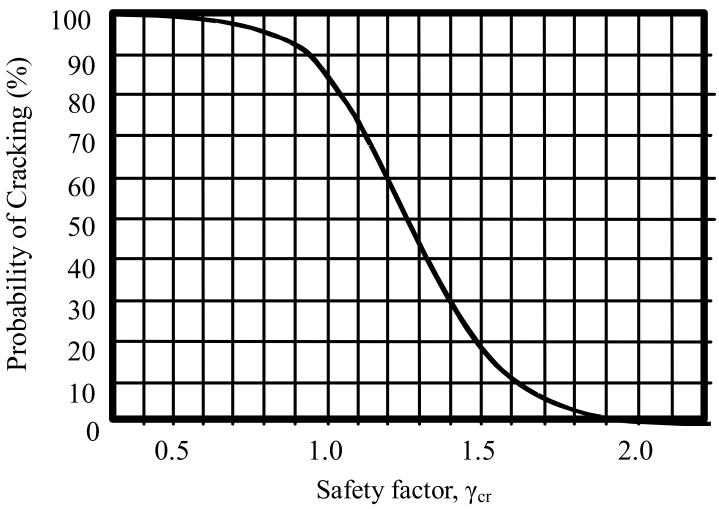


Fig. C4.2.1 Safety factor(γ_{cr}) vs. probability of cracking

What should be kept in mind here is that the probability of occurrence of cracking shown in Fig. C12.2.1 is the result obtained by conducting thermal analysis by the two-dimensional finite element method and using the CP (compact procedure) method of stress analysis. Under different conditions for stress analysis, somewhat different cracking probability curves would be obtained. The cracking probability curve shown above should be applied to a cross section in which stress change occurs gently as in a wall or wall-like structure or a slab or slab-shaped structure. It should be kept in mind that the curve should not be applied to corner regions, etc.

If the influence of autogenous shrinkage does not need to be taken into account, a simple, the cracking index $I_{cr}(t)$ may be conservatively calculated based solely on thermal analysis results using Eq. (C12.2.2) and Eq. (C12.2.3).

When stresses due to the internal restraint are predominant:

$$\text{Thermal cracking index } I_{cr}(t) = 15/\Delta T_i \qquad \text{(C12.2.2)}$$

When stresses due to the external restraint are predominant:

$$\text{Thermal cracking index } I_{cr}(t) = 10/(R \cdot \Delta T_o) \qquad \text{(C12.2.3)}$$

where,

ΔT_i: temperature difference between the inside and outside of the member at the peak temperature (^{o}C),

ΔT_o: difference between the maximum average temperature of the member and its equilibrium temperature with the ambient air temperature (^{o}C),

R: factor of the external restraint, which may be 0.5 when placed on relatively soft rock, 0.65 when placed on medium hard rock, 0.8 when placed on hard rock and 0.6 when placed on the previous concrete.

An example of a concrete member in which stresses due to internal restraint are predominant is a slab cast on soft ground, and an example of a concrete member in which stresses due to external restraint are predominant is a slab cast on a rock or massive concrete base. In deriving the indicated equations, the critical tensile strain in the concrete of 100×10^{-6} was used, and autogenous shrinkage was ignored.

12.2.3 Verification of crack width

(1) Crack width shall be verified by an appropriate method.

(2) If crack width is to be restricted, the limit value of crack width shall be determined taking into consideration the environmental conditions, the dimensions and shape of the structure, construction methods and concrete mix proportions. In general, the limit values of crack width associated with steel corrosion indicated in Section 8.3.2 may be used.

(3) If it is difficult to calculate crack width, verification may be made in terms of the probability of occurrence of cracking.

[Commentary] (1) Methods of crack width calculation include statistical methods, the CP Crack Width Method proposed by the committee on temperature and stress in mass concrete of the Japan Concrete Institute, and FEM in which the bond characteristics of reinforcing steel are taken into consideration. The method in which the post-cracking behavior of concrete is modeled is an attempt at faithful reproduction.

(2) Even if the occurrence of cracking is permitted, cracks must not be allowed to affect the required performance of the structure. It is therefore necessary to keep crack width within an acceptable range so that the performance of the structure is not impaired. The most important thing to be careful about when considering the influence of cracks on the structure is the corrosion of internal reinforcing steel. With respect to initial cracking, the limit values of crack width associated with steel corrosion indicated in Section 8.3.2 may be used as in the verification related to flexural cracking. This does not apply, however, if the limit value of crack width is determined by the appearance of the structure.

If the occurrence of cracking is permitted, attention needs to be given to cracking patterns. For example, in a wall or wall-like structure or a box culvert, longitudinal cracks directly over the main reinforcement might cause the adhesion with the reinforcing bars to decrease so that the strength of the structure decreases. The occurrence of such cracking, therefore, should not be permitted.

(3) Fig. C12.2.2 shows the relationships among crack width, the thermal cracking index and the reinforcement ratio based on experiment results. The figure shows the relationship between the maximum crack width and thermal cracking index for actual wall-type structures and specimens for different reinforcement ratios in the full cross section of the retrained body. L/H ranges from 10.0 to 15.0 (L and H: length and height of the restrained body). The longitudinal factor of the external restraint at the mid-height of the wall is 0.5 to 0.7. The maximum reduction in mean temperature in the cross section is in the range between 20°C and 54°C.

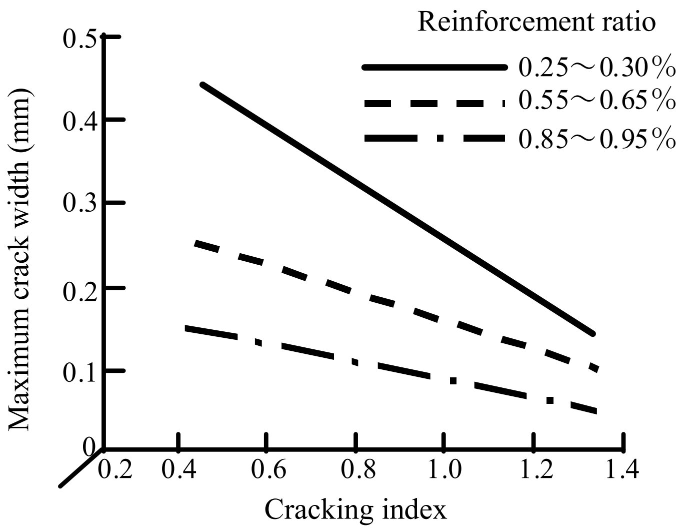


Fig. C12.2.2 Relationship between maximum crack with and cracking index

The width of the crack due to hydration of cement converges to a certain value as the tensile stress accumulated in the concrete is released due to cracking. The crack width is determined by the shrinkage of the concrete and the restraining forces of the restraining body and reinforcing steel that prevent the deformation. The crack width may become extremely large in cases where the reinforcement ratio is small. If temperature falls after the occurrence of cracks, the concrete is deformed further due to shrinkage. Then, the crack width increases unless new cracks occur. To control the crack width, therefore, the deformation of concrete due to shrinkage should be controlled. To that end, maximizing the thermal cracking index by changing the materials, mix proportion and construction method is effective. If concrete-related measures alone are not enough to control crack width, the method of controlling crack width by use of reinforcing steel may be used. Then, it is recommended to use reinforcing steel of smaller diameter as long as the ease of construction is ensured.

12.2.4 Calculation of stress and crack width

(1) The thermal analysis of concrete shall be conducted using adequate methods in accordance with factors such as the type and configuration of the structure and.

(2) Stresses due to volume changes of concrete shall, in general, be calculated, by determining volume changes of restraints such as concrete, reinforcing steel and rock, soil or foundation, so that the boundary and compatibility conditions and the equilibrium condition are satisfied.

(3) If the occurrence of cracking due to the hydration of cement is expected, crack width shall be estimated by an appropriate method.

[Commentary] (1) The methods of the thermal analysis of concrete include numerical methods such as finite element method, finite difference method, simplified numerical method such as Schmidt or Carlson method, and simplified calculation methods with the use of figures and tables, whichever can be used. Thermal analyses of concrete can also be classified into linear analyses, which assume that heat generation characteristics and thermal properties are not dependent on temperature, and nonlinear analyses, which allow for temperature dependence. However, since each method has limitation in its application, it is necessary to select the most adequate method considering the required accuracy of the analysis and various conditions of the structures subjected to the analysis. In thermal analysis, it is necessary to determine a valid analytical model and boundary conditions in accordance with environmental and other conditions of the objective structure. Calculation of temperature changes and thermal stresses may be conducted until the temperature change becomes practically stationary.

The determination of boundary conditions is as fundamental as that of the analytical model and has a significant influence on the results of the analysis. The heat transfer boundary is a boundary where heat is transferred between the concrete and atmosphere and the fixed temperature boundary is a boundary where the temperature is kept constant. In general, radiation can be ignored in the thermal analysis of concrete.

The rate of heat generation in concrete is strongly influenced by temperature. Since temperatures in a structure are usually not uniform, different parts of the structure usually generate heat at different rates. In a thermal analysis, therefore, it is desirable that thermal properties be determined from mix parameters such as the type of cement, chemical and nonchemical admixtures, cement content and water-cement ratio, and calculated rates of heat generation reflecting temperature dependence be used.

(2) Since mass concrete structures are usually more rigid than ordinary concrete structures, simplification of the compatibility conditions including the boundary conditions may result in lower accuracy of analysis. It is desirable, therefore, to use an analysis method in which both the compatibility and equilibrium conditions are satisfied and volume changes and unsteady mechanical properties of concrete can be considered (e.g., finite element method). Results of finite element analysis may vary depending on element resolution, analytical domain and boundary conditions, and the properties of the restraining body and the restrained body. If the finite element method is used, therefore, it is important to determine these parameters carefully, paying attention to past analysis data, so that the required level of accuracy is achieved.

There are various approximation methods that can be used as a means of calculating stresses in mass concrete structures in which volume changes due to temperature changes are predominant over those due to autogenous shrinkage. In general, volume change at the construction stage is most greatly affected by temperature change. Depending on the type of cement or admixture or mix proportions, however, autogenous shrinkage due to hydration may become too large to ignore. In such cases, it may be possible to enhance the accuracy of stress analysis by taking the effects of those factors into consideration.

When the minimum member size is smaller than about 400 mm, temperature changes, autogenous shrinkage and drying shrinkage, which occurs later than the first two, may become too great to ignore. In such cases, it is desirable to consider the effect of drying shrinkage as well as autogenous shrinkage.

(3) Methods for calculating crack width include statistical methods, the CP Crack Width Method proposed by the Committee on Thermal Stress in Mass Concrete of the Japan Concrete

Institute, and an FEM-based method that takes into consideration the bond characteristics of reinforcing steel.

12.3 Verification Related to Cracking due to Shrinkage

It shall be verified that cracks caused by the shrinkage of concrete such as drying shrinkage do not affect the required performance of the structure.

[Commentary] (1) Cracks caused by the shrinkage of concrete such as drying shrinkage mar the appearance of concrete structures and cause the airtightness of concrete to decrease. Shrinkage of concrete includes not only drying shrinkage resulting from the dissipation of the moisture in the concrete but also autogenous shrinkage resulting from the hydration of the cement in concrete. Usually, the two types of shrinkage occur in combination. These types of shrinkage cracks may occur as shallow cracks found at the surfaces of a structure or deep cracks that reach reinforcing bars or even cracks penetrating the full depth of members depending on such factors as the concrete materials used, mix proportion, the shape and dimensions of the structure, restraint conditions, and environmental conditions such as temperature and humidity. The influence of shrinkage cracks in a structure, therefore, on the performance of the structure varies widely. Conventionally, it has been believed that the influence of cracks caused by drying shrinkage on the performance of a structure is minor mainly because cracks of this type often occur in members with relatively low degrees of importance in a structure, because they tend to close when wetted, and because water is not supplied to the reinforcing bars inside when cracks are open because of dryness. Since, however, excessively large shrinkage cracks may affect the stiffness or deflection of structural members, it is desirable that it be verified at the design stage that the required performance of the structure is not affected. Autogenous shrinkage and drying shrinkage of concrete are dealt with in this chapter because most of these types of cracks are thought to occur before the structure goes into service.

Depending on the dimensions of members, mix design conditions, environmental conditions, etc., cracks may occur because the concrete in a structure is exposed to a dry environment while stress caused by volume changes due to temperature changes and by autogenous shrinkage is accumulated in the concrete. It is desirable, therefore, that stress introduced into the concrete in the structure be evaluated in view of volume changes of concrete due to temperature changes, autogenous shrinkage and drying shrinkage, and the occurrence of cracking be predicted accordingly.

CHAPTER 13 STRUCTURAL DETAILS OF REINFORCEMENT

11.0 Notation

γ_i : **structure factor**
γ_b : **member factor**
N'_d : **design compressive strength of concrete**
f'_{yd} : **design compressive yield strength of reinforcement**
f'_{cd} : **design compressive strength of concrete**
p_b : **balanced reinforcement ratio**
ε'_{cu} : **ultimate strain of concrete**
f_{yd} : **design tensile yield strength of reinforcement**
E_s : **modulus of elasticity of reinforcement**
$A_{w\,\min}$: **minimum amount of vertical stirrups**
b_w : **web width of concrete section**
s : **stirrup spacing**
M_{twd} : **design torsional capacity given in Section 9.2.3.2.**
A_m : **effective torsional area**
f_{wd} : **design yield stresses of transverse reinforcement**
u : **length of the centerline of transverse reinforcement**
ϕ : **diameter of reinforcing bar**
f_{bod} : **design bond strength of concrete**
c : **concrete cover**
A_t : **area of transverse reinforcement**

13.1 General

(1) General structural details described in this chapter shall be observed in the design of reinforced concrete structures.

(2) If the structural details determined for each structural types in other specifications, it shall be also observed in the design.

[Commentary] (1) This chapter is described about the structural details of reinforcing bars in reinforced concrete structures. The structural details are based on the design method described in Chapters 7, 8, 9, 10, 11 and 12. Standard construction performance laid down in the Standard Specifications for Concrete Structures "Materials and Construction" are the prerequisite for the structural details in this chapter.

(2) Structural details in other chapters are only applied to members or structures that are described in those chapters. In cases when there is a conflict in the provisions in other chapters and those given in this chapter, the former provisions shall be followed. However, the common structural details in this chapter, except the special provisions for each structural types (i.e. prestressed concrete, steel-concrete composite structures), shall be followed.

13.2 Concrete Cover

Concrete cover shall be determined to ensure the bond strength that is the prerequisite for structural performance of a reinforced concrete structure, and the requirements for fire resistance, durability, social importance of the structure, and errors in construction. Minimum concrete cover shall be larger than the summation of diameter of reinforcing bar and construction error margin.

[Commentary] The bond strength, ensured by structural detail of concrete cover described in this section, is the prerequisite for verification of usability and safety of a structure. Minimum concrete cover shall be larger than the diameter of reinforcing bar. It is necessary to ensure the sufficient concrete cover for preventing steel corrosion and enhancing fire resistance, and so on. Concrete cover shall be determined taking into consideration material quality of concrete, diameter of reinforcing bars, surrounding environment of a structure, something of harmful on the concrete surface, dimension of member, construction error, social importance of a structure, and so on. As far as the protections against fire and corrosion of reinforcement are concerned, the concrete cover shall be determined in a manner given in Section 10.7 and 8.3, respectively. Concrete cover should be more than the maximal values of diameter of bars, concrete cover from verification of durability and fire resistance, considering construction error margin.

Provisions of this section shall be applicable to cases, which use steel reinforcing bars. Concrete cover for other materials than steel may be based upon different requirements. As the results, designed concrete cover is larger than the either diameter of reinforcing bars and one that satisfies durability and fire resistance, considering the construction error. The concrete cover is larger than diameter of bars, and it can satisfy the durability and fire resistance performance.

In this chapter, the provisions are applied to steel reinforcing bars. It is necessary to consider another manner for other materials.

13.3 Clear Distance

In order to ensure that concrete surrounds the reinforcement appropriately, clear distance shall be determined taking into consideration type of member, maximum size of aggregate, diameter of reinforcing bars, construction error, and so on.

[Commentary] Clear distance shall be determined to satisfy enough workability. Workability of concrete construction is evaluated by relationship between concrete slump and clear distance of reinforcing bars.

Therefore, the construction workability shall be verified by slump assumed from concrete mixing. The relationship between concrete slump and clear distance of reinforcing bars are given in Chapter 4 in the Standard Specifications for Concrete Structures "Structural Performance Verification".

13.4 Arrangement of Reinforcement

13.4.1 Arrangement of longitudinal reinforcement

(1) Minimum reinforcement

(i) For a reinforced concrete member under the action of a predominantly axial force, longitudinal reinforcement of not less than 0.8% of concrete area shall be provided.

The concrete area mentioned here is the minimum concrete area required to resist only the applied axial force.

In case the provided concrete area is greater than the minimum defined above, it is desirable that the longitudinal reinforcement is at least 0.15% of the actual concrete area.

(ii) For a linear member under the action of predominantly flexural moments, at least 0.2% of concrete area shall be provided as longitudinal tensile reinforcement in principle. For a T-shaped cross-section, it is necessary to use the effective width of concrete compressive area in the verification. For a T-shaped cross-section at least 0.3% of effective concrete area, which is defined as the product of the effective depth d of the cross-section and the web width b_w, shall be provided as longitudinal tensile reinforcement.

(2) Maximum reinforcement

(i) For a reinforced concrete member under the action of a predominantly axial force, the provided longitudinal reinforcement shall not exceed 6% of concrete area in principle.

(ii) For a linear member under the action of predominantly flexural moments, the provided longitudinal tensile reinforcement shall not exceed 75% of the amount required for balanced failure in principle.

(3) Arrangement

(i) Additional reinforcement shall be provided in concrete where cracks due to temperature gradient, shrinkage, and so on are anticipated.

(ii) Additional reinforcement to prevent concrete cracking shall be distributed around the cross-section. The diameter and spacing of reinforcement shall be as small as possible.

(iii) Maximum spacing of longitudinal and transverse reinforcement shall not exceed 300mm in principle.

[Commentary] (1)(i) For a reinforced concrete member under the action of a predominantly axial force, the provision of minimum longitudinal reinforcement is the same as that has been conventionally used.

There is a probability of cracking in a concrete member due to shrinkage, temperature gradient, etc. To keep width of such cracks within harmless limits, the present specification stipulates the use of at least 0.15% of the concrete area as longitudinal reinforcement even in cases where the provided concrete area is greater than that required from calculations.

In arranging reinforcement according to this provision, it is preferred to use reinforcements that have smaller diameter compared with dimension of members and can bond with surrounding concrete satisfactorily. As far as the layout of the reinforcement is concerned, it is desirable that it should be distributed around the cross-section of the member. The minimum concrete area (A_c) required to resist the applied axial force may be obtained using Eq. (C13.4.1).

$$A_c = \gamma_i \gamma_b N'_d \Big/ \left(0.008 f'_{yd} + 0.85 f'_{cd}\right) \qquad \text{(C13.4.1)}$$

where, γ_i : structure factor

γ_b : member factor

N'_d : design compressive strength of concrete (N/mm^2)

f'_{yd} : design yield strength of longitudinal reinforcement in compression (N/mm^2)

f'_{cd} : design compressive strength of concrete (N/mm^2)

(ii) The intent of this provision is to ensure that designed linear reinforced concrete members have the desired flexural characteristics. When the ratio of tensile reinforcement becomes extremely small, the load to cause yielding of the provided reinforcement may become smaller than that to cause onset of flexural cracking in concrete. In such a case, the member failure may be brittle and the reinforcing bars yield or fracture immediately after initiation of cracking. In some cases, members may fail with only a single localized crack much like a plain concrete member without reinforcement. Provision of nominal tensile reinforcement, not less than 0.2%, prevents this type of failure when normal concrete having a characteristic strength of about 30 N/mm^2 or less and reinforcement with a design yield stress of about 350 N/mm^2 are used.

When high strength concrete is used, the minimum reinforcement to prevent brittle failure in flexural members should be determined in accordance with Eq. (C13.4.2).

When high strength reinforcement is used, a lower reinforcement ratio may be used.

This rule may be relaxed when extremely excessive reinforcement is used than that is derived by computation. However, in the case when tensile reinforcement ratio is extremely low, desirable structural characteristics already are not shown. In this case, reinforcement ratio is not to be less than 0.15%.

In order to prevent brittle failure of a linear reinforced concrete flexural member, the amount of reinforcement must be large enough to prevent simultaneous occurrence of onset of flexural cracking and failure of member. The minimum reinforcement ratio at which cracking moment does not exceed the flexural moment at yield of main reinforcement may be calculated using Eq. (C13.4.2).

$$p_{\min} = 0.058 \left(\frac{h}{d}\right)^2 \frac{f'^{2/3}_c}{f_{sy}} \qquad \text{(C13.4.2)}$$

(2)(ii) If the section is excessively reinforced, not only the actual placing of the reinforcement is difficult, but also the failure of the member is governed by compressive failure of the concrete, and is brittle. Hence, an upper limit on the reinforcement ratio is stipulated in relation to the balanced reinforcement ratio, which may be obtained using Eq. (C13.4.3) for ultimate state. The coefficient α in the equation is an approximate value in the case when the stress-strain relationship shown in Fig. 5.2.1 is used.

$$p_b = \alpha \frac{\varepsilon'_{cu}}{\varepsilon'_{cu} + f_{yd}/E_s} \cdot \frac{f'_{cd}}{f_{yd}} \qquad \text{(C13.4.3)}$$

where, p_b : balanced reinforcement ratio

$\alpha = 0.88 - 0.004 f'_{ck}$, where $\alpha \leq 0.68$

ε'_{cu} : ultimate strain of concrete, for which value shown in Fig. 5.2.1 may be used

f_{yd} : design tensile yield strength of reinforcement (N/mm^2)

E_s : modulus of elasticity of reinforcement, usually 200 kN/mm^2

(3)(i) The harmful cracking for durability may be occurred in concrete member due to temperature gradient, shrinkage, and so on. Adaptive reinforcement shall be arranged at the surface of member, concrete joint, and so on to prevent harmful cracking.

(ii) With the same amount of reinforcement, crack width is decreased by using small diameter and small spacing of reinforcement. To minimize cracking, it is prefer to use the reinforcement with small diameter and to arrange it in small spacing.

(iii) The spacing of reinforcement prefers to be less than 300mm from view of the crack distribution characteristics.

13.4.2 Arrangement of transverse reinforcement

(1) Arrangement of stirrup

(i) In the case of a linear member, stirrups not less than 0.15% of the concrete area shall be provided throughout the length of the member. The spacing shall exceed neither 3/4 the effective depth of the member, nor 400mm in principle. This provision may not be applied to planar members.

(ii) In the case of a linear member, if calculations indicate the need for shear reinforcement, the spacing of stirrups shall exceed neither 1/2 the effective member depth, nor 300mm. The same amount of shear reinforcement shall be provided over a distance equal to the effective member depth from the end of the zone where the need for shear reinforcement is indicated from the calculations.

(2) Arrangement of ties

(i) The spacing of ties shall exceed neither 12 times diameter of longitudinal bar, nor the minimum dimension of the section in principle. In the plastic hinge region, the spacing of ties shall exceed neither 12 times diameter of longitudinal bar, nor 1/2 times the minimum dimension of the section. The ties shall be enclosed longitudinal bars.

(ii) Using the ties in the rectangle section, the interval distance of ties in the section shall exceed neither 48 times diameter of ties, nor 1m. The intermediate ties shall be arranged, for that interval distance between ties and intermediate ties in the section does not exceed these limit values.

[Commentary] (1)(i) Diagonal cracking in members without shear reinforcement is often accompanied by a sudden reduction in member strength. When the behaviors of the whole structure are affected by the sudden failure of members, it is necessary to consider the improvement of member ductility and the re-establishment of load carrying mechanism. Therefore, even when shear reinforcement for a linear member is not necessary according to computation, the minimum amount of vertical stirrups by Eq. (C13.4.4) should be arranged in order to avoid sudden failure due to diagonal cracks caused by shrinkage of concrete, difference of temperatures, etc.

$$A_{w\min}/(b_w s) = 0.0015 \qquad \text{(C13.4.4)}$$

where, $A_{w\,\min}$: minimum amount of vertical stirrups

b_w : web width

s : stirrup spacing

With the arrangement of reinforcement specified by Eq. (C13.4.4), the increase in the shear capacity by the shear reinforcement due to Eq. (9.2.6) becomes approximately equal to V_{cd}, in general.

This minimum value required is based on deformed bars. When plain bars with small yield and bond strengths are used, it is desirable to arrange them approximately 1.5 times the volume by Eq. (C13.4.4).

(ii) In order to ensure the effective performance of stirrups as shear reinforcement, the spacing of stirrups need to be determined so that the stirrups must intersect with the diagonal cracks in web concrete. The spacing shall not exceed one half the effective depth of member cross sections nor 300mm in order to ensure that stirrups function as additional reinforcement to avoid cracking due to shrinkage and others.

Considering the shear cracking diagonal to the member axis, the range where stirrups and tie reinforcement shall be arranged in the range as a calculated necessary length plus outer sub-ranges with the same distance as the effective depth of member cross section. In this case the required amount of shear reinforcement in the sub-ranges may be taken as that at the edge of the range considered necessary by computation.

(2)(i) Transverse reinforcement, ties and spiral hoops, is effective to prevent shear cracking and buckling of longitudinal bars. Also, transverse reinforcement enhances the shear strength and confining the core concrete. The amount of reinforcement shall be verified in Chapter 11 and the spacing of ties shall not exceed the limit values described in this section.

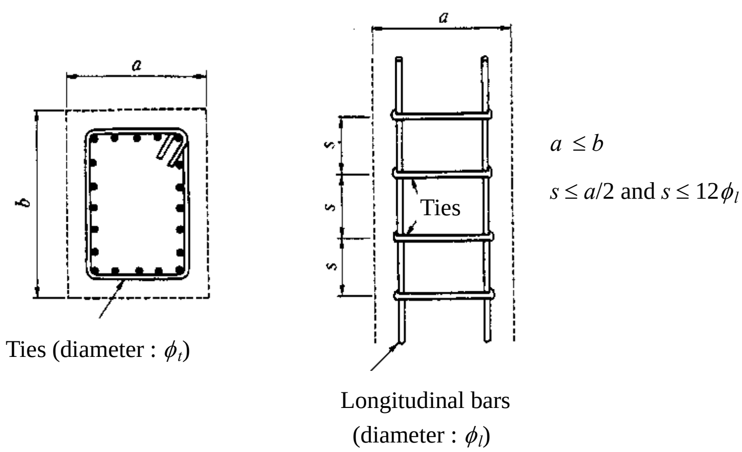


Fig. C13.4.1 Arrangement of ties enclosing longitudinal bars

(ii) When concrete members have large cross-sections, concrete may not be sufficiently confined by ties at the distance from the corner. The intermediate ties shall be arranged.

13.4.3 Arrangement of torsion reinforcement

(1) The minimum torsion reinforcement for a linear member shall be calculated using Eq. (13.4.1).

$$\Sigma A_{tl} = M_{tud}\, u/(3A_m f_{ld}) \qquad \textbf{(longitudinal reinforcement)}$$

$$\Sigma A_{tw} = M_{tud}\, s/(3A_m f_{wd}) \qquad \textbf{(transverse reinforcement)} \qquad \textbf{(13.4.1)}$$

where, M_{twd} **: design torsional capacity given in Section 9.2.3.2.**

A_m **: effective torsional area (rectangular section:** b_0d_0**, circular or cylinder section:** $\pi d_o^2/4$**)**

f_{ld}, f_{wd} **: design yield stresses of longitudinal and transverse reinforcement, respectively**

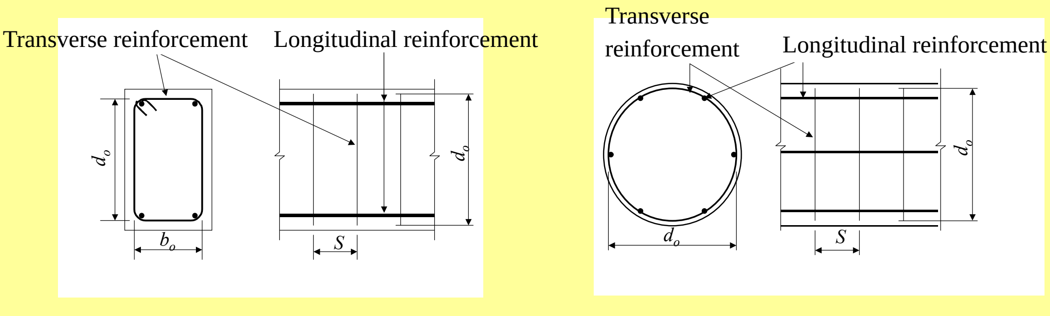
s **: longitudinal spacing of transverse reinforcement that works effectively as torsion reinforcement**

u **: length of the centerline of transverse reinforcement (rectangular section:** $2(b_0+d_0)$**, circular or cylinder section:** πd_0**)**

(2) The torsion reinforcement comprises closed transverse reinforcement and longitudinal reinforcement perpendicular to the transverse reinforcement, as shown in Fig. 13.4.1.

(3) The longitudinal reinforcement which works effectively as torsion reinforcement shall be arranged symmetrically with respect to the vertical and horizontal axes of the cross section of a member.

(4) The transverse reinforcement which works effectively as torsion reinforcement shall have acute or semicircular hooks at the ends, enclose the longitudinal reinforcement, and be anchored to the inner concrete. In cases when the distance between the position of the transverse reinforcement and the outer perimeter of the cross section exceeds 0.2 times the member width, the transverse reinforcement should not be considered as effective torsion reinforcement in principle.

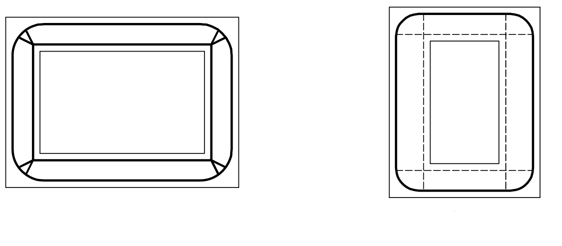


(a) Rectangular cross section **(b) Circular cross section**

Fig. 13.4.1 Arrangement of torsion reinforcement

(5) For a rectangular cross section, at least one longitudinal reinforcing bar shall be provided at each corner of the cross section, as shown in Fig. 13.4.1(a). In the case of a circular cross section, at least six longitudinal reinforcing bars shall be arranged at equal intervals, as shown in Fig. 13.4.1(b).

(6) The transverse reinforcement for a box section shall be provided as shown in Fig. 13.4.2.



(a) Concrete flange and web are thin. (b) Concrete flange and web are thick.

Fig. 13.4.2 Arrangement of transverse reinforcement for a box section

(7) Appropriate amount of torsion reinforcement shall be provided in portions where required. The same reinforcement shall also be extended into adjacent segments up to a length equal to the overall height or the diameter of the cross section of the member. In the remaining portions of the member, the minimum amount of reinforcement shall be provided.

[Commentary] (1) According to the existing experimental data, it has been confirmed that the torsional failure characteristics are greatly improved when torsion reinforcement with the torsional capacity corresponding to that of concrete is provided. However, if the amount of reinforcement corresponding to it is specified as the minimum reinforcement, excessive amount of reinforcement may be given when the design torsional moment slightly exceeds the negligible torsion. Therefore, the torsion reinforcement corresponding to one-half of the torsional capacity provided by the resistance of concrete is specified as the minimum amount. Also, for the torsional moment exceeding this value, the reinforcement corresponding to the design torsional moment may be provided.

(2) The most effective arrangement of torsion reinforcement is such that torsion reinforcement crosses the member axis at an angle of 45 degrees. However, if torsional moment subject in cyclic pattern, the reinforcement arrangement becomes complex and also bending of reinforcement becomes difficult. Meanwhile, the combination of longitudinal reinforcement and closed transverse reinforcement perpendicular to the longitudinal one, as shown in Fig. 13.4.1, has a shape similar to the vertical stirrup against shear forces, and the fabrication and the bending of reinforcement is simple, although the effectiveness for torsion reinforcement is reduced to some extent. Therefore, it may be considered practical and is specified as the basic arrangement configuration of torsion reinforcement. Also, this is the arrangement configuration which is used to derive the equation for the torsional capacity.

(3) When the reinforcement for flexure satisfies the requirements for the arrangement configuration of torsion reinforcement, it may be considered as torsion reinforcement.

(4) If the ends of transverse reinforcement are not sufficiently anchored to the inner concrete, they are not adequate as torsion reinforcement. Hence this requirement is provided. If the reinforcement for shear forces satisfies the codes for the arrangement of torsion reinforcement, it

may be considered as torsion reinforcement. If more than one set of transverse reinforcing bars for torsion reinforcement are used, the side length of the transverse reinforcement may be the centerline of the transverse reinforcement group. When the distance between the transverse reinforcement and the outer member surface is more than 0.2 times the member width, it may not be considered as transverse reinforcing bars for torsion reinforcement. This is because the transverse reinforcement arranged in the central region of the cross section does not work effectively against torsion.

(7) The region reinforced by torsion reinforcement is extended beyond the length required from calculation by the height of member for ensuring safety, because unexpected loading and growth of cracks may be caused.

Also, the longitudinal and transverse reinforcement when torsional reinforcement is required from calculation should be spaced so that the torsional cracks always cross the reinforcement and the diagonal compression of concrete should be effectively maintained.

13.5 Bend Configurations of Reinforcement

(1) When bended reinforcement is used, bend configuration and arrangement shall be determine taking into consideration the bearing stress of the concrete.

(2) Configuration of standard hooks is followed with Section 13.6.2.

[Commentary] (1) Inner radium of bend configuration of reinforcement shall be determined not to provide large bearing stress to concrete.

13.6 Development of Reinforcement

13.6.1 General

(1) In order to achieve their full strength, reinforcing bars shall be firmly anchored in concrete so that the bar ends are not pulled out of the concrete.

(2) Ends of the reinforcing bars shall be sufficiently embedded into concrete by the matter shown as below.

(i) Bars are developed through bond stress between the reinforcement and the concrete.

(ii) Bars are developed by provision of a hook.

(iii) Bars are developed by suitable mechanical device.

(3) If the method described in Item (2) (i) or (ii) is used, basically the shapes of standard hooks shall be in accordance with Section 13.6.2, and the development length shall be calculated in accordance with Section 13.6.3, and the bar ends shall be anchored taking into consideration the type of structure or member, the state of loading, arrangement of reinforcement, the state of stress at the anchoring location, etc.

(4) If a method other than those mentioned in Item (3) is used, the method shall satisfy the required anchoring performance taking into consideration the type of structure or member, the state of loading, arrangement of reinforcement, the state of stress at the

anchoring location, etc.

(5) Longitudinal reinforcement shall be anchored in view of the stress state of the reinforcing bars in the anchoring zone and the characteristics of the member.

[Commentary] (1) In reinforced concrete, reinforcing bars and concrete must act together in resisting external loads. Thus, under the action of external loads, the anchorage of reinforcement in concrete is very important. Since the effects of local bond can be neglected with sufficient anchorage of reinforcement, only the anchorage of ends of the bars is described in this section.

(2) In order to allow development of the full strength of reinforcing bars, either the end of the bars should be sufficiently embedded in the concrete, or provisions for hooks, appropriately made, so that the reinforcing bars do not get simply pulled out of the concrete.

Hooks may not be provided in the anchored regions in the case of deformed bars, but sufficient reinforcement in the perpendicular direction should be provided to enable development of bond. However even when deformed bars are used, it is recommended that hooks are provided in cases such as fixed end in members, both ends of tensile reinforcement in footing, tensile reinforcement at free end of cantilever beam, to ensure that even onset of large cracks does not cause the bars to be pulled out.

(3) If the bar ends are anchored by the method described in Item (2) (i) or (ii) and no special examination is conducted, the reinforcing bars may be anchored in accordance with the Design: Standards, Part 5, of this Specification.

(4) For example, other anchoring methods are shown below.

(a) Out of specification such as, high-strength reinforcing bars, large diameter reinforcing bars, special deformed bars, etc. These are embedded in concrete with the development length.

(b) Ends of reinforcing bars with special bend configurations

(c) End of bars welded with steel plate, nut, metal, and so on. In addition, end of bars welded into steel members.

(d) The development length is shortened by improving the bond strength between reinforcing bars concrete.

If an anchoring method other than those described in Item (3) is used, the method must satisfy the required performance according to the type of structure and load characteristics. In general, performance requirements include those listed below. Anchoring performance details and verification methods shall be in accordance with the Recommendations for Design, Fabrication and Evaluation of Anchorages and Joints in Reinforcing Bars (2007).

1) Static load capacity

2) High-stress cyclic load capacity

3) High-cycle load capacity

4) Reliability attributable to construction

5) Others (performance at low temperature, etc.)

(5) The anchoring of longitudinal reinforcement varies depending on the stress state of reinforcing bars in the anchoring region and the characteristics of the member. It is therefore

necessary to determine the development length of reinforcing bars, taking into consideration the influence of these factors. If no special examination is conducted, the Design: Standards, Part 5, of this Specification should be followed.

13.6.2 Standard hooks

(1) Standard hooks are defined as semi-circle hooks, rectangular hooks, and acute-angled hooks.

(2) The configurations of standard hooks shall be as specified in Items (i) to (iii) below (see Fig. 13.6.1).

(i) Semicircle hook is defined as one having the end of the bar bent through 180 degrees and has a straight extension which measures neither less than 4 times bar diameter nor less than 60mm.

(ii) Acute-angled hook is defined as one having the end of the bar bent through 135 degrees and has a straight extension which measures neither less than 6 times bar diameter nor less than 60mm.

(iii) Rectangular hook is defined as one having the end of the bar bent through 90 degrees and has a straight extension which measures not less than 12 times bar diameter.

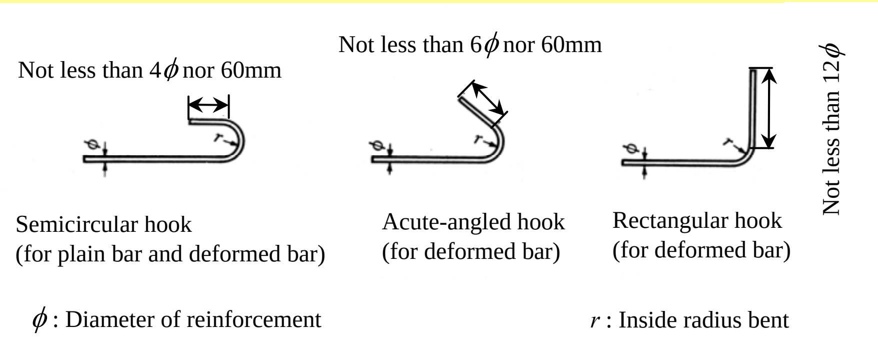


Fig. 13.6.1 Configuration of hook at free end of bar

(3) Standard hooks for the longitudinal reinforcement shall satisfy the requirements specified in Items (i) to (iii) below.

(i) When using plain bars as longitudinal reinforcement, the semicircular hook shall be used as the standard hook.

(ii) Inside radius of bend for hook shall not be less than that given in Table 13.6.1, where ϕ is diameter of the bar.

(4) Standard hooks for stirrups and hoops shall satisfy the requirements specified in Items (i) to (v) below.

(i) A standard hook shall be provided at the end of stirrups, ties and hoops.

(ii) In cases stirrups, ties and hoops are made using plain bars, semicircular hooks shall be used.

(iii) In cases stirrups are made using deformed bars, rectangular or acute-angled hooks can be used.

(iv) In case ties are made using deformed bars, semicircular or acute-angled hook shall be used.

(v) Inside radius of bend for stirrups and ties shall not be less than the values shown in Table 13.6.1. For stirrups made with bars of diameter less than 10mm, however, inside radius of bend may be taken as 1.5ϕ, where ϕ is diameter of the bar.

Table 13.6.1 Inside radius of bend for hook

Type		Inside radius of bars (r)	
		Longitudinal bars	Stirrups and ties
Plane bar	SR235	2.0ϕ	1.0ϕ
	SR295	2.5ϕ	2.0ϕ
Deformed bar	SD295A, B	2.5ϕ	2.0ϕ
	SD345	2.5ϕ	2.0ϕ
	SD390	3.0ϕ	2.5ϕ
	SD490	3.5ϕ	3.0ϕ

[Commentary] (2) The length of the straight extension of the hook has been determined to ensure that the hook is effective and to facilitate the making hook. For reinforcement bent at less than 90 degrees, provisions in Section 13.6.3 should be followed.

(3) The inside radius of bend of hooks here is slightly greater than that specified for the bending test in JIS G 3112 "Steel Bars for Concrete Reinforcement", considering the need to appropriately distribute concrete, and ensuring that the reinforcement is not damaged and that the hooks are effective. The fracture of reinforcing bars may be occurred with the small inside radius of bend. The radius of bend of hook shall not be less than the verified value described in this section.

(i) The bond strength between plain steel bars and concrete is considerably lower than that between deformed steel bars and concrete and varies widely depending on the surface properties of plain bars, the properties of the concrete around the plain bars, etc. According to pull-out test results, reinforced concrete members reinforced with hookless plain bars usually fail because of the slipping out of reinforcing bars. For these reasons, semicircular hooks must be installed at the ends of plain bars in order to achieve reliable anchoring.

(4) Ties serve to confine the core concrete and to prevent buckling of the longitudinal reinforcing bars. In order to satisfy the required performance, the ties shall be arranged at close intervals, and anchored sufficiently.

(5) Smaller inside radius of bend for stirrups and ties have been provided for facilitating the arrangement of reinforcing bars within the cross section. It is necessary to ensure that the reinforcing bars are not damaged by bending.

13.6.3 Basic development length

(1) Basic development length (l_d) shall be calculated using Eq. (13.6.1) and modified by Items (i) to (iii) below. However, modified l_d shall not be less than 20ϕ.

$$l_d = \alpha \frac{f_{yd}}{4 f_{bod}} \phi \tag{13.6.1}$$

where, ϕ : diameter of reinforcing bar

f_{yd} : design tensile yield strength of reinforcement

f_{bod} : design bond strength of concrete, which may be obtained using Eq. (5.3.2), assuming γ_c to be 1.3, and with the condition that $f_{bod} \le 3.2$ N/mm^2.

$\alpha = 1.0$ (in case of $k_c \le 1.0$)

$= 0.9$ (in case of $1.0 < k_c \le 1.5$)

$= 0.8$ (in case of $1.5 < k_c \le 2.0$)

$= 0.7$ (in case of $2.0 < k_c \le 2.5$)

$= 0.6$ (in case of $2.5 < k_c$)

where, $k_c = \dfrac{c}{\phi} + \dfrac{15 A_t}{s \phi}$

c : concrete cover of reinforcing bar or half of the clear distance between reinforcement, whichever is smaller.

A_t : area of transverse reinforcement arranged perpendicular to the assumed splitting failure surface

s : distance between the centers of transverse reinforcement

(i) Basic development length, l_d, is given by Eq. (13.6.1). Basic development length, provided with standard hooks, may be reduced by 10ϕ.

(ii) The basic development length in compression reinforcement may be taken as 0.8 of that obtained as shown in accordance with Eq. (13.6.1). It shall not be reduced in case of using standard hooks.

(iii) In cases the anchored reinforcing bar is located at a depth less than 300mm from the concrete surface, and at an angle less than 45 degrees from the horizontal, the basic development length shall be give by Item (i) or (ii).

(2) In cases when the provided area of steel (A_s) is larger than the amount required from computations (A_{sc}), the development length, l_0, may be estimated using Eq. (13.6.2).

$$l_0 \ge l_d \cdot (A_{sc} / A_s) \tag{13.6.2}$$

where, $l_0 \ge l_d/3$, $l_0 \ge 10\phi$, ϕ : diameter of reinforcement

(3) Development length in the case of reinforcement, which is bent in the anchored portion shall be taken as given below (see in Fig. 13.6.2).

(i) In cases when the inside radius of bend is not less than 10 times the diameter of the bar, the entire length of the reinforcement including the bent portion shall be considered effective.

(ii) In cases when the inside radius of bend is less than 10 times the diameter of the bar and the straight portion beyond the bend extends more than 10 times diameter of the bar, the straight portion shall be considered effective.

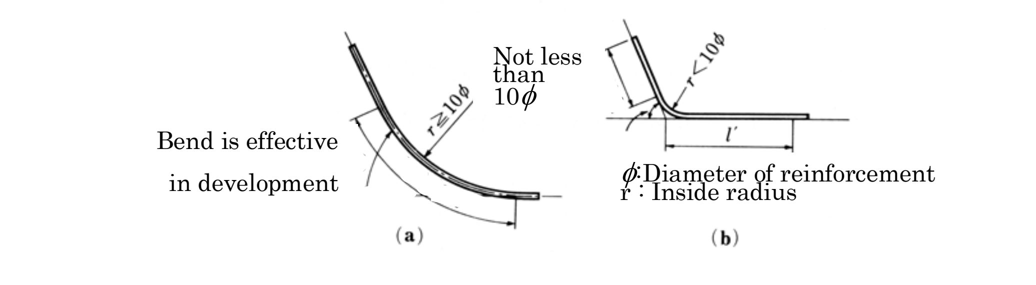


Fig. 13.6.2 Development length of reinforcement with bend

[Commentary] (1) The required development length for reinforcing bars depends on the type of the bar, strength of concrete, cover thickness, and condition of transverse reinforcement. These factors shall be appropriately considered when calculating the required basic development length.

The required development length, l_o, in cases when transverse reinforcement is provided may be given using Eq. (C13.6.1).

$$l_0 = \frac{\left(\dfrac{f_{yd}}{1.25\sqrt{f'_{cd}}} - 13.3\right)\phi}{0.318 + 0.795\left(\dfrac{c}{\phi} + \dfrac{15A_t}{s\ \phi}\right)} \tag{C13.6.1}$$

where, f_{yd} : design yield strength of reinforcement (N/mm^2)

f'_{cd} : design compressive strength of concrete (N/mm^2), which may be obtained using Eq. (C13.6.2), with γ_c of 1.3.

$$f'_{cd} = \frac{f'_{ck}}{\gamma_c} \tag{C13.6.2}$$

where, $c/\phi \leq 2.5$.

It is very difficult to take into account of the effect of all the factors such as type of the bar, strength of concrete, cover thickness, and condition of transverse reinforcement. The factor α has been introduced to take into account these effects.

The required development length for tensile reinforcement provided with standard hooks has been reduced as the length of the hook is added to the development length, and, the bearing stress of the concrete confined within the hook is also expected to transfer forces. Though, in principle, the extent of reduction in the development length of tensile reinforcement with standard hooks should vary depending upon the type of bar, strength of concrete, etc., a uniform simplified value of 10ϕ in all cases has been recommended on the basis of provisions in standards in other countries. In compression reinforcement, the development length shall not be reduced on account of presence of hooks.

(2) The development length should be calculated using the basic development length l_d determined on the basis of the size of the bars and their arrangement, and the strength of the concrete, and modified in taking into consideration the actual condition of use.

In cases that the amount of reinforcement exceeds that required by computation, the basic development length may be reduced. However a certain minimum basic development length, l_0, should be determined to ensure the safety for additional forces.

13.7 Splices in Reinforcement

(1) Appropriate splices shall be provided in the reinforcement taking into account factors such as the type and diameter of the bars, state of stress, and location of splices, and so on.

(2) To the extent possible, splices shall not be located at cross-sections of members subject to high stresses.

(3) Splices shall not be clustered at the same cross-section of a member, and the maximum number of joints permitted in the same cross section shall be one in every two bars. To ensure that splices do not cluster at the same section, in general, the standard distance between two staggered splices in the longitudinal direction shall not be less than the sum of the splice length and 25 times the bar diameter.

(4) Clear distance between two adjacent splices or between a splice and a bar shall not be less than the maximum size of coarse aggregate.

(5) When splicing is provided after distributing reinforcing bars, adequate allowance shall be made for operation of jointing machines.

(6) Concrete cover of splices shall be provided in accordance with requirements of Section 13.2.

(7) If lap jointing is used, in general, the lap length is determined by the basic development length indicated in Section 13.6.3, and joints shall be provided taking into consideration such factors as the type of structure or member, the state of loading, arrangement of reinforcement, the stress state at the joint location, etc.

(8) If joints other than lap joints are used, the joints shall satisfy joint performance requirements according to the type of structure, the state of loading, arrangement of reinforcement, the stress state at the joint location, etc.

[Commentary] (1) Strength and reliability of splices depend on such factors as kind of splices, construction methods, the type of bars, state of stress, and location of splices. Therefore, appropriate splices shall be provided so that their performance can be well exhibited in response to the above-mentioned factors.

(2) Splices in reinforcement could become weak points, and become the cause for reduction in strength of the structural members that contain splices, when the splices are provided in regions subject to high stresses. Thus, splices at cross sections subject to high tensile stresses, such as the mid-spans, are preferably avoided.

(3) In cases when splices with weak points are concentrated in a single cross-section, the structural performance of the member may be seriously decreased. In addition, certain types of splices provided in a certain area also hamper proper compaction of concrete, and therefore, the location of splices should be staggered. The recommendation for the splices to be staggered by a distance equal to the sum of the splice length plus 25 times the bar diameter, has been made considering this requirement may ensure adequate strength of the joint through bond action, even if some of the splices are poor. The adverse effects of splices on compaction of concrete may also be considered to be minor provided the provisions for staggering the splices are followed.

If splices cluster at the same cross-section in a less favorable condition, an appropriate joint strength must be determined. The performance of joints and verification methods are as described in the Japan Society of Civil Engineers' Recommendations for Design, Fabrication and Evaluation of Anchorages and Joints in Reinforcing Bars (2007).

(4) Requirements have been waived from the practical consideration that the amount of reinforcing steel increases at joints in comparison with normal cross sections and that a requirement of too much clear distance may cause difficulty in proportioning of members because of increase in spacing of reinforcement. However, in the case of splices, the maximum possible spacing has been recommended, as concrete tends to spread unevenly in the neighborhood of splices.

(5) When splices are fabricated after arranging the reinforcing bars, the required clear distances may be considerably greater than the maximum size of coarse aggregate to allow for the operation of a jointing machine. Therefore, it is important to consider the method and procedure of fabrication of splices at the design stage itself, in order to avoid problems in the construction stage.

(6) This requirement has been included from the consideration that splices using sleeves, couplers, gas pressure welding joints and others should have adequate concrete cover, at the same level as reinforcing bars.

It is noted that concrete cover to stirrups or ties at joints shall conform to provisions of Section 13.2.

(7) If lap joints are used and no special examination is conducted, joints may be provided in accordance with the Design: Standards, Part 5, of this Specification.

(8) If joints other than lap joints are used, the joints must satisfy joint performance requirements according to the type of structure, the state of loading, arrangement of reinforcement, the stress state at the joint location, etc. Commonly specified performance requirements are listed below. Details of performance requirements for joints and verification methods are referred to the Recommendations for Design, Fabrication and Evaluation of Anchorages and Joints in Reinforcing Bars (2007).

1) Static load capacity

2) High-stress cyclic load capacity

3) High-cycle load capacity

4) Reliability attributable to construction

5) Others (performance at low temperature, etc.)

13.8 Reinforcement of Members

Each member shall be provided with necessary reinforcement, taking into consideration the shape and stiffness of the member, boundary conditions, load characteristics, the state of loading, an unexpected situation such as steel corrosion, concrete cracking, initial defect, construction workability, and so on.

[Commentary] Reinforcement for concrete members includes not only reinforcement calculated through the performance verification described in this chapter but also nonstructural reinforcement and reinforcement provided according to empirical judgment.

The arrangement of reinforcement shall be determined taking into consideration the shape and stiffness of members such as beams, columns, rigid frames, arches, slabs, footings, shells and walls, the ratio to the stiffness of adjoining members, boundary conditions such as the type of joint structure, load characteristics, and the state of loading. It is also necessary to consider the construction ability as well as member characteristics and the risks of problems such as steel corrosion, unexpected cracking of concrete, unexpected initial defects, etc.

These types of reinforcement must be arranged according to the results of numerical analysis or experimental tests. The special examination should be managed by the responsible engineer.

If no special examination is conducted, arrangement of reinforcement may be provided according to the Design: Standards, Part 5, of this Specification.

CHAPTER 14　OTHER STRUCTURAL DETAILS

14.1　General

General structural details of reinforced concrete structures, prestressed concrete structures and steel-concrete composite structures shall be provided in accordance with this chapter.

[Commentary]　This chapter specifies common structural details of reinforced concrete structures, prestressed concrete structures and steel-concrete composite structures that are not directly related to verification methods, such as items specified to make up for structural weaknesses.　If structural details relevant to this chapter are specified separately in other parts of this Specification, those specifications must be followed.

14.2　Beveling

Corners of the reinforced concrete members shall be beveled.　Especially in cases when the member of structures in cold regions or structures likely to be subjected to severe weather action, beveling shall be carefully considered at the design stage, and the chamfers or fillets shall be clearly indicated in design drawings.

[Commentary]　Since corners of the reinforced concrete member tend to be damaged by freezing or impact, it is important to provide appropriate chamfers or fillets, as may be required.　Especially, in case of members in cold regions or otherwise subjected to severe weather action, larger chamfers or fillets should be provided.

14.3　Additional Reinforcement for Exposed Surfaces

Additional reinforcement shall be provided near large exposed surfaces in order to prevent occurrence of harmful cracks due to shrinkage and changes in temperature.

[Commentary]　Use of closely spaced small diameters reinforcing bars is recommended as additional reinforcement.　Such reinforcement is more effective to prevent the cracking of concrete. In the case of retaining wall, use of more than 500mm^2 (sectional area of bars) of additional horizontal reinforcement per one-meter height of wall is recommended with spacing not exceeding 300mm.　The concrete cover should be provided in accordance with the provisions of Section 13.2.

14.4　Reinforcing for Concentrated Reactions

Additional reinforcement shall be provided in portions under concentrated reaction and excessive stress concentration.

[Commentary]　Concentration of stresses occurs in structural components such as mid supports of continuous slabs or beams, connections of columns and footings, where they are subjected to concentrated reactions, or portions where the cross-sectional area changes drastically.　Since it is difficult to actually calculate stresses in such portions, appropriate additional reinforcement should be provided on the basis of test results or cracks in similar existing structures.

14.5 Reinforcing for Openings

Appropriate additional reinforcement shall be providing around all openings in structural members, such as slabs and walls, to safeguard against cracks due to stress concentration.

[Commentary] Cracks due to stress concentration tend to occur around openings in such structural members as slabs and walls. Occurrence of these cracks depends on specific local conditions, and therefore methods of reinforcement are based on available results, such as approximate calculations, experimental tests describing the actual conditions, and past cracking records. The additional reinforcement is generally provided as shown in Fig. C14.5.1. Such reinforcement should be extended to allow for appropriate development length in the bars beyond the corners.

The primary and distribution reinforcement which cannot be provided because of the openings should be arranged around the openings in a manner that requirement for the amount of reinforcement at any cross-section is satisfied.

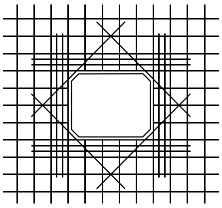


Fig. C14.5.1 Additional reinforcement around opening

14.6 Construction Joints

14.6.1 General

(1) Location and direction of construction joints shall be decided so as not to have any detrimental effects on the strength, the durability and external appearance of structures.

(2) Construction joints shall not be located on the portion subjected to large shear force, and it is recommended that direction of construction joints is crossed at right angles to the direction of compressive force in member. In the case construction joints are located on the portion subjected to large shear force, the notch shall be provided or construction joints shall be reinforced appropriately.

(3) It is not recommended to locate the construction joints for the concrete structures in coastal environments. If the construction joints are located, the durability shall be considered.

(4) Construction joints shall be located with appropriate spacing for the water-tight structures.

(5) It is desirable that construction joints are indicated in design drawings.

[Commentary] (1) Since the location and direction of construction joints affect the strength of the structure, they should be located in portions where the shear force is less and, at right angles to the direction of compressive force, according to the requirements specified herein. In the case of connections of columns and beams, considering the settlement of concrete, the construction joints may be located at the top of columns connected with beams. Since construction joints render the reinforcement susceptible to corrosion, it is advisable not to locate these joints in a region where corrosive material may penetrate through such joints.

(2) Construction joints may be weak-points for shear force. It is necessary to locate the construction joints on the position subjected to small shear force, and it is recommended that the direct of construction joints is crossed at right angles to the direction of compressive force in member to enhance the shear strength of construction joints. In the case that construction joints are located on the portion subjected to large shear force, the notch shall be provided or construction joints shall be reinforced with appropriate reinforcing bars.

(4) If the large mass concrete is constructed without joints, the structure could not have the water-tight performance by large cracking. The verification is to ensure the water-tight performance requirement.

14.6.2 Construction joints of columns or walls integrated with slabs

Construction joints of column or wall integrated with slab is located near the boundary with slab in principle. Haunches shall be constructed with slab continuously. Also, structure with coving shall be constructed with slab continuously.

[Commentary] The concrete of haunches in slab cannot shrink and sink with columns and walls, because it is supported by formwork directly. Considering the haunches behave with slab, haunches shall be constructed with slab continuously.

14.6.3 Construction joints of slab

Construction joints of slab are located near the center of span of slab or beam in principle. In the case that main beam crosses transverse beam at the center of span, construction joints of beam shall be located at distance of two times width of transverse beam. Construction joints shall be reinforced by diagonal tensile steel bars to enhance the shear strength.

[Commentary] In general, shear action is small and direction of construction joints is crossed at right angles to the direction of compressive force near the center of span. Construction joints are located near the center of span because the decreasing of strength of slab and beam are not serious. However, in the case that the main beam crosses the transverse beam at the center of span, it is appropriate that construction joints of beam shall be located from distance of two times width of transverse beam. In this case, large shear force subjects to the construction joints, and it is recommended that the construction joints shall be reinforced by diagonal tensile steel bars through 45 degree (see Fig. C14.6.1).

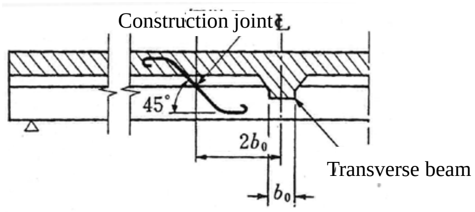


Fig. C14.6.1 Reinforcement of construction joins

14.6.4 Construction joints of arches

The direction of construction joints shall be crossed at right angles to axis of arch member in principle. In the case that direction of construction joints is crossed at parallel to axis of arch member, it is necessary to provide the construction joints taking into consideration the location, the method of reinforcement, and so on.

[Commentary] In the case that construction joints are not located in direction at the right angles to axis of arch member, shear force subject along the construction joints and it may become a structural weak point. This provision is to prevent such shear action.

When the direction of construction joints crosses at the parallel to axis of arch member, construction joints are subjected to shear force from eccentricity of live load, and sufficient consideration is needed.

14.7 Expansion Joints

(1) Location and the function of expansion joints shall be decided in a manner that provides the most effective protection against cracking, and, to make the movements due to shrinkage or expansion as free as possible. These shall be clearly indicated in design drawings.

(2) Expansion joints shall not be confined by both sides of structures.

(3) Joint sealer and water stop shall be provided in expansion joints when needed.

(4) In the case that expansion joints is to be moved in the longitudinal direction, it is recommended that the notch is provided and dowel bar is used.

[Commentary] (1) In the case of structures like retaining walls cracks may occur on account of high stresses due to restraining movements induced by changes in temperature or moisture, or high stresses due to inconstancy of the changes in a section. Therefore, the location and structure of expansion joint shall be determined as specified herein, and, shall be indicated in design drawings.

(2) Locating expansion joints, there are a case to separate both sides of structures and member completely and a case to separate only concrete and to be strung by reinforcing bar (see Fig. C14.7.1).

(3) It is recommended to use the joint sealer when sand might enter the space of expansion joints. The joint sealer and filler are made of the asphalt, the rubber, the resin, and so on. The

stretch water stop is recommended to use for the water-tight structures.　The water stop is made of the sheet copper, the stainless plate, the vinyl chloride resin, and rubber, and so on.

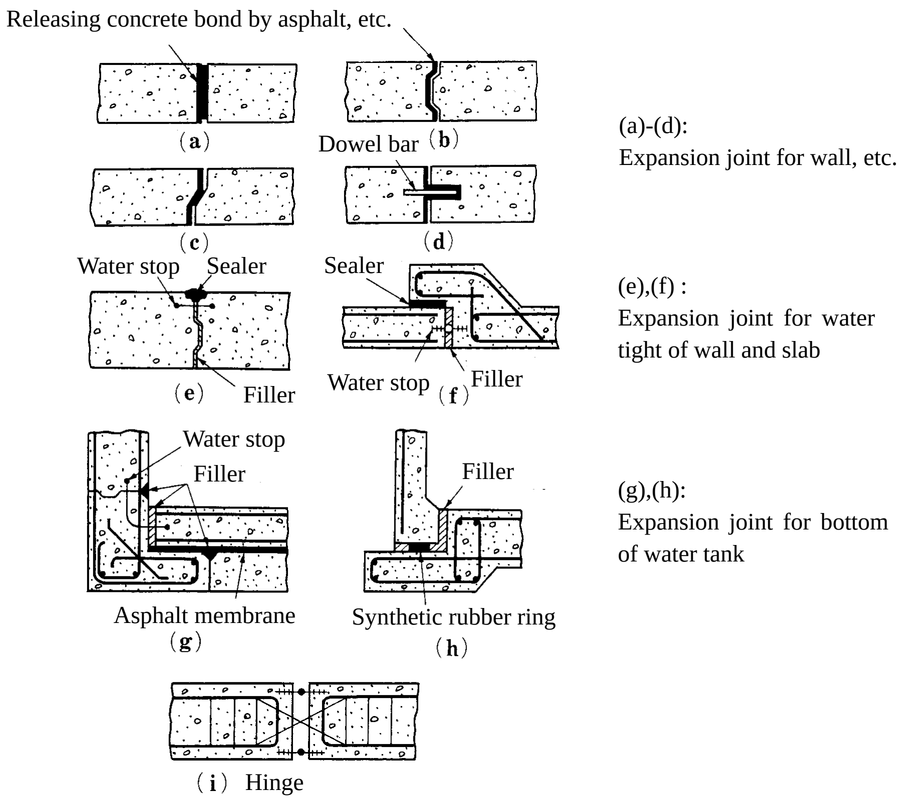


Fig. C14.7.1　Expansion joints

14.8　Crack Inducted Joints

When crack inducted joints is located to control cracking, the type and location of crack inducted joints shall be determined not to decrease the strength and function of the structure.

[Commentary]　The deformation of concrete structure might be caused by the heat hydration of cement, the temperature change, the drying shrinkage, and so on.　The concrete crack might occur if the deformation is restricted.　Therefore, the crack inducted joints are installed at prescribed intervals to concentrate the concrete crack on the predicted location.　When the crack inducted joints are provided, the sufficient consideration is necessary as for the interval and rate of sectional loss caused by the crack inducted joints, the maintenance method of reinforcing bar for steel corrosion, the ensuring the prescribed concrete cover, the selection of material for the joint filler. In general, it is recommended that the interval of crack inducted joints is one to two times the height of cross section and that the rate of sectional loss is 30 to 50%.　Figure C14.8.1 shows the example of the crack induced joints.　These examples show that a notch in vertical direction on both surfaces of the wall and that bond of concrete surface is released by parting agent.　In addition, a vinyl chloride pipe, precast concrete or the iron plate, and so on, with the concentration of stress, are embedded into the wall section.　The ratio of sectional loss is defined as the ratio of the total length, given as the summation of notch depth and unbond depth, to the original wall thickness.

When the crack induced joint is provided in the water-tight structure, it is recommended that water stop is used. When the water leak from the crack induced joints and the steel corrosion of reinforcing bar are concerned, the treatments shall be done as shown in Fig. C14.8.2.

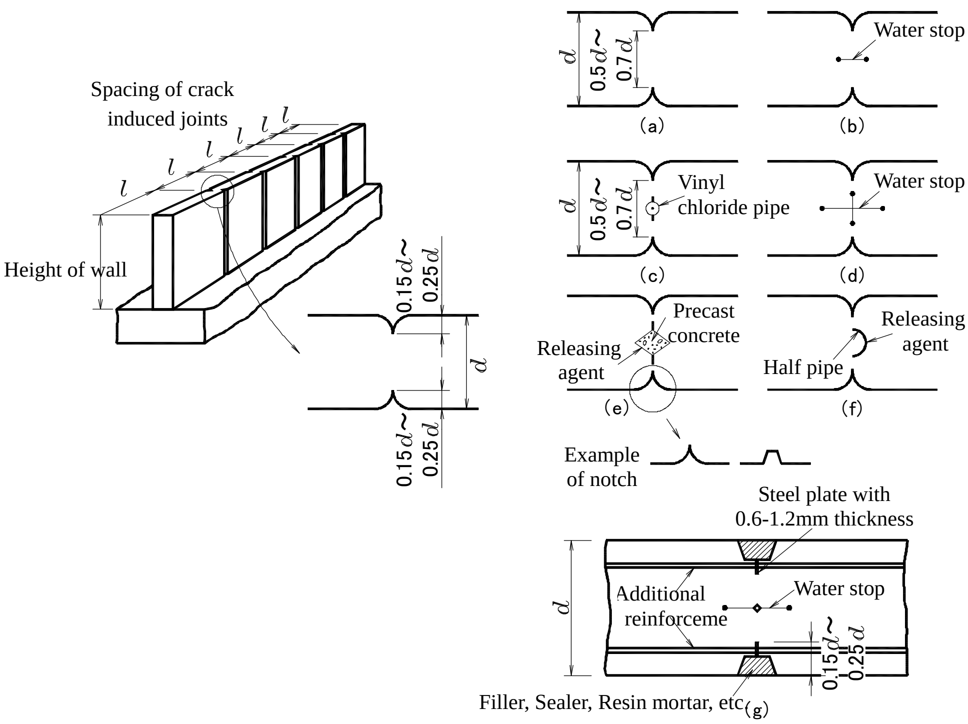


Fig. C14.8.1 Example of crack induced joints

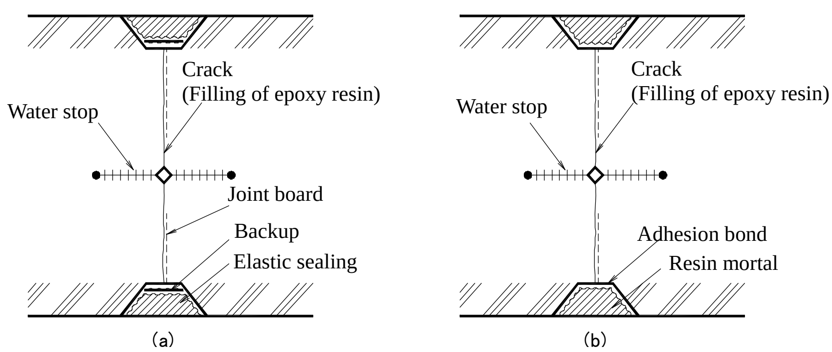


Fig. C14.8.2 Example of details of crack inducted joints

14.9 Water-Tight Structures

When water-tightness is required for reinforced concrete structures, arrangement of reinforcement, spacing and arrangement of construction or expansion joints, or other details shall be determined so as to prevent harmful cracks.

[Commentary] For reinforced concrete structures that require water-tightness, it is necessary to construct watertight concrete. It is especially important that steps are taken to minimize cracking at the design stages. Therefore, changes in temperature, shrinkage, differential settlement of foundation, will appropriately accounted for in the design stage, so as to decrease possibilities of cracking by such ways as appropriate detailing of reinforcement, limiting stresses in reinforcement, providing expansion or construction joints with appropriate spacing and location. Also in the design stage, provisions for water-stops, etc. at construction joints and expansion joints should be made to ensure water-tightness of the structure.

14.10 Drainage and Water Proofing

(1) For structures in contact with water, provisions for drainage and waterproofing shall be considered when needed.

(2) Water proofing shall, as a rule, be provided on the face subjected directly to water pressure.

[Commentary] (1) For reinforced concrete structures requiring water-tightness, before considering waterproofing, the possibility of providing appropriate drainage should be considered. For such structures as underpasses, for example, drainage may be the best way for maintaining water-tightness. Therefore, it is recommended that for such structures drainage should be considered first, and for cases where the objective is not satisfied with drainage alone, waterproofing may be considered to maintain the water tightness of the structure.

For structures in which water pressure act directly on one face and the other face is required to be dry, appropriate waterproofing should be provided considering cracking of concrete and other defects during construction.

(2) When waterproofing is provided at the face not directly subjected to water pressure, water may seep through the concrete and fill the space between the waterproofing and concrete face, and cause some damage to former. Therefore, waterproofing should, in principle, be provided on the face directly subjected to water pressure.

If the water between waterproofing and concrete face freezes, the affects are likely to be worse compared to a case without waterproofing. In cases where the structure is subjected to freezing and thawing, water proofing shall not be provided on the face not subjected to water pressure.

14.11 Protection of Concrete Surface

(1) In order to provide durability to portions subjected to abrasion, deterioration, impact, or other severe actions, concrete surfaces shall be protected using appropriate materials.

(2) Concrete surfaces exposed to environmental deterioration factors such as chloride attack, freezing and thawing actions and chemical attack should be protected with appropriate materials such as polymeric materials.

[Commentary] (1) For structures subject to running water with suspended sand, wave action, impact of traffic, or subject to chemical erosion, concrete surfaces need to be protected by such materials as timber, good quality stone, steel plates and polymer materials so that the portions subject to abrasion, corrosion, impact, or other severe actions may be rendered especially durable. When the actions are not severe, providing an additional concrete cover of 10mm may be considered as the countermeasure.

(2) Methods for enhancing the durability of structural components subjected to chloride attack, freeze-thaw cycles or chemical attack, include increasing the concrete cover over reinforcing bars and protecting the concrete surface with appropriate materials such as polymeric materials. In this case, it is necessary to substitute the surface protection for the appropriate time.

14.12 Haunches

Haunches shall be provided, in portions such as connections of rigid framed members, supports of fixed slabs or beams and supports of continuous slabs or beams. The effective height in design of the section where a haunch is provided may be determined, considering the dimension of the haunch. In general, only the portion of the haunch with an inclination less than 1/3 shall be considered effective.

[Commentary] Haunches are recommended to provide in portions such as connections of rigid framed members, supports of fixed slabs or beams and supports of continuous slabs or beams because stress-concentration tends to occur there.

14.13 Joint part of Members

The joint part of the columns, beams, and other members should be designed to ensure the sufficient load-carrying capacity before the members indicate plastic behavior.

[Commentary] It is assumed that the some members indicate plastic behavior in the seismic design to verify the Seismic Performance Grade 2 or Seismic Performance Grade 3. The structure cannot have required seismic performance if the joint part loses the load-carrying capacity before the members indicate plastic behavior. Therefore, the joint part of columns and beams is necessary to have higher load-carrying capacity than that of structural members. It is necessary that the dimension of cross section of the joint part is enlarged, the haunches are located, and the transverse reinforcing bars are arranged sufficiently, etc. In addition, the longitudinal reinforcement that enters the joint part from each member should be embedded into concrete sufficiently.

14.14 Structural Details of Members

Structural details of members shall be determined through examinations taking into consideration such factors as the shape and stiffness of the members, boundary conditions, load characteristics, the state of loading and construction workability.

[Commentary] Structural details and the minimum dimensions of members for beams, columns, rigid frames, arches, slabs, footings, shells and walls must be determined taking into consideration the shape and stiffness of the members, boundary conditions, load characteristics, the state of loading, construction workability, etc. The verification should be managed by the responsible engineer.

If such examinations are not conducted, structural details and the minimum dimensions of members may be determined in accordance with the Design: Standards, Part 1, of this Specification.

CHAPTER 15 PRESTRESSED CONCRETE STRUCTURE

15.0 Notation

A_c : **total cross-sectional area of concrete**

A_{cc} : **area of concrete in compressive zone of shear plain**

A_p : **cross-sectional area of prestressing tendon**

A_s : **cross-sectional area of tension reinforcement**

A_s' : **cross-sectional area of compression bar**

A_k : **area of shear key**

B : **outer diameter of steel shear key or width of concrete shear key**

d' : **distance from compression edge to compression bar**

d_p : **distance from compression edge to prestressing tendon**

d : **distance from compression edge to tension bar**

E_c : **Young's modulus of concrete**

E_p : **Young's modulus of tendon**

e_p : **distance between the centroid of member and the centroid of prestressing tendon**

e_s : **distance between the centroid of member and the centroid of tension bar**

e_s' : **distance between the centroid of member and the centroid of compression bar**

f_{puk} : **characteristic value for tensile strength of prestressing tendon**

f_{vk} : **limit value of shear stress that steel shear key can carry**

I_c : **total cross-sectional moment of inertia of concrete**

L : **embedment depth of steel shear key**

l : **length of tendon**

Δl : **setting length**

N : **number of shear key**

N : **number of tensioning times (number of groups of the tendon)**

n_p : **Young's modulus ratio of prestressing tendon to concrete** $(= E_p / E_c)$

$n_{s,j}, n_{s,k}$: **Young's modulus ratio of tension bar and compression bar to concrete at age**

t_j, t_k

n_{ps} : Young's modulus ratio of prestressing tendon to reinforcing bar $(= E_p / E_s)$

$P(x)$: prestressing force of considered cross section

P_i : prestressing force at prestressing work at the tensioning end of tendon

ΔP : loss of tension force due to set of tendon

$\Delta P_i(x)$: loss of prestressing force immediately after prestressing

$\Delta P_t(x)$: time dependent variation of prestressing force

T_c : total tension force in concrete

t : height of shear key

V_k : shear capacity of shear key

V_{ks} : shear capacity of steel shear key

α : angular change of the tendon in radian

$\Delta\alpha$: equivalent angular change per meter of a length of the tendon.

β : coefficient that indicates shear plain configuration (0–1)

γ : apparent relaxation ratio of tendon

γ_b : member factor

ε'_{cs} : shrinkage strain of concrete

λ : friction coefficient of unit length

μ : average friction coefficient of solid contact

σ_a : bearing stress that acts to single shear key at execution or ultimate limit state

σ'_{cdp} : compressive stress of concrete at the location of prestressing tendon by permanent load

σ'_{cpg} : concrete compressive stress due to prestressing at the centroid of prestressing tendons

σ'_{cpt} : compressive stress of concrete at the location of prestressing tendon by prestress force just after prestressing

σ_l : diagonal tensile stress

σ_{nd} : average compressive stress which acts to shear plain vertically

σ_{pt} : tensile stress of prestressing steel just after prestressing

σ_{sl} : maximum permissible value for tensile stresses in tension reinforcement

σ_x	:	**normal stress**
σ_y	:	**stress perpendicular to σ_x**
$\Delta\sigma_p$	:	**prestress loss in prestressing tendon**
$\Delta\sigma_{pr}$	:	**loss of tensile stress due to relaxation**
$\Delta\sigma_{pcs}$	:	**stress loss of prestressing tendon by creep and shrinkage of concrete**
θ	:	**angle between shear key and vertical line**
φ	:	**creep coefficient of concrete**
φ_j, φ_k	:	**creep coefficient of concrete loaded at age t_j ,and t_k**
τ	:	**shear stress due to shear and torsion**

15.1 General

(1) Provisions of this chapter describe general guidelines especially needed for design of ordinary prestressed concrete structures or their members.

(2) Level of prestressing shall be determined such that the structure or member fulfills its desirable functions and discharge its function in a safe and economical manner.

[Commentary] Use of prestressed concrete enables improvement in the crack characteristics at the serviceability limit state, and reduces the required cross sectional area with use of high strength steel, and thus, affords a wide range of options for many types of structures and construction methods. The designer of a prestressed concrete structure must follow the requirements of this Specification and verify, in accordance with the provisions of this chapter concerning requirements specific to prestressed concrete structures that the performance requirements related to safety, serviceability, etc., are met.

As far as treatment of the prestressing force introduced to concrete members for the purpose of design calculations is concerned, it may, in general, be treated as a load in the consideration of the serviceability limit state. At the ultimate limit state, only the indeterminate force may be considered as the effect of the prestressing force is included in calculating the ultimate strength of the cross-section.

When calculating response values, it is necessary to take moment redistribution into consideration and to model the prestressing force by a method suitable for the purpose of verification and for the analysis method used. In general, methods such as the method of treating prestressing force as a fixed external force and the method of directly modeling tendons as chord members (one-dimensional linear elements) can be applied. It is necessary to perform modeling, taking into consideration such factors as the anchoring locations of tendons and whether concrete members and tendons are bonded, so that the prestress that introduced into the concrete members can be allowed for appropriately.

(1): Provisions of this chapter shall be applicable to ordinary prestressed concrete structures or members using prestressing tendons. If high-strength materials outside the scope of Chapter 5 are used as when concrete having a characteristic compressive strength exceeding 100 N/mm^2 is used, design values for the materials must be considered separately. But provisions of this chapter are not to be applied to the following structures or members:

i) structures or members where the prestressing force is transferred by a method other than prestressing tendons. For example in cases where jacks are used to prestress, e.g. in arched members or in concrete pavement, or in cases where concrete is cast on the tension side of a steel girder, in which prior flexural moment has been induced, and which is released after hardening of the concrete.

ii) prestressed steel reinforced concrete structures or members; prestressed composite structures or members consisting of rolled sections, steel plates or built-up sections and concrete.

iii) prestressed concrete structures or members exposed to abnormal temperature. A range of the normal temperature may be considered from 0°C to 40°C.

iv) factory products such as prestressed concrete piles, prestressed concrete pipes, etc.

Means of introducing permanent prestress into concrete members can be classified, according to the timing of tensioning of tendons, into two types: the pretensioning method and the post-tensioning method. The post-tensioning method can be further classified into three types: the internal tendon method, the external tendon method and the combined use of the two methods.

In the internal tendon method, tendons are placed through concrete members. They are either tendons bonded to the concrete by use of grout to integrate the tendons with the concrete or tendons unbonded to the concrete. Structure or member prestressed by unbonded tendon is separated from those prestressed by external tendons, because the relative distance between concrete and unbonded tendon does not vary, and because unbonded tendon arrangement differs from the external tendon system. However, additional considerations should be required to the structure or member with unbonded tendon, such as the increase of crack width, reduction of flexural capacity, minimum amount of tendon, fatigue characteristics of anchorage, etc.

On the other hand, in the external tendon system described in this chapter the tendons are provided outside of the concrete member and transfer the prestress through anchorages and deviators. Arrangements for permanent rust-proofing and corrosion protection treatment of tendons are provided. In this chapter, tendons that can be used for the external tendon method are referred to as "external tendons," and this chapter deals external tendons whose effective depth is not greater than the depth of the member. Therefore, the provisions of this chapter may not be enough for the treatment of temporary prestressing during construction, additional external tendons for repair or reinforcement without prior planning, stay-cable of cable-stayed bridges whose stress is regulated because of the large stress change by fluctuated load. Furthermore, it may also not be enough for large eccentric tendons such as tendon's location excesses the effective height of girder, for fatigue by vibration, which is caused by direct effects of wind, is severe. For such cases, reference should be made to other codes or papers.

Furthermore, in the case of structures or members using external tendons, considerations other than those described in this chapter are required for the following cases.

- The increase of crack width because external tendons have no bond to the concrete member
- The fact that plane sections remain plane is not adopted to the external tendon
- Change in the prestressing force and change in the effective depth in the external tendon because of deformation in the concrete member
- Minimum tendon content

- Fatigue of tendon
- Protection and rust-proofing of tendon

(2): The level of prestress may generally be determined from considerations of flexural cracking limit state corresponding to the function of structure.

15.2 Classification of Prestressed Concrete

(1) This chapter classifies prestressed concrete structures into PC (prestressed concrete) structures and PRC (prestressed and reinforced concrete) structures, and describes standard verification methods applicable to these structures.

(2) PC structures do not permit cracking in serviceability-related verification and are structurally designed to control the edge stress in concrete by introducing prestress.

(3) PRC structures permit cracking in serviceability-related verification and are structurally designed to control crack width by installing deformed reinforcing bars and introducing prestress.

[Commentary] (1): The behavior of a prestressed concrete structure may be radically different before and after the occurrence of cracking. Therefore, depending on whether cracking is permitted under normal service conditions, methods of calculating stress in materials and methods of strengthening to resist bending moment such as the use or nonuse of reinforcement for crack distribution vary considerably. Therefore, prestressed concrete structures are classified, according to the definitions of Items (2) and (3), into PC (prestressed concrete) structures and PRC (prestressed and reinforced concrete) structures, and it is verified, by the method described in this chapter, that the degree of prestressing has been determined appropriately.

Flexural and other behavior at failure of prestressed concrete structures varies depending on whether tendons are installed inside or outside the concrete members. Similarly, flexural and other behavior varies depending on whether the tendons used are bonded to the concrete members. Therefore, it is necessary to classify prestressed concrete structures into PC structures and PRC structures and to classify verification methods according to tendon arrangement and whether tendons are bonded to the concrete. Therefore, this chapter classifies tendons into three types, namely, internal tendons, unbonded prestressing tendons and external tendons, and also classifies prestressed concrete structures according to the concept shown in Fig.C15.2.1. In this case, prestressing tendons used in the pre-tensioning method can generally be classified as internal tendons because those prestressing tendons can be treated like internal tendons used in the post-tensioning method. Pregrouted prestressing tendons are made by filling sheaths with slow- and cold-setting resin in place of ordinary grout used for prestressed concrete, keeping the unbonded state until prestress is introduced, bonding after resin set over time. The tendons can also be classified as internal tendons because after the filling material sets, those tendons can be treated like internal tendons used in the post-tensioning method.

Prestressed concrete structures should be classified each member or each cross section. By using PC structures and PRC structures together according to the purpose and performance requirements, rational prestressed concrete structures can be constructed.

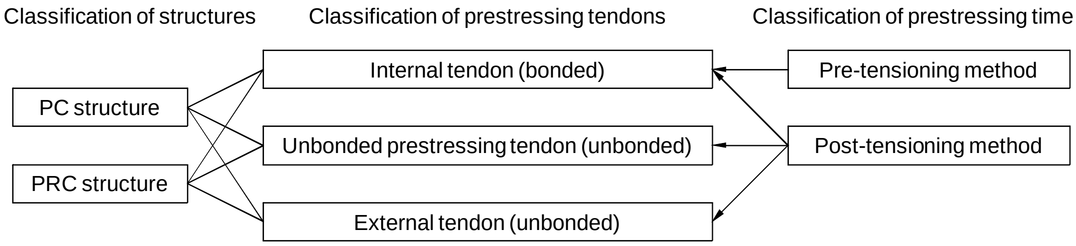


Fig. C15.2.1 Classification of prestressed concrete structures

(2): Serviceability verification for PC structures does not permit the occurrence of cracking. In PC structures, the edge stress and diagonal tensile stress in the concrete are restricted in accordance with the requirements specified in Section 15.7.2 by introducing prestress. Stress of material may be calculated by assuming a fully effective cross section. As a general rule, the restraining effect of reinforcement is taken into account as an influence on the creep and shrinkage of concrete. However, in general, the restraining effect of reinforcement does not need to be taken into consideration because the amount of reinforcement is relatively small.

(3): Serviceability verification for PRC structures permits the occurrence of cracking. In PRC structures, crack width is controlled by placing deformed bars and introducing prestress. To be more specific, as in reinforced concrete structures, crack spacing is controlled by using the crack distribution effect of deformed bars, and increases in steel stress are controlled by prestressing. Therefore, as a general rule, examinations for flexural cracking look at the tensile reinforcement located closest to the concrete surface, and flexural crack width is calculated by using the value of 1.0 for the factor (k_1) expressing the influence of the steel surface configuration on crack width in Eq. (7.4.4). The restraining effect of both prestressing tendons and reinforcing bars is taken into consideration as an influence on the creep and shrinkage of concrete.

For PRC structures, in theory, any degree of prestressing can be specified by setting load state to be verified by stages according to such factors as the frequency and duration of variable loads and accidental loads, the percentage of permanent loads in all loads, determining the limit state at the tension edge in each load state. PRC structures cover the entire intermediate range in the spectrum between reinforced concrete structures and PC structures. In general, if a highly accurate verification method taking into consideration the influence of cracking is as-needed used, the scope of application of the method is very wide.

However, no verification method has been proposed that makes it possible to allow for the influence of cracking over time at the loading stages and the subsequent stages and examine different limit state by stages. There is as yet no highly accurate verification method capable of allowing for those factors. There is as yet no highly accurate verification method capable of allowing for those factors. If a special verification method capable of allowing for the influence of cracking over time is not used, it is recommended that a limit state in which the tensile stress in the concrete does not exceed the limit value is set by assuming that cracking does not occur because of permanent loads, and it is verified by using, for example, a formula for calculating the amount of decrease in tensile stress in prestressing tendons derived by assuming a fully effective cross section of concrete that the degree of prestressing has been determined appropriately. The limit value of the tensile edge stress may be determined according to the flexural crack strength of concrete.

By determining such limit state, the occurrence of cracking at an early age of concrete can be prevented. Therefore, the growth over time of the width of cracks caused by variable loads, etc., can be controlled. If a limit state with a limit value of the tensile stress of zero (a state of decompression in which the occurrence of tensile stress at the tension edge is prevented) is chosen, it can be thought that even if cracks occur because of variable loads, etc., the cracks practically close under permanent loads. Therefore, the limit value of the tension edge stress in the concrete should be set appropriately according to the frequency and duration of variable loads and accidental loads the percentage of permanent loads in all loads, etc., as mentioned earlier. If the influence of restraint on the shrinkage of concrete is thought to be great, it should be taken into consideration.

15.3 Prestressing Force

(1) Prestressing force shall be calculated from Eq. (15.3.1).

$$P(x) = P_i - [\Delta P_i(x) + \Delta P_t(x)] \quad \textbf{(15.3.1)}$$

where, $P(x)$ **: prestressing force of considered cross section**

P_i **: prestressing force at prestressing work at the tensioning end of tendon**

$\Delta P_i(x)$ **: loss of prestressing force immediately after prestressing to be calculated taking into account the influence of the following:**

I) elastic deformation of concrete

II) friction between tendon and sheath

III) Set during anchoring of tendon

IV) others

$\Delta P_t(x)$ **: time dependent variation of prestressing force; to be determined taking into account the influence of the following:**

V) relaxation of prestressing tendon

VI) creep of concrete

VII) shrinkage of concrete

VIII) restraint by reinforcement

(2) If statically indeterminate force occurs because of prestressing force, the response value may be calculated by using the prestressing force calculated from Eq. (15.3.1) as the characteristic value of the load.

[Commentary] (1): The items whose influence needs to be taken into account when calculating the loss in prestressing force, $\Delta P_i(x)$ and $\Delta P_t(x)$, are as follows:

I) Influence of elastic deformation of concrete

In the pretensioning method, usually, prestress is introduced by releasing more than one tendon pretensioned to a predetermined level. Therefore, loss of tension force in the tendons due to the elastic deformation of the concrete must be taken into consideration. The loss in the average tensile stress in the prestressing tendons may be calculated from Eq. (C15.3.1).

In construction carried out by the post-tensioning method, usually, tendons are tensioned one by one or group by group. Consequently, the concrete undergoes elastic deformation each time a tendon or tendon group is tensioned, and the tension force in the previously tensioned tendons gradually decreases under the influence of the tendons tensioned subsequently. In such cases, the loss in the average tensile stress in the internal tendons and unbonded prestressing tendons may be calculated from Eq. (C15.3.2). The amount of decrease in the average tension in the external tendons may be calculated from Eq. (C15.3.3) by assuming that it does not vary among anchoring points.

(i) For prestressing tendon in the pretensioning method

$$\Delta\sigma_p = n_p \sigma'_{cpg} \tag{C15.3.1}$$

(ii) For internal tendon and unbonded prestressing tendon

$$\Delta\sigma_p = \frac{1}{2} n_p \sigma'_{cpg} \frac{N-1}{N} \tag{C15.3.2}$$

where, $\Delta\sigma_p$: prestress loss in prestressing tendon

n_p : Young's modulus ratio of prestressing tendon to concrete $n_p = E_p / E_c$

σ'_{cpg} : concrete compressive stress due to prestressing at the centroid of prestressing tendons

N : number of tensioning times (number of groups of the tendon)

(iii) For external tendon

$$\Delta P_p = \frac{1}{2} E_p A_p \frac{\Delta l}{l} \frac{N-1}{N} \tag{C15.3.3}$$

where, ΔP_p : loss of prestressing force in external tendon

E_p : Young's modulus of external tendon

A_p : cross-sectional area of external tendon

Δl : amount of movement of concrete member between anchoring points due to initial tension P_i

l : distance between anchoring points

N : number of tensioning times (number of groups of the tendon)

However, if the anchoring point of the external tendon is located at or near the centroid and longitudinal deformation is not restrained, the strain between anchoring points, $\Delta l / l$, in Eq. (C15.3.3) may be calculated from Eq. (C15.3.4).

$$\frac{\Delta l}{l} = \frac{P}{E_c A_c} \tag{C15.3.4}$$

where, P : prestressing force introduced into external tendon

E_c : Young's modulus of concrete

A_c : total cross-sectional area of concrete

II) Influence of friction between tendon and sheath

In the post-tensioning method, as the distance from the tensioning end (jack location) increases mainly, tensile force in the tendon decreases because of the friction between the tendon and the sheath and the friction between the external tendon and the protection tubes at the deviators. In general, factors causing such decreases in tension can be classified into those related to changes in the angle of the centroidal line and those related to tendon length. Therefore, the tensile force in the tendon in the design cross section may be calculated from Eq. (C15.3.5) in view of these influences. In the external tendon method, all external cables other than those in the anchorage zones or the deviator zones are placed outside the concrete member. Therefore, the influence of friction in this section does not need to be taken into consideration.

$$P_x = P_i \cdot e^{-(\mu\alpha+\lambda x)} \tag{C15.3.5}$$

where, P_x : tension force in tendon in design cross section

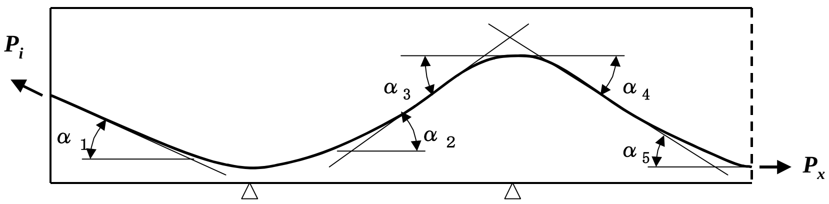
P_i : tension force in tendon at jack location

μ : friction coefficient per radian of angle change

α : angular change (radian) (See Fig. C15.3.1).

λ : friction coefficient of unit length

x : length of tendon from tensioning end to design cross section



$\alpha = \Sigma^n{}_1\, \alpha_i = \alpha_1 + \alpha_2 + \alpha_3 + \alpha_4 + \alpha_5 + \cdots\cdots + \alpha_n$

Fig. C15.3.1 Angular change of centroidal line of tendon

Usually, the values of μ and λ need to be determined experimentally because they vary depending on such factors as the state of the inside surface of the sheath and the type of prestressing tendon. However, if steel sheaths are used, the values shown in Table C15.3.1 may be used. For the friction coefficient in the anchorage and deviator zones of external tendon, the values shown in Table C15.3.2 may be used according to the type of protection tube material. However, if a special type of sheath or spacer is used to reduce friction, or if prestressing tendon with special surface processing with resin, coating film, etc. is used, or if friction is cut off by impacting the prestressing tendon, the friction coefficient may be determined separately by referring to relevant data such as conventional experiment results.

Table C15.3.1 Friction coefficient (steel sheath)

Type	μ	λ (unit: l/m)
Prestressing tendon, prestressing strand	0.30	0.004
Prestressing bar	0.30	0.003

Table C15.3.2 Friction coefficient of prestressing steel and protection tube (prestressing tendon, prestressing strand)

	μ	λ (unit: l/m)
Steel pipe	0.30	0.004
Polyethylene pipe	0.15	0.004

In the internal tendon method, if the length of tendon is about 40 m or less and the angular change of the tendon is about 30 degrees or less, Eq. (C15.3.6) may be used as an approximation:

$$P_x = P_i(1 - \mu\alpha - \lambda x) \qquad \text{(C15.3.6)}$$

It is also possible to calculate P_x by replacing the term λ to allow for the influence of tendon length in Eq. (C15.3.6) with the additional angular change per meter of tendon length. In this case, λ may be calculated from Eq. (C15.3.7).

$$\lambda = \mu\Delta\alpha \qquad \text{(C15.3.7)}$$

where, $\Delta\alpha$: additional angle change per meter of a length of the tendon (radian).

III) Influence of set during anchoring of tendon

The term "set" means the movement of the tendon drawn to the anchorage with anchoring wedges. If set occurs when a tendon is anchored, the loss of the tension force in the tendon must be allowed for. Because anchoring wedges cause a relatively large setting length, it is necessary to check on the setting length in advance, and to examine the loss of tension force in the tendon and the zone of influence of the set by assuming a somewhat larger-than-expected length of set to be on the safe side. Since the setting length varies among anchorage elements, it is necessary to determine the setting length for each anchoring device by referring to literature such as the Recommendations for Design and Construction of Prestressed Concrete Structures (1991).

If there is no friction between the tendon and the sheath when internal tendons or unbonded prestressing tendons are used, the loss of tension force is constant over the length of the tendon as shown in Fig. C15.3.2 (a). In this case, the area enclosed by a', b', o', o", b" and a" can be calculated as $\Delta P \times l$, and the loss of tension force in the tendon due to set can be calculated directly from Eq. (C15.3.8):

$$\Delta P = \frac{\Delta l}{l} E_p A_p \qquad \text{(C15.3.8)}$$

where, ΔP : loss of tension force due to set of tendon

Δl : setting length

l : length of tendon

E_p : Young's modulus of tendon

A_p : cross section area of prestressing tendon

If there is friction between the tendon and the sheath, the loss of tension force in the tendon may be determined by the graphical method by following the Steps 1) to 4) described below assuming that the friction during the tensioning operation and the friction that occurs when the tendon is relaxed because of set are the same.

1) Determine from Eq. (C15.3.5) the distribution a', b' and o' of the tension in the tendon immediately before anchoring when assuming that the initial tension force at the tensioning end is P_i, taking into account the friction between the tendon and the sheath.

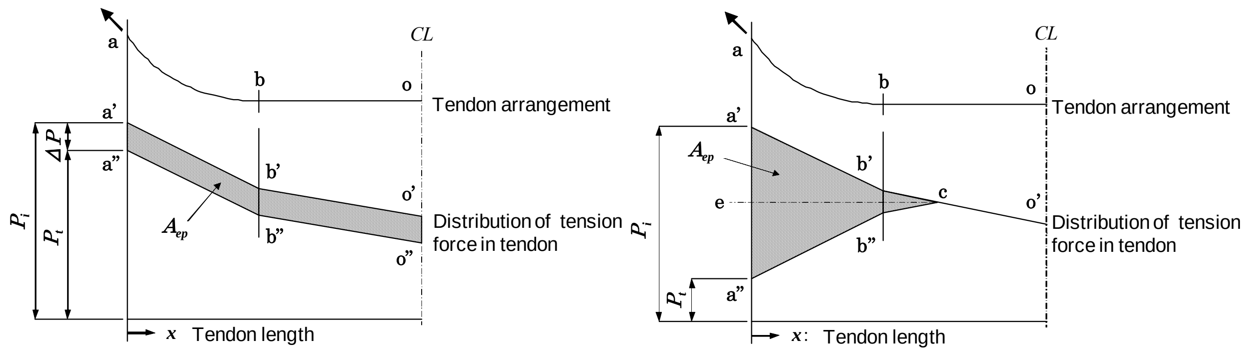
2) Assume the location c under the influence of set and find a'', b'' and c that are symmetric with a', b' and c with respect to the horizontal axis c-e.

3) Calculate the area enclosed by a', b', c, b'' and a'', find the location c at which $A_{ep} = E_p A_p \Delta l$ (see Eq. (C15.3.9)) holds and determine the distribution of a'', b'', c and o' of the tension force in the tendon immediately after anchoring.

4) Find the tension in the tendon in an arbitrary design cross section from the distribution a'', b'', c and o''. The tension force in the tendon at the tensioning end is P_i immediately after anchoring.

$$\Delta l = \frac{A_{ep}}{A_p E_p} \tag{C15.3.9}$$

where, A_{ep} : area of the shaded region shown in Fig. C15.3.2 (b) (= area enclosed by a', b', c, b'' and a'')

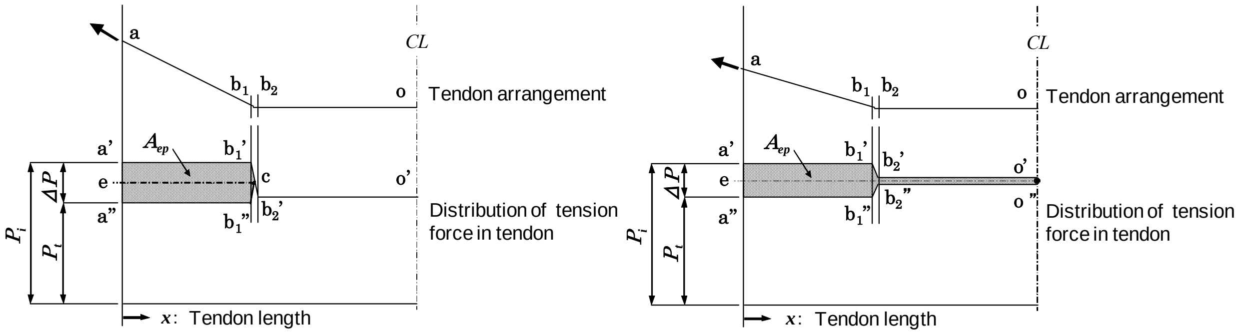


(a) In the case where there is no friction between tendon and sheath

(b) In the case where there is friction between tendon and sheath

Fig. C15.3.2 Distribution of tension force in internal tendon and unbonded prestressing tendon after the occurrence of set

The loss of tension force due to set that occurs when an external tendon is anchored may be calculated by a method similar to the method used for internal tendons. However, since the free part of the tendon is not affected by friction, the tension force in that section decreases uniformly because of set. In general, the length of the section in which an external tendon is in contact with deviators (Section B1–B2 in Fig. C15.3.3) is short. Therefore, the influence of set may reach the halfway point of an external tendon (Location c) as shown in Fig. C15.3.3 (b) if angular change of the centroidal line of the external cable is small or if the setting length is large. When considering the influence of set in connection with an external tendon, the tension force in the tendon in an arbitrary design cross section can be determined by calculating the area enclosed by a', b_1', c, b_1" and a" or the area enclosed by a', b_1', b_2', o', o", b_2", b_1" and a" on the basis of Eq. (C15.3.9) (see Fig. C15.3.3), paying careful attention to the considerations mentioned above



(a) In the case where c is located in section b_1- b_2

(b) In the case where c is not located in section b_1- b_2

Fig. C15.3.3 Distribution of tension force in external tendon after the occurrence of set

IV) Other influences

Other influences such as the deformation (plastic deformation) of precast concrete block joints should be taken into consideration.

V) Influence of relaxation of prestressing tendon

The prestress loss in prestressing tendon due to relaxation may be calculated from Eq. (C15.3.10):

$$\Delta\sigma_{pr} = \gamma\sigma_{pt} \tag{C15.3.10}$$

where, $\Delta\sigma_{pr}$: prestress loss in prestressing tendon due to relaxation of prestressing steel

γ : apparent relaxation ratio of prestressing steel (See Table 5.3.1.)

σ_{pr} : tensile stress of prestressing steel just after prestressing

If the external tendon method is used, usually the loss of prestressing force due to the creep and shrinkage of concrete is smaller than in the internal tendon method. In such cases, the apparent relaxation ratio tends to become greater than the values shown in Table 5.3.1. For this reason, if the initial tensile stress is between 50% and 75% of the tensile strength, the apparent relaxation ratio γ of an external tendon used to calculate the loss of prestressing force should be calculated from Eq. (C15.3.11) after determining the net relaxation ratio γ0 from Table C5.3.2 and Fig. C5.3.1.

$$\gamma = \gamma_0\left(1-2\left(P_t - P_e\right)/P_t\right) \tag{E15.3.11}$$

where, γ : apparent relaxation ratio of external tendon

γ_0 : net relaxation ratio of prestressing steel

$P_t - P_e$: loss of prestressing force due to creep and shrinkage

P_t : tension force in external tendon immediately after prestressing

P_e : prestressing force in external tendon determined to allow for the influence of creep and shrinkage

6) Creep of concrete, 7) Shrinkage of concrete

As a general rule, the loss of tensile stress in a tendon due to the creep and shrinkage of concrete is to be determined through an appropriate creep analysis taking the influence of reinforcement into consideration. The loss of tension force in an external tendon may be calculated from Eq. (C15.3.12):

$$P_e = P_t - \frac{n_p \rho_p \cdot \varphi \cdot P_t + E_p \cdot A_p \cdot \varepsilon'_{cs}}{1 + n_p \rho_p (1 + \chi\varphi)} \tag{C15.3.12}$$

where, P_e : prestressing force in external tendon determined taking into account the influence of creep and shrinkage

P_t : prestressing force immediately after introducing into external tendon

n_p : Young's modulus ratio of prestressing tendon to concrete; $n_p = E_p / E_c$

ρ_p : cross-sectional area ratio of tendon; $\rho_p = A_p / A_c c$

ϕ : creep coefficient of concrete

χ : aging factor; usually, $\chi = 0.8$

ε'_{cs} : shrinkage strain of concrete

E_p : Young's modulus of external tendon

E_c : Young's modulus of concrete

A_p : cross-sectional area of external tendon

A_c : total cross-section area of concrete

For the purpose of deriving this equation, it is assumed that the tension changes in the external cable due to flexural strain in the cross section can be ignored because Navier's hypothesis cannot be applied to external tendons. It is also assumed that the strain changes in external tendon is the sum of the shrinkage strain ε'_{cs} in the concrete and the longitudinal creep strain in the whole cross section of concrete due to prestressing force. In this case, the strain changes in concrete and the strain changes in external tendon can be expressed by Eq. (C15.3.13) and Eq. (C15.3.14), respectively, and Eq. (C15.3.12) is an equation derived by developing these equations.

$$\Delta\varepsilon_c(t=\infty)=\frac{P_t}{E_c A_c}\varphi-\frac{P_t-P_e}{E_c A_c}(1+\chi\varphi)+\varepsilon'_{cs} \tag{C15.3.13}$$

$$\Delta\varepsilon_p(t=\infty)=\frac{P_t-P_e}{E_p A_p}-\Delta\varepsilon_c(t=\infty) \tag{C15.3.14}$$

where, $\Delta\varepsilon_c(t=\infty)$: strain change in concrete since immediately after introducing until $t=\infty$

$\Delta\varepsilon_p(t=\infty)$: strain change in external tendon since immediately after introducing until $t=\infty$

Equation (C15.3.12) allows only for the influence of the creep and shrinkage of concrete on the prestressing force in external tendons. If both internal and external tendons are used, it is necessary to calculate the prestressing force in the external tendons by an appropriate method, taking into account the influence of the creep of the concrete due to the prestressing force in the internal tendons.

If the loss of the prestressing force in an external tendon due to the creep and shrinkage of concrete is to be calculated exactly, it is necessary to determine the strain change at the centroid of the external tendon due to the creep and shrinkage of the concrete member over the entire length of the external tendon and calculate, from the average strain thus determined, the loss of the tension force in the external tendon. This requires very complex and laborious calculation, but recently it has become possible to model a structure or an external tendon according to the configuration of the structure, support conditions, etc., and calculate the prestressing force in the external tendon through structural analysis. The three methods shown in Fig. C15.3.4 are widely used, and they are briefly described below. Because these methods can also be used for other applications such as the calculation of design sectional forces, those methods should be used rationally according to the limit states considered at the design stage.

(i) In the member evaluation method, an external tendon is modeled as a chord member, and the initial tension force is given as an equivalent axial strain. Because the deformation of the chord member follows the deformation of the girder, the influence of the creep and shrinkage of concrete and the influence of loads can be analyzed directly as the tension changes in the chord member.

(ii) In the external force evaluation method, prestressing force is let to act as a concentrated load or a distributed load at the anchoring locations and the deviator locations of external tendon. The action point of each load and the action direction are clearly indicated, and the prestressing force can be clearly evaluated as an external force in the free part of the tendon.

(iii) In the internal force evaluation method, prestressing force is let to act in the analysis model as internal forces (axial force N, eccentric moment M), and statically indeterminate forces due to the prestressing force are calculated. This is based on the conventional internal tendon method, and prestressing force (internal force) due to the external tendon and negative shear force need to be considered separately.

Each type of analysis model and the external tendon prestressing force calculation approaches described in this section are summarized in Table C15.3.3.

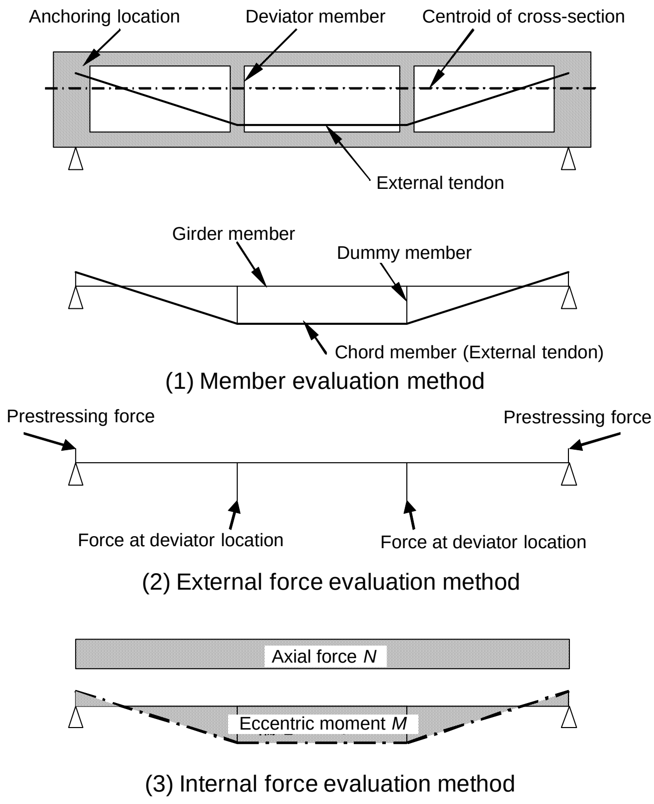


Fig. C15.3.4 Modeling of member with external tendon

(2): In the case of a statically indeterminate structure, it is possible to prevent the occurrence of statically indeterminate force due to prestressing force by using appropriate prestressing steel arrangements. However, if this method is used, the deformation of the member due to prestressing force is restrained so that statically indeterminate force occurs. Therefore, when considering stresses in materials, this statically indeterminate force must be taken into consideration. Statically indeterminate force due to prestressing force may be greatly affected by a cross-sectional change if the member has a non-uniform cross section. This needs to be kept in mind.

If prestressing force is given at different times or if the degree of redundancy of the structure changes during the prestressing period, statically indeterminate force changes with the progress of creep of concrete. In such cases, the influence of such changes must be taken into consideration. To be more specific, for example, if a continuous girder is constructed by connecting together a number of simple girders erected earlier or if a continuous girder is constructed by joining the ends of cantilever girders erected earlier, the degree of redundancy changes during the construction of the structure. If the structural system changes during construction in this way, the creep deformation of the structural system before the change is restrained so that new statically indeterminate forces may occur or may changes with the progress of creep. These need to be taken into consideration in the verification process as the influence of the creep and shrinkage of concrete as indicated in Chapter 6. In the case of a prestressed concrete structure, it is also necessary to take the influence of prestressing force into consideration.

Table C15.3.3 Analysis models and external tendon prestressing force calculation approaches in different evaluation methods

	(1) Member evaluation method	(2) External force evaluation method	(3) Internal force evaluation method
(1) Loss of prestressing force immediately after prestressing			
1) Elastic deformation of concrete	Eq. C15.3.3, Eq. C15.3.4	Ditto	Ditto
2) Influence of friction	Eq. C15.3.5	Ditto	Ditto
3) Set during anchoring	Eq. C15.3.9	Ditto	Ditto
(2) Loss of prestressing force over time			
1) Relaxation of external tendon	This can be allowed for as axial strain (thermal load), but the relaxation ratio is calculated from Eq.C15.3.11.	Eq. C15.3.10, Eq. C15.3.11	Ditto
2) Creep of concrete 3) Shrinkage of concrete	The influence of the creep and shrinkage of concrete can be analyzed directly as tension changes in a chord member.	Eq. C15.3.12	Ditto
4) Restraint by reinforcement	This can be allowed for by using a model and analytical tools capable of analyzing the influence of reinforcement.	To be considered separately	Ditto
(3) How to express prestressing force in analysis model	Expressed as axial strain (thermal load)	Expressed as concentrated load or distributed load	Prestressing forces (N, M) are simply added together. Statically indeterminate force is determined by letting prestressing force act as internal force.
(4) Calculating method of prestressing force in design cross section	Automatic calculation can be done by letting the prestressing force calculated in (1) act in accordance with (3).	The prestressing forces calculated in (1) and (2) are let to act in accordance with (3).	Ditto

15.4 Calculation of Response Values

15.4.1 General

(1) Response values used for the performance verification of a prestressed concrete structure shall be calculated in accordance with Chapter 7 after classifying the structure either as a PC structure or a PRC structure.

(2) Design response values used for verification related to serviceability or verification related to fatigue failure may be calculated by the methods describing in this section by using design sectional forces, etc.

[Commentary] This section describes methods for calculating design stresses for materials and design crack width from design sectional forces among the response values used for verification related to the serviceability and fatigue failure of prestressed concrete structures. If sectional forces are calculated from the results of analyses such as finite element analyses using the constitutive equations for the stress–strain relationship for materials when calculating design crack width, Section 7.4.2 must be followed.

Other response values such as the displacement and deformation of members must be calculated in accordance with Section 7.4.5, etc., taking the influence of prestress into consideration.

15.4.2 Design stresses for materials due to bending moment and axial force

(1) Design stresses for concrete and steel due to bending moment and axial force may be calculated on the basis of the assumptions listed below. Stress in concrete and the tendon under variable load should be considered as acting in addition to those calculated in (2).

(i) Fiber strain is proportional to the distance from the neutral axis of the member cross section.

(ii) Concrete and reinforcing bar are generally treated as elastic solid.

(iii) In the case of PC structure, the gross cross section of concrete is effective.

(iv) In the case of PRC structure, tensile stress in concrete may generally be neglected.

(v) Young's modulus of concrete and reinforcing bar is calculated in accordance with Section 5.2.5 and 5.3.4, respectively.

(vi) In the case of tendons bonded with concrete, any increase in strain as the same as that for concrete at the same location.

(vii) Area of sheath in the longitudinal direction shall not be considered to be effective cross section.

(viii) The modulus of the composite section made of the prestressing tendon and concrete shall be calculated considering the ratio of Young's modulus of the prestressing tendon to concrete.

(2) Stresses in concrete and the tendon under permanent load shall be calculated considering the effect of the tendon's relaxation, creep and shrinkage of concrete, and the restraint due to reinforcing bars.

[Commentary] (1): As in the conventional method, design stresses for the materials of a PC structure due to bending moment and axial force may be calculated by a method based on the elastic theory assuming a fully effective cross section of concrete. Cross-sectional parameters after prestressing tendons and concrete are integrated are determined taking into account Young's modulus ratio of the prestressing tendons and the concrete as mentioned in Item (viii) of this section. In general, the influence of reinforcement does not need to be taken into consideration because the amount of reinforcement is relatively small.

For PRC structures, it is recommended that a limit state in which cracking of concrete does not occur because of permanent loads be set. In this case, the stress due to permanent loads and prestress calculated by assuming a fully effective cross section of concrete and the stress due to variable loads calculated for a cross section with flexural cracks cannot be added together. Therefore, it is necessary to calculate design stresses for the materials of a PRC structure after the occurrence of flexural cracking by assuming a state in which the cross section with flexural cracks is being acted upon simultaneously by prestress, permanent loads and variable loads. In general, the calculation method described below may be used.

In a PRC structure before the occurrence of flexural cracking, usually, tensile stress is occurring in the prestressing tendons and compressive stress is occurring in the reinforcement. Therefore, design stresses for the materials of a PRC structure after the occurrence of cracking may be calculated as for a reinforced concrete structure reinforced with prestressing tendons and reinforcing bars, assuming that the axial force in each tendon in the state in which the concrete stress at each tendon location is zero acts externally as an eccentric axial force. This calculation may be done by the method described in Items 1) to 4) below.

In this method, in view of the basic assumption that cracking does not occur because of permanent loads, axial force in each tendon in the state in which concrete stress is zero is calculated, assuming that the state in which permanent loads are acting is the initial state. Because it may be assumed that the prestress acting in the cross section remains constant, calculation can be done regardless of the action of loads. Therefore, this method is practical and easy to use.

1) How to find the neutral axis

The location of the neutral axis is determined by using Eq. (C15.4.1):

$$\frac{M + N(d_p - d_N) + P_1'(d_s' - d_p) + P_1(d_p - d_s)}{N - P_1' + P_0 + P_1} = \frac{I_{cx} + n_s \cdot I_{sx}' + n_p \cdot I_{px} + n_s \cdot I_{sx}}{Q_{cx} + n_s \cdot Q_{sx}' - n_p \cdot Q_{px} - n_s \cdot Q_{sx}} + (d_p - x) \qquad \text{(C15.4.1)}$$

where, x : distance from compression edge to the neutral axis

d_s', d_p, d_s : distance from compression edge to the centroid of compression bar, the centroid of prestressing tendon and the centroid of tension bar

M : acting bending moment

N : acting axial force

d_N : distance from compression edge to the point at which axial load acts

P_0 : prestressing force in tendon in the state in which concrete stress at the centroid of the prestressing tendon is zero

P_1', P_1 : compression bar reaction and tension bar reaction in the state in which concrete stress is zero at the centroid of compression bar and the centroid of tension bar

Q_{cx}, I_{cx} : geometrical moment of area and geometrical moment of inertia of concrete in the compression zone with respect to the neutral axis

Q_{sx}', I_{sx}' : geometrical moment of area and geometrical moment of inertia of compression bar with respect to the neutral axis

Q_{px}, I_{px} : geometrical moment of area and geometrical moment of inertia of prestressing tendon with respect to the neutral axis

Q_{sx}, I_{sx} : geometrical moment of area and geometrical moment of inertia of tension bar with respect to the neutral axis

n_s : Young's modulus ratio of reinforcing bar to concrete $n_s = E_s / E_c$

n_p : Young's modulus ratio of prestressing tendon to concrete $n_p = E_p / E_c$

Reaction forces from the compression bar, prestressing tendon and the tension bar are calculated from Eq. (C15.4.2). However, as in the bending stress formula for ordinary reinforced concrete structures in Eq. (C15.4.1), σ'_{s0} is regarded as positive if the stress in the compression bar is compressive, and σ_{p0} and σ_{s0} are regarded as positive if the stresses in the prestressing tendon and the tension bar, respectively, are tensile. In such cases, as shown in Fig. C15.4.1, the reaction force P'_1 of the compression bar acts as an eccentric tensile axial force, and the reaction forces P_0 and P_1 of the prestressing tendon and the tension bar, respectively, act as eccentric compressive axial forces.

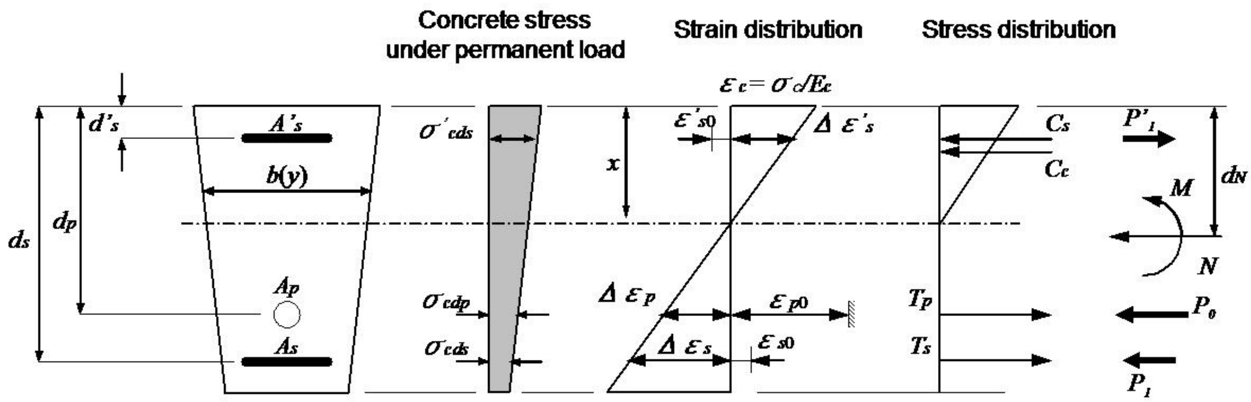


Fig. C15.4.1 Flexural stress distribution in a PRC structure

$$P'_1 = A'_s \cdot \sigma'_{s0} = A'_s\left(\Delta\sigma'_{scs} + n_s \cdot \sigma'_{cds}\right)$$

$$P_0 = A_p \cdot \sigma_{p0} = A_p\left(\sigma_{pd} + n_p \cdot \sigma_{cdp}\right) \tag{C15.4.2}$$

$$P_1 = A_s \cdot \sigma_{s0} = A_s\left(\Delta\sigma_{scs} + n_s \cdot \sigma_{cds}\right)$$

where, A'_s, A_p, A_s : cross-sectional area of compression bar, prestressing tendon and tension bar

$\sigma'_{s0}, \sigma_{p0}, \sigma_{s0}$: stresses in compression bar, prestressing tendon and tension bar under permanent loads after all prestress losses end

$\Delta\sigma'_{scs}, \Delta\sigma_{scs}$: stress variations in compression bar and tension bar by the creep and shrinkage of concrete (see Eq. (C15.4.8) and Eq. (C15.4.9)).

σ_{pd} : prestressing tendon stress under permanent loads calculated taking into consideration the influence of the relaxation of prestressing tendon and the creep and shrinkage of concrete, and restraint by reinforcement

$\sigma'_{cds}, \sigma_{cdp}, \sigma_{cds}$: concrete stress at the centroid of compression bar, the centroid of prestressing tendon and the centroid of tension bar under permanent loads

Because the left-hand side of Eq. (C15.4.1) is known and the right-hand side is a function along the neutral axis x, x can be calculated by solving a cubic equation. However, since the cross section to be considered is usually complex in shape, solving Eq. (C15.4.1) analytically is laborious. A method that can be used in such cases is to calculate the geometrical moment of area and the geometrical moment of inertia by assuming x and repeat calculations until the right-hand side of the equation is equal to the left-hand side.

2) Concrete stress

Concrete stress σ_c is calculated by substituting x in Eq. (C15.4.3).

$$\sigma_c = \frac{\left(N - P_1' + P_0 + P_1\right)\cdot x}{Q_{cx} + n_s \cdot Q_{sx}' - n_p \cdot Q_{px} - n_s \cdot Q_{sx}} \tag{C15.4.3}$$

3) Stress variation in reinforcement

Stress variation in the compression bar, $\Delta\sigma_s'$, and stress variation in the tension bar, $\Delta\sigma_s$, from the state in which concrete stress at the centroid of the reinforcement is zero are calculated from Eq. (C15.4.4) and Eq. (C15.4.5), respectively.

$$\Delta\sigma_s' = n_s \cdot \sigma_c \cdot \frac{x - d_s'}{x} \tag{C15.4.4}$$

$$\Delta\sigma_s = n_s \cdot \sigma_c \cdot \frac{d_s - x}{x} \tag{C15.4.5}$$

4) Tensile stress variation in prestressing tendon

Tensile stress variation in prestressing tendon, $\Delta\sigma_p$, from the state in which concrete stress at the centroid of the reinforcement is zero is calculated from Eq. (C15.4.6):

$$\Delta\sigma_p = n_p \cdot \sigma_c \cdot \frac{d_p - x}{x} \tag{C15.4.6}$$

The calculation of the increase in strain in unbonded prestressing steels or external tendons must be considered separately by using, for example, the member evaluation method shown in Table C15.3.3 because Bernoulli-Navier's hypothesis cannot be applied. If these methods are not used, concrete stress may be calculated by regarding as a PC structure or PRC structure acted upon by eccentric axial force due to prestressing force, ignoring the increase in stress in the unbonded prestressing steels and external tendons.

(2): If internal tendons are used, in order to calculate the stresses in the concrete and steel of a PC structure or PRC structure due to permanent loads, the decrease over time in the tensile stress in the prestressing tendons may be calculated by either of the methods described in Items 1) and 2) below.

If unbonded prestressing steel or external tendons are used, in theory, it is possible to calculate the strain variation at the centroid of the tendons due to the deformation of the concrete member and then calculate the decrease over time in the tensile stress in the tendons from the average strain thus determined. If all tendons are used as external tendons, the decrease in the tensile stress in the external tendons may be calculated in accordance with Section 15.3. However, if unbonded prestressing tendons or external tendons are used or if these are used in conjunction with internal tendons, in general the decrease in the tensile stress in the tendons may be calculated by using the methods described in this section for the three reasons listed below:

(i) Calculation by the theoretical method mentioned above would be very complex and laborious.

(ii) Under normal service conditions, the deformation of a member is very small. Therefore, the influence of strain variations at the tendon location is thought to be small.

(iii) The decrease in the tensile stress in external tendons due to the creep and shrinkage of concrete is smaller than when internal tendons are used.

1) PC structure

For PC structure, too, it is a general rule to take the influence of reinforcement restraint into consideration, but in general the influence of reinforcement restrain does not need to be considered because only a small amount of reinforcing steel is used in a PC structure. In this case, the decrease in the tensile stress in the prestressing steel may be calculated from Eq. (C15.4.7).

The standard method of calculation is to use the creep coefficient of plain concrete and shrinkage strain, but a creep coefficient and shrinkage strain determined taking into consideration the influence of the relatively small amount of longitudinal reinforcement in the concrete may be used. This is based on the empirical knowledge gained by using the verification method combining these design values and Eq. (C15.4.7). For example, if a considerable amount of empirical knowledge such as the knowledge about PC structures in prestressed concrete bridges is available, a creep coefficient and shrinkage strain determined appropriately according to such empirical knowledge may be used.

$$\Delta\sigma_{pcs} = \frac{n_p \cdot \varphi(\sigma'_{cpt} + \sigma'_{cdp}) + E_p \cdot \varepsilon'_{cs}}{1 + n_p \cdot \dfrac{\sigma'_{cpt}}{\sigma_{pt}} \cdot \left(1 + \dfrac{\varphi}{2}\right)} \tag{C15.4.7}$$

where, $\Delta\sigma_{pcs}$: loss of stress in the prestressing tendon due to creep and shrinkage of concrete

φ : creep coefficient of concrete

ε'_{cs} : shrinkage strain of concrete

n_p : ratio of Young's modulus of prestressing tendon to concrete $n_p = E_p / E_c$

σ_{pt} : tensile stress of prestressing tendon just after prestressing

σ'_{cpt} : compressive stress of concrete at the location of prestressing tendon by prestressing force just after prestressing

σ'_{cdp} : compressive stress of concrete at the location of prestressing tendon by permanent load

2) PRC structure

For PRC structure, the effect of restraint due to reinforcing bars should, in principle, be considered. If a limit state in which crack does not occur at tension edge of the concrete under permanent loads is set, in general the decrease in the tensile stress in the prestressing steel and the stress variation in the tension bar and the compression bar may be calculated from Eq. (C15.4.8) by using the creep coefficient and shrinkage strain of plain concrete. When the effect of shrinkage before prestressing cannot be neglected, variation of stress in tendon due to creep until prestressing may be calculated using Eq. (C15.4.9). In such cases, variation of stress in the tendon may be calculated by adding the value calculated using Eq. (C15.4.8) and Eq. (C15.4.9).

Further, in the case of post-tensioned prestressing, the term relating to the prestressing tendon in Eq. (C15.4.9) may be neglected.

$$\begin{Bmatrix} \Delta\sigma_{scs,k} \\ \Delta\sigma_{s'cs,k} \end{Bmatrix} = n_{s,k} \begin{bmatrix} 1+\alpha_{ssk}+\alpha_{psk}n_{ps}\beta_{13} & \alpha_{s'sk}+\alpha_{psk}n_{ps}\beta_{23} \\ \alpha_{ss'k}+\alpha_{ps'k}n_{ps}\beta_{13} & 1+\alpha_{s's'k}+\alpha_{ps'k}n_{ps}\beta_{23} \end{bmatrix}^{-1} \begin{Bmatrix} \varphi_k\left(\sigma_{cps,k}+\sigma_{cds,k}\right)+E_{c,k}\varepsilon'_{cs,k} \\ \varphi_k\left(\sigma_{cps',k}+\sigma_{cds',k}\right)+E_{c,k}\varepsilon'_{cs,k} \end{Bmatrix}$$

$$\Delta\sigma_{pcs,k} = n_{ps}\left(\beta_{13}\Delta\sigma_{scs,k}+\beta_{23}\Delta\sigma_{s'cs,k}\right) \tag{C15.4.8}$$

$$\begin{Bmatrix} \left\{\Delta\sigma_{scs,j}\right\}_{cs} \\ \left\{\Delta\sigma_{s'cs,j}\right\}_{cs} \end{Bmatrix} = n_{s,j} \begin{bmatrix} 1+\alpha_{ssj}+\alpha_{psj}n_{ps}\beta_{13} & \alpha_{s'sj}+\alpha_{psj}n_{ps}\beta_{23} \\ \alpha_{ss'j}+\alpha_{ps'j}n_{ps}\beta_{13} & 1+\alpha_{s's'j}+\alpha_{ps'j}n_{ps}\beta_{23} \end{bmatrix}^{-1} \begin{Bmatrix} \varphi_j\sigma_{css,j} \\ \varphi_j\sigma_{css',j} \end{Bmatrix}$$

$$\left\{\Delta\sigma_{pcs,k}\right\}_{cs} = n_{ps}\left(\beta_{13}\left\{\Delta\sigma_{scs,k}\right\}_{cs}+\beta_{23}\left\{\Delta\sigma_{s'cs,k}\right\}_{cs}\right) \tag{C15.4.9}$$

where, $\beta_{13} = \left(d_p - d'\right)/\left(d-d'\right)$ $\quad\quad$ $\beta_{23} = \left(d-d_p\right)/\left(d-d'\right)$

$\alpha_{ssk} = n_{s,k}A_s\left(1/A_c + e_s^2/I_c\right)J_k$ $\quad\quad$ $\alpha_{ssj} = n_{s,j}A_s\left(1/A_c + e_s^2/I_c\right)J_j$

$\alpha_{s'sk} = n_{s,k}A_{s'}\left(1/A_c - e_{s'}e_s/I_c\right)J_k$ $\quad\quad$ $\alpha_{s'sj} = n_{s,j}A_{s'}\left(1/A_c - e_{s'}e_s/I_c\right)J_j$

$\alpha_{psk} = n_{s,k}A_p\left(1/A_c + e_pe_s/I_c\right)J_k$ $\quad\quad$ $\alpha_{psj} = n_{s,j}A_p\left(1/A_c + e_pe_s/I_c\right)J_j$

$\alpha_{ss'k} = n_{s,k}A_s\left(1/A_c - e_se_{s'}/I_c\right)J_k$ $\quad\quad$ $\alpha_{ss'j} = n_{s,j}A_s\left(1/A_c - e_se_{s'}/I_c\right)J_j$

$\alpha_{s's'k} = n_{s,k}A_{s'}\left(1/A_c + e_{s'}^2/I_c\right)J_k$ $\quad\quad$ $\alpha_{s's'j} = n_{s,j}A_{s'}\left(1/A_c + e_{s'}^2/I_c\right)J_j$

$\alpha_{ps'k} = n_{s,k}A_p\left(1/A_c - e_pe_{s'}/I_c\right)J_k$ $\quad\quad$ $\alpha_{ps'j} = n_{s,j}A_p\left(1/A_c - e_pe_{s'}/I_c\right)J_j$

and, $J_k = 1+\varphi_k/2$, $J_j = 1+0.8\varphi_j$, age *j* may be the intermediate age of age *k*.

where, $\Delta\sigma_{pcs,k}$: loss of tensile stress of prestressing tendon by creep and shrinkage of concrete after prestressing

$\Delta\sigma_{scs,k}$: stress loss of tension bar by creep and shrinkage of concrete after prestressing

$\Delta\sigma_{s'cs,k}$: stress loss of compression bar by creep and shrinkage of concrete after prestressing

$\left\{\Delta\sigma_{pcs,j}\right\}_{cs}$: stress variation of prestressing tendon by creep which occurs by shrinkage before prestressing

$\left\{\Delta\sigma_{scs,j}\right\}_{cs}$: stress variation of tension bar by creep which occurs by shrinkage before prestressing

$\left\{\Delta\sigma_{s'cs,j}\right\}_{cs}$: stress variation of compression bar by creep which occurs by shrinkage before prestressing

φ_j, φ_k : creep coefficient of concrete loaded at ages, t_j, and t_k

$\varepsilon'_{cs,k}$: shrinkage strain of concrete after prestressing at age t_k

$n_{s,j}, n_{s,k}$: Young's modulus ratio of tension bar and compression bar to concrete at ages t_j, and t_k

$$n_{s,j} = E_s / E_{c,j} \quad , \quad n_{s,k} = E_s / E_{c,k}$$

$E_{c,j}$, $E_{c,k}$: Young's modulus of concrete at ages t_j, and t_k. It may be considered that $E_{c,k} = E_c$

n_{ps} : Young's modulus ratio of prestressing tendon to reinforcing bar

$$n_{ps} = E_p / E_s$$

$\sigma_{cpp,k}$: compressive stress of concrete at age t_k at the location of prestressing tendon by prestressing force just after prestressing, and it may be calculated by following equation.

$$\sigma_{cpp,k} = \beta_{13}\sigma_{cps,k} + \beta_{23}\sigma_{cps',k}$$

$\sigma_{cps,k}$: compressive stress of concrete at age t_k at the location of compression bar by prestressing force just after prestressing

$\sigma_{cdp,k}$: compressive stress of concrete at age t_k at the location of prestressing tendon by permanent load, and it may be calculated by following equation.

$$\sigma_{cdp,k} = \beta_{13}\sigma_{cds,k} + \beta_{23}\sigma_{cds',k}$$

$\sigma_{cds,k}$: compressive stress of concrete at age t_k at the location of tension bar by permanent load

$\sigma_{cds',k}$: compressive stress of concrete at age t_k at the location of compression bar by permanent load

$\sigma_{csp,j}$: tensile stress of concrete at age t_j at the location of prestressing tendon by shrinkage until prestressing, and it may be calculated by following equation.

$$\sigma_{csp,j} = \beta_{13}\sigma_{css,j} + \beta_{23}\sigma_{css',j}$$

$\sigma_{css,j}$: tensile stress of concrete at age t_j at the location of tension bar by shrinkage until prestressing

$\sigma_{css',j}$: tensile stress of concrete at age t_j at the location of compression bar by shrinkage until prestressing

A_p : cross-sectional area of prestressing tendon

A_s : cross-sectional area of tension bar

A_s' : cross-sectional area of compression bar

e_p : distance between the centroid of member and the centroid of prestressing tendon

e_s : distance between the centroid of member and the centroid of tension bar

e_s' : distance between the centroid of member and the centroid of compression bar

d', d_p, d : distance from compression edge to compression bar, prestressing tendon, and tension bar

A_c : total cross-sectional area of concrete

I_c : total cross-sectional moment of inertia of concrete

15.4.3 Design material stresses caused by shear force and torsional moment

(1) Design diagonal tensile stress of concrete caused by shear force and torsional moment may be calculated, assuming a fully effective cross section, from Eq. (15.4.1):

$$\sigma_I = \frac{(\sigma_x + \sigma_y)}{2} + \frac{1}{2}\sqrt{(\sigma_x - \sigma_y)^2 + 4\tau^2} \quad \textbf{(15.4.1)}$$

where, σ_I : design diagonal tensile stress of concrete

σ_x : normal stress

σ_y : orthogonal stress to σ_x

τ : shear stress caused by shear force and torsional moment

(2) Design stress of the shear reinforcement and design stress of the torsion reinforcement of a PRC structure may be calculated in accordance with Section 7.4.3 (2) and (3), respectively.

[Commentary] (1) Diagonal tensile stress of concrete is calculated by using the normal stress and shear stress calculated by assuming a fully effective cross section of concrete. If prestressing tendons are inclined, design shear force V_d may be calculated as $V_{pd} + V_{rd} - V_{prd}$, where V_{prd} is the vertical component of the tensile force of the prestressing tendons, and $(V_{pd} + V_{rd})$ is the loading shear force. If the vertical component of the tensile force of the prestressing tendons is greater than the shear loading force after introducing the prestress, it should be take care of the direction of shear force might be reversed.

In the design of a concrete structure, it is recommended to practice to draw up a structural plan so that the influence of torsion can be ignored. If a structural system cannot be designed without taking torsional stiffness into consideration, shear stress by the torsional moment must be taken into account.

Shear stress by shear force and torsional moment may be calculated, according to the elasticity theory, by the methods described below.

1) Shear stress caused by shear force may be calculated from Eq. (C15.4.10):

$$\tau = \frac{V_d \cdot Q}{b_w \cdot I} \tag{C15.4.10}$$

where, τ : shear stress caused by shear force

V_d : design shear force (= $V_{pd} + V_{rd} - V_{prd}$)

Q : first moment of area with respect to the neutral axis of the cross section outside the shear stress calculation location

b_w : width of the main section of the member

I : second moment of area with respect to the neutral axis of the cross section

2) Shear stress caused by torsional moment may be calculated as described in Items (a) to (c) below.

(a) In the case of a rectangular cross section (see Fig. C15.4.2), shear stress may be calculated from Eq. (C15.4.11) and Eq. (C15.4.12):

$$\tau_{t1} = \frac{M_{td}}{k_1 \cdot b^2 \cdot h} \tag{C15.4.11}$$

$$\tau_{t2} = k_2 \cdot \tau_{t1} \tag{C15.4.12}$$

where, τ_{t1} : shear stress along longer sides

τ_{t2} : shear stress along shorter sides

M_{ad}: design torsional moment

B: length of shorter side

h: length of longer side

k_1, k_2 : coefficients shown in Table C15.4.1

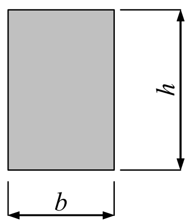


Fig. C15.4.2 Rectangular cross section

Table C15.4.1 Coefficients k_1 and k_2

h/b	1.0	1.2	1.5	2.0	2.5	3.0	4.0	5.0	7.0	10	20	∞
k_1	0.208	0.219	0.231	0.246	0.258	0.267	0.282	0.292	0.303	0.313	0.323	0.333
k_2	1.000	0.930	0.859	0.795	0.766	0.753	0.745	0.743	0.742	0.742	0.742	0.742

(b) In the case of a T-shaped cross section (see Fig. C15.4.3), shear stress may be calculated from Eq. (C15.4.13):

$$\tau_{ti} = \frac{M_{td}}{I_t} \cdot b_i \cdot \eta_i \tag{C15.4.13}$$

where, τ_{ti} : shear stress of each rectangular section

M_{td} : design torsional moment

I_t : torsional second moment of area

$I_t = \Sigma k_i \cdot a_i \cdot b_i^3$

a_i : length of a longer side of each rectangle

b_i : length of a shorter side of each rectangle

k_i : coefficient shown in Table C15.4.2

η_i : coefficient shown in Table C15.4.3

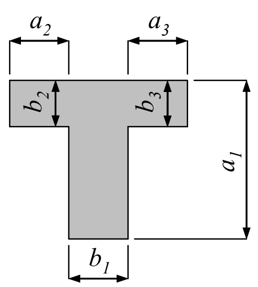


Fig. C15.4.3 T-shaped cross section

Table C15.4.2 Coefficient k_i

a_i/b_i	1.0	1.2	1.5	1.75	2.0	2.5	3.0	4.0	5.0	10	∞
k_i	0.140	0.166	0.196	0.214	0.229	0.249	0.263	0.281	0.292	0.313	0.333

Table C15.4.3 Coefficient η_i

a_i/b_i	1.0	1.2	1.5	1.75	2.0	2.5	3.0	4.0	5.0	10	∞
η_i	0.675	0.759	0.848	0.895	0.930	0.968	0.985	0.997	0.999	1.0	1.0

(c) In the case of a box-shaped cross section (see Fig. C15.4.4), shear stress may be calculated from Eq. (C15.4.14).

$$\tau_{ti} = \tau_{Si} + \tau_{Bi} \tag{C15.4.14}$$

$$\tau_{Si} = \frac{M_{td}}{I_{t1}} \cdot b_i \cdot \eta_i$$

$$\tau_{Bi} = \frac{M_{td}}{I_{t1}} \cdot \frac{2A}{C \cdot t_i}$$

where, τ_{ti} : shear stress of each rectangular section

M_{td} : design torsional moment

I_t : torsional second moment of area

$$I_t = I_{tS} + I_{tB}$$

$$I_{tS} = \Sigma k_i \cdot a_i \cdot b_i^3$$

$$I_{tB} = \frac{4A^2}{C}$$

$$A = b \cdot h$$

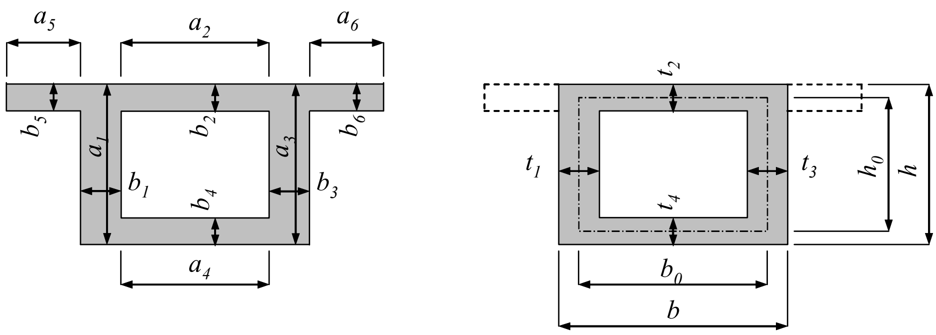
$$C = \frac{h_0}{t_1} + \frac{b_0}{t_2} + \frac{h_0}{t_3} + \frac{b_0}{t_4}$$

where, a_i : length of longer side of each rectangle shown in Fig. C15.4.4 (a)

b_i : length of shorter side of each rectangle shown in Fig. C15.4.4 (a)

k_i : coefficient shown in Table C15.4.2

η_i : coefficient shown in Table C15.4.3



(a) When calculating τ_{Si} and I_{tS} (b) When calculating τ_{Bi} and I_{tB}

Fig. C15.4.4 T-shaped cross section

(2) In general, only stirrups are used as shear reinforcement for prestressed concrete structures, and bent-up bars are not used. For this reason, design tensile stress of the shear reinforcement of a PRC structure to resist shear force may be calculated from Eq. (7.4.1) and Eq. (7.4.2) shown in Section 7.4.3.

Design tensile stress of the torsion reinforcement of a PRC structure to resist torsional moment may be calculated from Eq. (7.4.3). If the tendons for a member subjected to torsional moment are inclined, torsional moment M_{tped} by prestressing force may occur even in a statically determinate structure. Design torsional moment M_{tpd} by dead loads may be calculated by subtracting the torsional moment M_{tped} from the torsional loading moment. When calculating design torsion capacity M_{tcd} in the no-torsion-reinforcement case, the influence of prestressing force on the torsion capacity must be allowed for β_{nt} in Eq. (9.2.26) shown in Section 9.2.3.2.

15.4.4 Design flexural crack width

(1) Design flexural crack width w_d in a PRC structure may be calculated in accordance with Section 7.4.4.

(2) As a general rule, the reinforcing bars to be considered in connection with the verification related to flexural cracking shall be tension reinforcement bars closest to the concrete surface, and reinforcement stress shall be calculated in accordance with Section 15.4.2.

[Commentary] Similarly to reinforced concrete structures, PRC structures described in this chapter are basically designed to control crack spacing by using the crack distribution effect of deformed steel bars and control the increase in steel stress by use of prestressing. If internal tendons are used a structure of this type, deformed bars for crack width control are usually placed at locations closer to the concrete surface than the internal tendons. If external tendons are used, rust preventions for prestressing steel are considered. As a general rule, therefore, in the verification of flexural crack width, only the deformed bars closest to the concrete surface need to be taken into consideration. In Eq. (7.4.4), the coefficient k_1, which allows for the influence of the surface configuration of steel on crack width, may be assumed to be 1.0 to calculate flexural crack width corresponding to the increase of the stress of bars.

In Eq. (7.4.4), ε'_{csd}, which considers the creep and shrinkage and so on, needs to be determined taking into consideration the cross-sectional shape of the member, environmental conditions, the magnitude of stress, etc. Since the value of ε'_{csd} varies among verification items, it needs to be calculated by using the values indicated in each chapter of this Specification. If the value for the location of the center of gravity of the tension reinforcement is used, 1.0 must be used for the coefficient k_3, which considers the influence of multilayer reinforcement, in Eq. (7.4.4) regardless of the number of reinforcement layers.

However, in the case of a special type of structure, a structure in which the influence of longitudinal restraint is great, a structure constructed by a special construction method is to be as a PRC structure, it is recommended that not only the influence of the shrinkage and creep of concrete on crack width but also other influences such as the age at which cracking occurs be taken into consideration to determine the value of ε'_{csd} in accordance with Section 5.2.8., and flexural crack width should be calculated by ε'_{csd}.

15.5 Verification of Durability

(1) Verification of durability should be made in accordance with Chapter 8, taking the influence of prestressing into consideration.

(2) As a general rule, verification of steel corrosion should be applied to reinforcing steel and prestressing steel. Prestressing steel should be protected not to be damaged with corrosion for the design service life.

(3) When a PRC structure is constructed in chloride environment, as a general rule, plastic sheaths should be used to protect the prestressing steel from corrosion. In this case, the verification of steel corrosion may be ignored.

(4) When external tendons and unbonded prestressing tendons are used, verification of steel corrosion should be applied to the entire tendon system including anchorages and deviators. However, the verification may be ignored if the durability performance for steel corrosion has already been ensured by experimental tests.

[Commentary] (1) Similarly to the verification for a reinforced concrete structure, verification of steel corrosion for a PRC structure considers flexural cracking, shear cracking and torsion cracking.

In a PRC structure, the influence of shear cracking and torsion cracking on steel corrosion may be ignored if the requirements described in Chapter 8 are satisfied. To be more specific, the influence of shear cracking may be ignored if the design shear force V_d ($= V_{pd} + V_{rd} - V_{prd}$) is less than 79% of the shear capacity V_{cd} calculated from Eq. (9.2.4). Similarly, the influence of torsion cracking may be ignored if the design torsional moment M_{td} is less than 70% of the design torsion capacity of a bar member without torsion reinforcement calculated from Eq. (9.2.26). If the influence of shear cracking and torsion cracking on steel corrosion is taken into consideration, an appropriate evaluation method must be used. However, more examinations may not be needed if it is confirmed that the design tensile stresses of the shear reinforcement and torsion reinforcement subjected to dead loads are less than the values shown in Table C8.3.1, respectively.

(2) If prestressing steel is damaged with corrosion, stress concentration occurred by decreasing of cross-sectional area of steel and hydrogen embrittlement may eventually cause the prestressing steel to fracture, thereby degrading the structural performance considerably. Even in a normal environment, it is recommended that resin-coated prestressing tendons, plastic sheaths, etc., capable of effectively protecting prestressing tendons from corrosion such as water, oxygen and chloride ions be used.

As a general rule, the verification of steel corrosion must be made for the reinforcing bars and prestressing tendons closest to the concrete surface. In the verification of durability for internal tendons and unbonded prestressing tendons, design values of concrete cover of sheaths may be used as design values c_d of concrete cover.

If anchoring elements or couplers for prestressing tendons are located closer to the concrete surface than the prestressing tendons, verification needs to be made concerning their corrosion. It is recommended that anchorage elements are appropriately protected in accordance with Section 15.9.6 in order to prevent corrosion.

(3) For the reason described in the commentary in Section 8.3.2 (2), limit value of crack width has not been specified for prestressing tendons in chloride environment (see Table 8.3.2). When a structure is used in chloride environment, therefore, it is desirable that the concrete cracking is not occurred in a PC structure. If a PRC structure is used, as a general rule, plastic sheaths capable of protecting prestressing steel from corrosion are used. If such sheaths are used, verification of corrosion for prestressing steel may be ignored. It is also recommended, in order to enhance the corrosion resistance of prestressing tendons, that plastic sheaths be used when a PC structure is constructed in chloride environment. In such cases, only the reinforcing bars closest to the concrete surface need to be taken into consideration in the verification.

If plastic sheaths are used, anchorage elements should be protected from corrosion by the grout capping, that are comparable to sheaths in terms of shielding performance in order to enhance the corrosion resistance of prestressing tendons. If steel sheaths and plastic sheaths of the same inside diameter are used, usually the outside diameter of the plastic sheaths is greater than that of the steel sheaths. Usually, members made with such sheaths are larger. The influence of such size increases needs to be evaluated.

(4) This chapter assumes the use of permanently rust- and corrosion-proof tendons as external tendons. It is necessary to conduct verification of steel corrosion for the entire tendon system including anchorage elements and deviators and use tendons with corrosion resistance for the design service life.

A typical rust-proofing structure for external tendons consists of three layers, namely, a protective sheath, filler material and prestressing tendons. Examples of external tendon rust-proofing structures are shown in Table C15.5.1. An appropriate combination of elements should be selected in view of the environmental conditions, etc. Case 7, in which resin-coated prestressing tendons are used without protective sheathing, should be used only in a low-humidity and mild environment.

Table C15.5.1 Examples of rust-proof structures for external tendons

<table>
<tr><th>Case</th><th>Protective sheathing</th><th>Filler material</th><th>Type of prestressing steel</th></tr>
<tr><td>1</td><td>Polyethylene sheath</td><td rowspan="2">Cement grout</td><td rowspan="2">Ordinary prestressing steel</td></tr>
<tr><td>2</td><td>Transparent sheath</td></tr>
<tr><td>3</td><td rowspan="4">Polyethylene sheath</td><td>Grease</td><td rowspan="2">Galvanized prestressing steel</td></tr>
<tr><td>4</td><td>None</td></tr>
<tr><td>5</td><td>Cement grout</td><td rowspan="3">Resin-coated prestressing steel*</td></tr>
<tr><td>6</td><td>None</td></tr>
<tr><td>7</td><td>None</td><td>None</td></tr>
</table>

* Type of resin-coated steel

1) Epoxy resin coated steel

2) Polyethylene resin coated steel

3) Epoxy resin and polyethylene resin coated prestressing steel

Unbonded prestressing tendons can be largely classified, according to the type of rust-proof structure, into two types: coated unbonded prestressing tendons and sheathed unbonded prestressing tendons. A tendon of the former type consists of a prestressing steel tendon coated with asphalt, polymer or other rust-preventive material and covered with protective taping. A tendon of the latter type consists of a prestressing steel tendon encased in polyethylene or other sheathing and grease or other material filling the space between the steel and the sheath. Most of these tendons are manufactured at factories. Recently, unbonded prestressing tendons using epoxy resin coated steel has been developed in order to enhance the rust-proof performance. Another type of recently developed unbonded prestressing tendon is a pregrouted prestressing tendon, which uses slower setting resin. This cold-setting resin remains unbonded until prestress is introduced, and bonding is achieved as the resin hardens over time.

15.6 Verification of Safety

15.6.1 General

(1) Verification of safety shall be done in accordance with Chapter 9.

(2) Verification of cross-sectional failure shall be made in accordance with Section 9.2, taking into consideration the influence of prestress. With respect to the design flexural strength M_{ud} of a member subjected to bending moment or to bending moment and axial force, the flexural strength M_u may be calculated by the method described in this section.

(3) Verification of fatigue failure shall be made in accordance with Section 9.3, and response stresses in materials shall be calculated in accordance with Section 15.4.2, taking the influence of prestress into account. Verification for steel in PC structures may be ignored.

[Commentary] (2) In the verification of safety for flexural moment, PC structure is similar to RC structure's behavior if the steel yielding occurs before crush of concrete cover. The verification for PC structure may be done as reinforcing bars.

Plane conservation, however, can be applied neither to unbonded prestressing tendons nor nearly-unbonded tendons such as external tendons. Consequently, the amount of increase in the tensile stress in a tendon at the failure is less than in an internal tendon. In the external tendon, the effective height of an external tendon in a non-deviated free portion of the tendon becomes relatively small as the deformation of the member increases. Flexural strength in such cases is usually less than in cases that effective height does not change. A member with internal tendons and a member with unbonded prestressing tendons or external tendons require different evaluation methods of flexural strength. Flexural strength of those members, therefore, should be calculated by the each methods described in this section.

(3) In the case of that a concrete crack is not allowed in the verification of usability of PC structures, the steel of prestressing stress is small and the verification of steel stress for fatigue failure may be ignored.

Stress variation of unbonded prestressing tendons and external tendons is less than that of internal prestressing tendons. However, the stress may effect on the entire of prestressing tendons through the anchor of tendons. The damage of local portion in tendons depends on the fatigue strength. Therefore, it is necessary to care the fatigue failure of the unbonded tendons and external tendon. Especially, it is recommended to use the structure of anchor and deviator, which are ensured to have enough fatigue strength.

Also, it is necessary to care the resonant of external tendons with the member, vehicle vibrations, and mechanical equipments. Fatigue failure may occur in these cases. It is recommended to place the vibration control equipment and to arrange the external tendons at the small spacing.

Fretting fatigue may occur at the deviator when the external tendons are used. The external tendon has come in contact mutually, and the fatigue damage accumulated in the tendons by friction of each other. It is recommended to care the corner breaking of prestressing tendons, and to fill the grout cement in the sheath to restrain the displacement, and to protect the prestressing tendons with resin coating, and to prevent the tendons from coming in contact mutually with the spacer, etc.

15.6.2 Design flexural strength of PC members with internal tendons

The design flexural strength of a PC member with internal tendons subjected to flexural moment and axial force may be calculated on the basis of the following assumptions:

(i) Strain is proportional to the distance from the neutral axis of the cross section.

(ii) Tensile stress of concrete may be ignored.

(iii) As a general rule, the stress-strain curve of concrete is shown in Fig. 5.2.1 in Section 5.2.3.

(iv) The stress-strain curve for steel must be as shown in Fig. 5.3.1 in Section 5.3.3.

[Commentary] The flexural strength M_u of a member with bonded internal tendons may be calculated in accordance with Fig. C15.6.1 by the methods described in Items 1) to 4) below if the neutral axis is located in the cross section. In cases where there is lateral reinforcement, its flexural strength should be evaluated by other method.

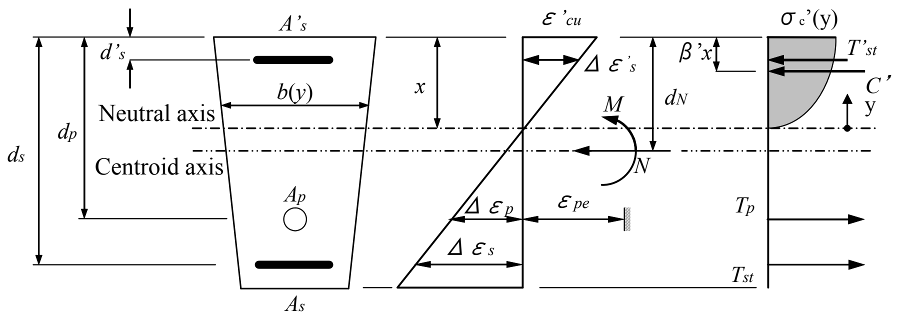


Fig. C15.6.1 Calculating the flexural strength M_u of a PC member with an internal tendon

1) Assume the location of the neutral axis and determine the strain distribution in the cross section on the basis of the assumption in Item (i) above by regarding the concrete strain at the compressive concrete surface as the ultimate compressive strain ε'_{cu}.

2) According to the strain distribution in the cross section, the compressive force resultant C' of the concrete is calculated from Eq. (15.6.1) on the basis of the assumption in Item (iii). Similarly, $\Delta\varepsilon'_s$, $\Delta\varepsilon_p + \varepsilon_p$ (ε_p : strain caused by prestressing of internal tendon) and $\Delta\varepsilon_s$ is calculated on the basis of the assumption in Item (iv). The compressive force resultant T'_{st} of the reinforcement, the tensile force resultant T_p of the internal tendon, the tensile force resultant T_{st} of the reinforcement are calculated from Eqs. (C15.6.2), (C15.6.3), and (C15.6.4), respectively.

$$C' = \int_0^x \sigma'_c(y) \cdot b(y) \cdot dy \tag{C15.6.1}$$

$$T'_{st} = A'_s \cdot \sigma'_s \tag{C15.6.2}$$

$$T_p = A_p \cdot \sigma_p \tag{C15.6.3}$$

$$T_{st} = A_s \cdot \sigma_s \tag{C15.6.4}$$

where, A'_s, A_p, A_s : cross-sectional areas of compression reinforcement, internal tendon and tension reinforcement

3) From the equilibrium of the forces in the cross section, Eq. (C15.6.5) can be given. This equation is a quadratic equation in which the neutral axis location x is an unknown. By solving this equation, the location of the neutral axis is given.

$$N'_d = C' + T'_{st} - T_p - T_{st} \tag{C15.6.5}$$

where, N'_d : design axial compressive force

4) If the neutral axis location x is determined, C', T'_{st} and T_{st} can be calculated. The location of the compressive stress resultant C' of the concrete, $\beta' \cdot x$, can be calculated from Eq. (C15.6.6), and the flexural strength of the cross section is calculated from Eq. (C15.6.7):

$$\beta' \cdot x = x - \frac{\int_0^x \sigma'_c(y) \cdot b(y) \cdot y \cdot dy}{C'} \tag{C15.6.6}$$

$$M_u = C'(d_N - \beta' \cdot x) + T'_{st}(d_N - d'_s) + T_p(d_p - d_N) + T_{st}(d_s - d_N) \tag{C15.6.7}$$

In structural analysis including Eq. (C15.6.7), flexural moment is usually determined with respect to the centroid of the cross section.

Except in the case where the strain of the cross section is completely compressive, the compressive stress resultant C' of the concrete may be calculated in accordance with Section 9.2.1.1 (3) by assuming an equivalent stress block. If $f'_{ck} \leq 50\text{N/mm}^2$ with a rectangular cross section [$b(y) = b$: constant], then $\beta' \cdot x = \beta \cdot x/2 = 0.4x$ and $C' = 0.68 f'_{ck} \cdot b \cdot x$.

15.6.3 Design flexural strength of PC members with unbonded prestressing tendons or external tendons

The design flexural strength of a PC member with unbonded prestressing tendons or external tendons subjected to bending moment and axial force may be calculated on the basis of the assumptions mentioned in Section 15.6.2, taking into consideration the influence of Items (i) and (ii) below:

(i) The assumption that strain of unbonded prestressing tendon or an external tendon is proportional to the distance from the neutral axis of the cross section cannot be applied.

(ii) In the case of a PC member with external tendons, the effective height of the external tendons in the section changes relatively.

[Commentary] Plane conservation cannot be applied to unbonded prestressing tendons and other unbonded tendons such as external tendons. The amount of increase, therefore, in the tensile stress in such a tendon at the failure state is less than that in a bonded internal tendon. In the case of an external tendon, the effective height of the external tendon in the free (i.e., not supported by deviators) portion of the tendon decreases relatively as the deformation of the member increases (see Fig. C15.6.2). Flexural strength tends to be lower than in the case where an unbonded prestressing tendon, whose effective height does not change. The flexural strength of prestressed concrete members with unbonded prestressing tendons or external tendons should be calculated by a method that appropriately allows for these factors into consideration.

Methods for calculating the flexural strength of members with unbonded prestressing tendons or external tendons can be classified into the three types described below. It is preferred to select a method taking into consideration such factors as the shape of the structure, support conditions, the state of loads and the nonlinear behavior of the member at the failure state.

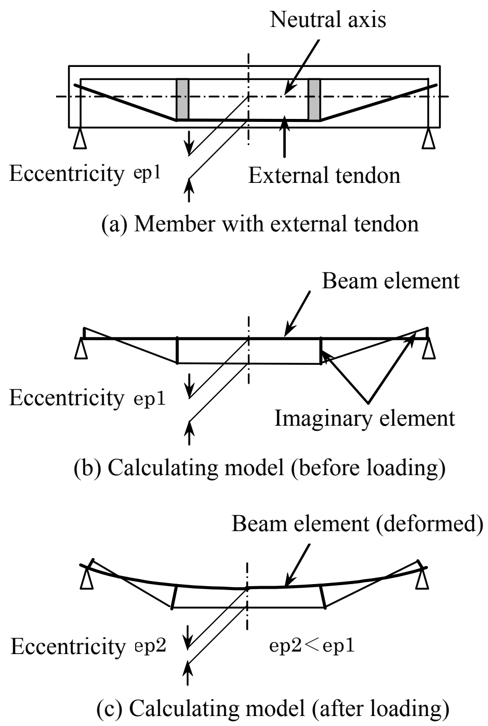


Fig. C15.6.2 Analysis model and deformation of a member with an external tendon

(i) Method of nonlinear analysis considering displacement

In this method, the flexural strength of a member with an unbonded prestressing tendon or an external tendon is calculated by applying nonlinear analysis that enables the calculation of the flexural strength of a structural system. By taking into consideration the influence of material nonlinearity and the influence of geometrical nonlinearity, the flexural strength of the member can be calculated through exact evaluation of the deformation of the concrete member with the increase in loads and the resultant increases or decreases in prestressing force due to changes in the tension of the unbonded tendon, decreases in the effective height of the external tendon due to changes in the location of the free portion of the tendon, etc.

In recent years, research on the flexural properties of members with unbonded tendons has made considerable progress, and the flexural strength is evaluated with high accuracy by using nonlinear analysis. In connection with members with unbonded prestressing tendons, the method of calculating flexural strength by verifying the compatibility of deformation (the amount of elongation of a tendon is equal to the total amount of elongation of the concrete at the same location as the tendon) has been proposed. This method is generally called "rigorous method," and its application to members with external tendons is currently examined.

According to research results concerning the application of nonlinear finite element analysis, external tendons should be modeled directly as chord members (one-dimensional linear elements) as shown in Fig. C15.6.2. In the case of an unbonded prestressing tendon, multiple deviators (imaginary members) should be used so that the influence of changes of the effective height can be ignored. Concrete members need to be modeled appropriately according to their cross sectional properties, etc. If the deformation of a member occurs only in a two-dimensional analytic space, the modeling method using planar elements can be used. If flexural deformation is so dominant that shear deformation can be ignored, the method of modeling the concrete member as a linear member with axial stiffness and flexural stiffness and using fiber models to which material-specific stress-strain relationship is applied as linear elements can be used (see Fig. C15.6.3). By so doing, usually, analysis time can be reduced.

A method based on nonlinear analysis can be applied to any type of prestressed concrete structure, regardless of the type of structure or tendon. If flexural strength needs to be calculated accurately or if the behavior of a member at the failure state is unknown as when a new type of structure with unbonded prestressing tendons or external tendons is constructed, it is desirable that flexural strength be calculated by using nonlinear analysis.

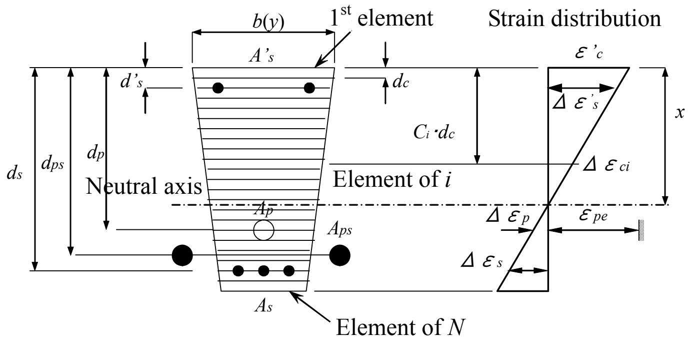


Fig. C15.6.3 Example of fiber modeling of a concrete member

(ii) Method allowing tension increase of tendon caused by displacement

In this method, unbonded prestressing tendons and external tendons are regarded as tension-resisting members, the amount of increase of the tensile stress, $\Delta\sigma_{ps}$, at the failure state is defined, and the flexural strength of the member is calculated in accordance with the method described in Section 15.6.2. Experiments and nonlinear analyses are used in combination to consider such factors as the type of structure, structural conditions and loading conditions, and if the amount of increase of the tensile stress, $\Delta\sigma_{ps}$, at a failure state can be appropriately defined in advance, the flexural strength of the member may be calculated by this method. The differences between the flexural strength calculation described here and the method described in Section 15.6.2 are explained below (see Fig. C15.6.4).

In this method, the location of the neutral axis is assumed first, and the compressive stress resultant C' of the concrete, the compressive stress resultant T'_{st} of reinforcement, the tensile stress resultant T_p of internal tendons and the tensile stress resultant T_{st} of reinforcement are calculated from Eqs. (C15.6.1) to (C15.6.4), respectively. In addition, the tensile stress resultant T_{ps} of unbonded tendons is calculated from Eq. (C15.6.8). Except in the case where the strain of the cross section is completely compressive, the compressive stress resultant C' of concrete may be calculated by assuming an equivalent stress block.

$$T_{ps} = A_{ps} \cdot \sigma_{ps} = A_{ps}\left(\sigma_{pse} + \Delta\sigma_{ps}\right) \qquad \text{(C15.6.8)}$$

where, A_{ps} : cross-sectional area of unbonded tendon

σ_{ps} : tensile stress of unbonded tendon at failure state

σ_{pse} : tensile stress caused by effective prestressing of unbonded tendon

σ_{ps} : amount of increase of tensile stress of unbonded tendon at failure state

Eq. (C15.6.5) and Eq. (C15.6.9) can be derived from the equilibrium of cross section. The location x of the neutral axis can be determined by solving that equation.

$$N'_d = C' + T'_{st} - T_p - T_{ps} - T_{st} \qquad \text{(C15.6.9)}$$

where, N'_d : design axial compressive force

If the location x of the neutral axis is determined, T_{ps} can be calculated in addition to C', T'_{st}, T_p and T_{st}. Since the location of action $\beta' \cdot x$ of the compressive stress resultant C' of the concrete can be calculated from Eq. (C15.6.6), the flexural strength of the cross section can be calculated from the equilibrium of cross section by using Eq. (C15.6.10).

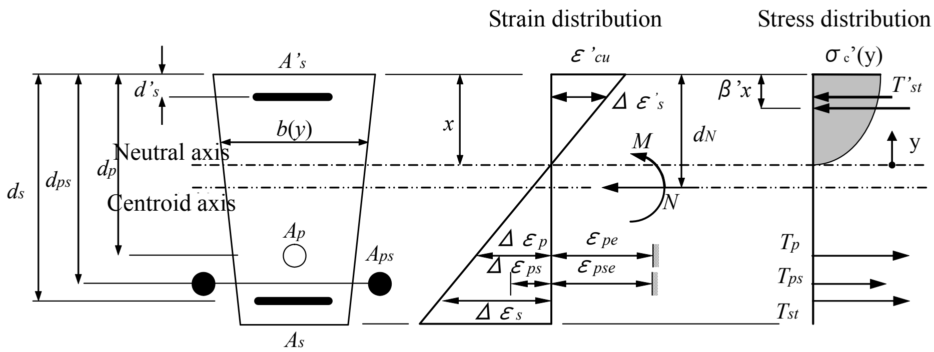


Fig. C15.6.4 Method of calculating the flexural strength M_u of member with unbonded prestressing tendon and external tendon

$$M_u = C'\left(d_N - \beta' \cdot x\right) + T'_{st}\left(d_N - d'_s\right) + T_p\left(d_p - d_N\right) + T_{ps}\left(d_{ps} - d_N\right) + T_{st}\left(d_s - d_N\right) \qquad \text{(C15.6.10)}$$

Equation (C15.6.8) was originally proposed as an equation applicable to unbonded prestressing tendons. When applying the equation to an external tendon, it is necessary to verify that the effective height of the external tendon does not change relatively at the failure state.

The amount of increase of the tensile stress of an external tendon, $\Delta\sigma_{ps}$, at the failure state is depended on many structural conditions such as the relationship between span length and the effective height of a tendon, the relationship between span length and the distance between tendon anchorages, the relationship between span length and deviator spacing, the cross-sectional area ratio between bonded internal tendons and external tendons, and the amount of tension reinforcement. It has also been found that the amount of increase of tensile stress in the case where a uniformly distributed load differs from that in the case where a concentrated load even under identical structural conditions. It is thought that load conditions (load types) also affect the amount of increase of tensile stress. Stresses in external tendons begin to increase significantly after bonded steel in the concrete has yielded. For this reason, if many internal tendons are used in a PC member with both internal and external tendons or if many tension reinforcing bars are used in a PRC member, it is important to determine the amount of increase, $\Delta\sigma_{ps}$, of tensile stress in view of the possibility of the occurrence of failure before the tensile stress of external tendon increases.

In general, an economical design is made possible by identifying the type of structure, structural conditions, loading conditions, the amount of increase, $\Delta\sigma_{ps}$, of the tensile stress of unbonded tendons.

(iii) Method ignoring tension increase of tendon caused by displacement

In this method, unbonded prestressing tendons and external tendons are regarded as tension-resisting members as in the method described in Item (ii), but the amount of increase of the tensile stress of unbonded tendons at the failure state is not taken into account. To be more specific, by assuming $\Delta\sigma_{ps} = 0$ in Eq. (C15.6.8), the flexural strength of a member with unbonded tendons can be calculated by following the steps similar to the method described in Item (ii). This method is usually the most conservative approach to design.

Another simple method is to reduce by 30% the flexural strength calculated by a method similar to the method used in cases where bonded tendons are used. This method has been used for long years in cases where unbonded prestressing tendons are used, and it may be applied to members with external tendons. When external tendons are used, it is also possible to regard the prestressing force introduced by external tendons as an external force. In this method, the internal force caused by the effective prestressing of external tendons is added, as an external force, to the flexural strength calculated by regarding internal tendons and reinforcing bars placed in the member as tension-resisting members. In general, these methods may be conservative. When using these methods, however, it is necessary to take other factors such as economy into consideration.

15.7 Verification Related to Serviceability

15.7.1 General

(1) Verification related to serviceability shall be done in accordance with Chapter 10 by classifying structures either as PC structures or PRC structures.

(2) Appropriate limits shall be set for stress in materials in accordance with the provisions of this section.

[Commentary] Verification related to the appearance, vibration, displacement and deformation, watertightness, etc., must be done in accordance with Chapter 10. If, however, the requirements described in Chapter 8 are met, verification related to shear cracking and torsion cracking associated with appearance may be omitted.

When considering limits of stress in ordinary concrete structures, only concrete stress due to bending and axial force is calculated for concrete. In serviceability-related verification for PC structures, however, it is necessary to verify that neither shear cracking nor torsion cracking occurs. This section, therefore, indicates limits for diagonal tensile stress in concrete due to shear force and torsional moment.

15.7.2 Maximum permissible value of stress

(1) The maximum permissible value of compressive stress in concrete due to flexure and axial loads shall be as given in Section 7.3. Also, the maximum permissible value for tensile stresses in prestressing tendon shall be $0.7 f_{puk}$, where f_{puk} is the characteristic value of tensile strength of the prestressing tendon.

(2) Extreme fiber tensile stress in the concrete of a PC structure shall be limited as specified in Items (i) and (ii) below:

(i) The value of flexural strength calculated using Eq. (3.2.4) shall be used as the maximum permissible value of tensile stress for concrete. However, it shall be ensured that tensile stresses do not occur at the location of joint in precast concrete members.

(ii) When stress in concrete is tensile, tension reinforcement in excess of the value obtained from Eq. (15.7.1) shall be provided. Further, the tension reinforcement shall be in the form of deformed bars.

$$A_s = T_c / \sigma_{sl} \qquad \textbf{(15.7.1)}$$

where, A_s **: cross-sectional area of tension reinforcement**

T_c **: total tensile force in concrete**

σ_{sl} **: maximum permissible value for tensile stresses in tension reinforcement. A value of 200 N/mm^2 may be used for deformed bars. However, bonded prestressing tendons located in tensile region of concrete may be considered as tensile reinforcement. In such cases, a value of 200 N/mm^2 may be used for pre-tensioning tendons and 100 N/mm^2 for post- tensioning tendons**

(3) Diagonal tensile stress in the concrete of a PC structure under a combination of permanent loads and variable loads shall be limited as specified in Items (i) to (iv) below:

(i) 75% of the value of the design tensile strength when either shear or torsion is considered.

(ii) 95% of the value of the design tensile strength when both shear and torsion are considered.

(iii) It may generally be sufficient to examine the diagonal tensile stress at two locations – one at centroid of the cross section and another at a location where the normal stress equals zero.

(iv) If the member is directly supported, examination for the diagonal tensile stress is not considered necessary for sections within a distance equal to half the depth of the member from the front face of support. Sections in that region shall, however, carry the same amount of shear reinforcement as that used at a distance of half the depth of the member.

[Commentary] (1) The limitation on the tensile stress is not necessary when examinations for cracking in concrete and the fatigue in prestressing steel are carried out. However, when the tensile stress exceeds the elastic limit, the situation becomes complicated because the assumptions made in structural analysis and calculation of stresses do not hold, and, the prestressing force cannot be dealt with as an external force. Therefore, it shall be ensured that the tensile stress does not exceed the elastic limit. Further, examination for the long-term relaxation of the prestressing steel shall be carried out under the application of an initial stress equal to 70% of the characteristic value of its tensile strength. In fact, the maximum permissible value has been determined in a manner that both conditions can be satisfied.

(2) (i) Tensile stress in concrete should be limited to the flexural cracking strength, taking into consideration of the effect of member size, in accordance with Section 5.2.1 (5). However, in the case of joints in precast concrete members as the longitudinal reinforcing bars are not continuous, no tensile stress shall be allowed even in the serviceability limit state. Table C15.7.1 shows the value of maximum permissible values of tensile stress in concrete in accordance with Section 5.2.1 (5).

Table C15.7.1 Limitation of tensile stress of concrete for PC structure (N/mm^2)

Loading condition	Cross-sectional height (m)	Specified design strength f'_{ck} (N/mm^2)					
		30	40	50	60	70	80
When variable load acts	0.25	2.3	2.7	3.0	3.4	3.7	4.0
	0.5	1.7	2.0	2.3	2.6	2.9	3.1
	1.0	1.3	1.6	1.8	2.1	2.3	2.5
	2.0	1.1	1.3	1.5	1.7	1.9	2.0
	more than 3.0	1.0	1.2	1.3	1.5	1.7	1.8

(ii) Tension reinforcement shall be provided in cases tensile stress exists in concrete, so that the difference between the stress calculated as an uncracked member and that calculated as a cracked member could be relatively small.

Alternately, the required amount of tension reinforcement may be calculated neglecting the tensile stress in concrete. However, the method specified in this section may be used because it is easy to use so long as the tensile stress in concrete does not exceed the design tensile strength, and given that it gives a conservative estimate.

The maximum permissible value for tensile stress for prestressing steel should be the stress difference between prestress induced at prestressing and the yielding stress divided by a safety factor, however there is no substantial study on how much the safety factor is. In this clause the constant values are given for the maximum permissible value for simplicity of design. The maximum permissible value is different for the pre-tensioning and post-tensioning tendon. The reason is that the bonding ability of tendon in post-tensioning is less than those of pre-tensioning because tendons in post-tensioning are usually thicker than those used in pre-tensioning.

For PC structure using unbonded tendon or PRC structure, when loads in excess design loads act and cracking occurs, problems such as a poor distribution of cracks, large crack widths, and occurrence of secondary forces on account of re-distribution of bending moment, may cause concern. Since the difficulties encountered could be greater than those in ordinary reinforced structure, it is advisable to adopt more appropriate load factor.

For concrete members using an external prestressing system, minimum amount of reinforcing bars should be provided as in the case of reinforced concrete members subjected to bending moment. In calculating the amount of required tension reinforcement, an increase in variable load may not be considered. Further, external tendons shall not be considered as part of the tensile reinforcement.

(3) (i), (ii) In order to control diagonal cracking in members designed as PC members, the maximum permissible values for the principal tensile stress are determined in a manner that they are consistent with the conventional allowable principal tensile stresses.

(iii) Diagonal tensile stress in concrete varies with locations in the member cross section. Stress calculation, therefore, needs to be made only the centroid of the member cross section and at the location where normal stress is zero. In cases when the web width on the tension side is smaller than that at the centroid of the section, the examination should be carried out at a location where the maximum diagonal tensile stress occurs.

(iv) In regions adjacent to the support of a beam, the support reaction causes a large compressive stress in the web in the vertical direction. This compressive stress acts advantageously for the considerations of diagonal tensile stress. Thus, the examination for the diagonal tensile stress may be omitted for sections within a distance equal to half the depth of the member.

In the case of pre-tensioned members, variation of the prestressing force should be considered in the design section for shear within the development length. It may be assumed that prestressing force varies parabolically within the development length, and is zero at the end of the member. The development length may be assumed as indicated below, provided that the concrete compressive strength at time of prestressing is not less than 35N/mm^2.

- For wire tendons having plain surface or indented wire tendons: 100 times the diameter of the wire tendon

- For deformed wire tendons having protruded surface and wire strand: 65 times the diameter of the wire tendon or the wire strand

15.8 Verification During Construction

During construction, it shall be ensured that given in (1) to (4) below:

(1) In view of safety from the breaking or yielding of tendons during construction, it shall be verified that the tensile stress in tendons during and immediately after tensioning be not greater than the values specified in Items (i) and (ii) below, where f_{puk} and f_{pyk} are characteristics values of tensile strength and yield strength of tendons.

(i) Stress in the prestressing tendon during prestressing does not exceed $0.8f_{puk}$ or $0.9f_{pyk}$, whichever smaller.

(ii) Stress in the prestressing tendon immediately after prestressing does not exceed $0.7f_{puk}$ or $0.85f_{pyk}$, whichever smaller.

(2) As a general rule, the occurrence of cracking in concrete shall not be permitted, and Items (i) and (ii) below shall be verified.

(i) Tensile stress in concrete does not exceed the value of flexural strength calculated using Section 5.2.1(5) taking a value of 1.0 for γ_c and using the concrete compressive strength at the time of examination instead of the characteristic compressive strength of the concrete.

Tension reinforcement is provided in concrete in regions where tensile stresses exist, and the minimum amount of this reinforcement is 3/4 times the value calculated using Section 15.7.2(2).

(ii) Diagonal tensile stress in concrete due to shear and torsional moment shall not be greater than the design tensile strength of the concrete. The design tensile strength of concrete may be calculated using the concrete compressive strength developed at the time of the verification with a value of 1.0 for γ_c.

(3) The maximum value of flexural and axial compressive stresses due to flexural moments, axial forces, and prestressing forces immediately after prestressing shall be 0.60 times and 0.50 times, respectively, of the characteristic value of compressive strength of concrete.

(4) Verification for the safety is carried out in accordance with the provisions described in Section 15.6 of the Specification.

[Commentary] The words "during construction" should include the periods during prestressing, immediately after prestressing, and other stages until the structure is actually in-service.

(1) The maximum permissible values for the tensile stress in prestressing steel during prestressing, P_i, and immediately after prestressing, $\{P_i - \Delta P_i(x)\}$, respectively, in Eq. (15.3.1), should be determined considering the safety for both – failure and yielding, of the tendon.

(2) In principle, occurrence of cracks during construction is not permitted because of the following reasons:

i) It difficult to control the crack width during construction.

ii) It difficult to control deflection after the stiffness decreases on account of cracking.

iii) Shrinkage and creep behavior of cracked concrete in the compression region at the serviceability limit state is not adequately understood.

In cases when the above mentioned conditions can be appropriately resolved, cracking during construction may be permitted. The maximum permissible value for the flexural tensile stress should be determined on the basis of considerations such as the combination(s) of loads during construction, magnitude of the flexural tensile stress, and the time when the flexural tensile stress acts.

When a tensile stress occurs for a short period during construction the amount of tension reinforcement may be reduced to 3/4 of the value required as per this section. The reduction is allowed on the basis that while the structure is in-service, concrete is usually under compression, and the amount of tension reinforcement required at that state has been independently calculated. However, the reduction may not be recommended in cases when the tensile stress acts for an extended period, because the ensuing crack width may grow considerably because of creep of concrete.

(3) If the ratio of stress in concrete immediately after prestressing, to the compressive strength is too high, secondary forces may develop causing non-linearity in the relationship between stress and strain. Further, the creep coefficient may be different from the value assumed in design. It is because of these reasons that a maximum permissible value is imposed in the present section.

(4) If safety during construction is to be verified, the material factor γ_c of 1.0 may be used as a characteristic value of compressive strength of concrete at the time of verification to calculate the design compressive strength, etc., of concrete.

15.9 Structural Details

15.9.1 General

If the performance verification of a prestressed concrete structure is made by any of the methods described in this chapter, the structural details specified in Chapter 13 and this section shall be followed.

[Commentary] This section specifies prerequisites for verification methods applicable to prestressed concrete structures such as requirements related to grout for prestressed concrete, requirements related to tendon arrangement such concrete cover over tendons and tendon spacing, and requirements for concrete reinforcement in anchorage zones. Concerning other details common to reinforced concrete structures, Chapter 13 must be followed.

15.9.2 Prestressed concrete grout

(1) Prestressed concrete grout shall meet quality requirements so as to fill prestressed concrete ducts completely and thereby protect the tendons and prevent them from corroding and strength requirements so as to make possible the structural integration of concrete members and tendons.

(2) Prestressed concrete grout shall be grout that has been verified, by the methods described in the Construction section of this Specification, to meet the quality and strength requirements. Similarly, it shall be verified that the arrangement, shapes, etc., of the prestressed concrete grout, sheaths and tendons used are combinations that have been verified to ensure complete filling of prestressed concrete ducts with grout.

[Commentary] (1) The purpose of prestressed concrete grout used in conjunction with bonded internal tendons is to protect tendons from corrosion and structurally integrate concrete members with tendons through bonding by filling prestressed concrete ducts. Incomplete prestressed concrete grouting might cause problems such as the corrosion of prestressing tendons, concentrated cracking and even a rapid fall in member strength due to the breaking of tendons. Because the degree of completeness of prestressed concrete grouting greatly affects the performance of the prestressed concrete constructed by using internal tendons, prestressed concrete grouting must be performed completely so that there is no harmful residual air.

(2) The verification of the completeness of prestressed concrete grouting is usually done at the construction planning stage by the method described in the Construction section of this Specification. At the design stage, the prestressed concrete grout to be used and the arrangement, shapes and other details of sheaths and tendons should be selected by referring to relevant information such as verified and proven examples of successful prestressed concrete grouting projects.

When selecting prestressed concrete grout, it must be verified that the selected grout meets the following quality and strength requirements:

(i) The selected prestressed concrete grout has adequate fluidity to fill the entire cross section uniformly.

(ii) Neither bleeding nor excessive changes in volume do not occur.

(iii) Corrosive substances are not contained in a harmful quantity.

(iv) The grout has adequate bond strength to structurally integrate concrete members and tendons.

It is also necessary to select such details as the diameters of sheaths to be used and the locations of intermediate vents and indicate them clearly on the design drawings.

(i) Sheath diameters (porosity) and intermediate vent locations should be determined so that harmful residual air is not left in the ducts.

(ii) Types of structure of inlets and outlets and grout hose diameters that minimize pressure loss should be selected.

(iii) Anchorage elements that enable complete filling of the space around the prestressing tendon up to the anchorage end should be used.

(iv) Systems that enable monitoring of grouting and the reinjection of prestressed concrete grout should be used on an as-needed basis.

The verification method described in the Construction section of this Specification is based on full-scale experiment results or past construction results. If, therefore, the amount of change in the angle of bend in tendons exceeds the range of full-scale experiment results or past construction data, it may not be possible to verify the completeness of filling by the verification method described in the Construction section of this Specification. In such cases, it is important to conduct prestressed concrete grouting tests, etc., in advance and confirm that complete prestressed concrete grouting is possible.

15.9.3 Cover of prestressing tendon

Cover for a prestressing tendon, sheath or group of sheaths, and anchoring devices shall not be less than the values given in Section 13.2 of the Specification. However, these values may not be applied to the end regions of a pre-tensioned member provided the prestressing steel in that region is specially treated for corrosion resistance.

[Commentary] In ordinary post-tensioned members, sheaths are often surrounded by stirrups, longitudinal reinforcing bars, etc. In such cases, the requirements for tendons and sheaths specified in Section 13.2 can be met by specifying concrete cover requirements for the reinforcing bars closest to the concrete surface. If, however, sheaths are not surrounded by stirrups, etc., usually a concrete cover not smaller than the sheath diameter should be used.

At the ends of pretensioned members, sufficient cover should be provided to ensure that damage on account of chloride induced corrosion does not cause damage in those portions. In cases when special countermeasures are taken, though the appropriate thickness of the cover may be determined separately, the efficiency of the countermeasures needs to be confirmed.

15.9.4 Clear spacing between tendons

Clear spacing between tendons or sheaths shall be determined in view of such factors as the type of tendon and the diameter of tendons or sheaths so that the space around tendons and sheaths can be completely filled with concrete, concrete can be compacted in reliable way and adequate bonding can be achieved.

[Commentary] In the pretensioning method, it is necessary to secure adequate clear spacing between prestressing tendons and between prestressing tendons and reinforcing bars. Similarly, in the post-tensioning method, it is necessary to secure adequate clear spacing between sheaths and between sheaths and reinforcing bars. If sheaths are bent, sufficient clear spacing should be secured so as to prevent the concrete between the sheaths or the sheaths from being damaged because of the bearing pressure of tendons acting on the inside surfaces of the sheaths.

With respect to details, the Design: Standards, Part 5, of this Specification must be followed.

15.9.5 Arrangements of tendons

(1) Prestressing tendons shall be so arranged that the loss in prestress due to friction is small and that there is no abrupt change in the cross-sectional area of the tendons throughout the length of the member.

(2) Bonded internal tendons should be placed in a pattern that has been verified, by the method described in the Construction section of this Specification, to enable complete prestressed concrete grouting.

(3) Prestressing tendons shall be extended straight for the required length from the bearing face of the anchoring device.

(4) When the tendon profile is curved, except in special cases, the radius shall be so determined that the loss in prestress is as small as possible and that the bearing stress acting on the concrete is within acceptable limits.

(5) In region(s) where the member may be subjected to reversal of moments depending on the combination of loads, tendons should be evenly distributed avoiding concentration.

(6) At the end support of a girder, some of the prestressing tendons should be provided along the bottom face and anchored in a region near the bottom face.

[Commentary] (1) Loss in prestress due to friction is proportional to the sum of the angular changes and the length of the tendon, and therefore, the effect due to such losses could be very large for a long tendon having a curved profile such as tendons used in a continuous girder. In such cases, the implementation of measures to make bend angles smaller such as placing prestressing tendons linearly as much as possible and anchoring prestressing tendons that have many bent sections at intermediate points along a girder in order to minimize friction-induced loss. If, however, prestressing tendons are anchored at intermediate points along a girder, it is necessary to avoid anchoring many prestressing tendons in the same cross section.

(2) Placement patterns of bonded internal tendons may be determined by the method described in the Construction section of this Specification.

(3) In order to prevent non-axial forces from acting in prestressing tendons during tensioning operation, tendons in a section of a certain length from the bearing surface of the anchorage element must be placed linearly in parallel with the axis of the anchorage element.

(4) When tension is applied to a prestressing tendon with a curved profile, a bearing force in the radial direction of the curvature acts on the concrete. To avoid excessive bearing stresses in concrete, the minimum inner radius of prestressing tendon should be approximately 40 times the diameter of the or more tendon when the tendon is in direct contact with the concrete, or approximately 100 times the diameter of the tendon or more when the tendon is provided in a duct that is later grouted to develop bond stresses.

The words "special cases" in this clause include cases where a loop-shaped tendon is embedded and anchored in the concrete, where a loop-shaped tendon is anchored using an anchoring device. In such cases, examination for safety should be carried out on the basis of experiments.

(5) In regions subjected to reversal of moment depending on loading conditions, tendons should be evenly distributed. Concentration of tendons tends to cause cracks in the concrete due to lack of reinforcing steel in regions near the top and bottom surfaces. Thus, prestressing tendons should be appropriately distributed in regions near the top and bottom surfaces (see Fig. C15.9.1).

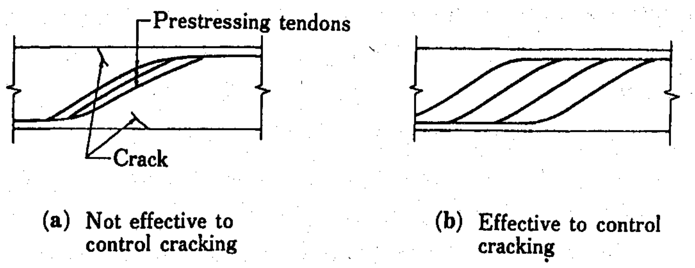


Fig. C15.9.1 Arrangement of prestressing tendons in region subjected to flexural moment reversal

(6) In cases tendons cannot be arranged as described above, additional longitudinal bars (instead of prestressing tendons) shall be provided as reinforcement.

15.9.6 Anchoring and connection of tendons and reinforcement of concrete in anchorage zones

(1) Anchorage elements and couplers for tendons for a member shall be arranged so as to ensure that the required prestress can be introduced into the member and shall be adequately protected in order to prevent damage and corrosion during the design service life of the structure.

(2) Cross-sectional shapes and dimensions shall be determined so that harmful cracks do not occur in the concrete in the anchorage zones because of the tensioning and anchoring of tendons, and the concrete in the anchorage zones shall be appropriately protected with reinforcing bars.

[Commentary] (1) In cross sections near anchorage elements, disturbance is caused by the diffusion of prestressing force and localized stress concentration. The commonly used stress calculation approach for a cross section subjected to eccentric axial force cannot be used. Anchorage elements, therefore, must be located at distances required for the prestress to act effectively on the cross section under consideration. If anchorage element is placed at an intermediate location on a member, as a general rule it should be located in the compression zone of the member. The fatigue strength of anchorage elements is under the influence of stress fluctuations is usually lower than that of tendons. When placing anchorage elements at intermediate locations on a member, therefore, it is advisable to place anchorage elements at locations that are sufficiently distant from regions with large fluctuations of stress and locations where stress fluctuations are minimal.

In the post-tensioning method, anchorage elements for a member are in many cases located near the surface of the member. Because such anchorage elements are vulnerable to external impacts and rainwater, they need to be protected appropriately. In general, it is good practice to install anchorage elements in recesses formed in members and, after the tensioning is completed, backfill those recesses with a backfill material such as concrete or mortar. In such cases, it is important to secure a concrete cover not smaller than the minimum concrete cover determined in the durability-related verification and carefully carry out backfilling in order to prevent detachment and chipping over time. Also, it is desirable that waterproofing measures be taken at backfills. It is also recommended that if anchorage elements are to be exposed for the purpose of inspection, re-tensioning, etc., in future, appropriate measures such as using replaceable rust-preventive materials be taken to protect the anchorage elements. In the pretensioning method, it is necessary to take appropriate rust prevention measures for anchorage end zones in order to protect tendons from corrosion.

Couples, as a general rule, should be placed near the centroid of the cross section or at locations where the amount of change in bending moment is small. If tendons are to be connected together in a curved section, sections of a certain length that adjoin the coupler must be straight, and these straight sections must be aligned with the coupler.

(2) If tendons are tensioned and anchored in the post-tensioning method, the concrete receives a concentrated load from the anchorage elements. In the pretensioning method, which achieves anchoring by means of the bonding of concrete and steel, each cross section in the development length zone receives a concentrated load. Because of these concentrated loads, tensile stress acts on the concrete near the anchorages. In order to prevent harmful cracks, therefore, it is necessary to reinforce the concrete with reinforcing bars. The concrete near anchorages may be reinforced in accordance with the Design: Standards, Part 5, of this Specification.

It is a standard requirement that the minimum spacing of anchorage elements, minimum concrete cover, etc., be determined experimentally. If a long and widely used anchoring method that has been proved to be sufficiently safe is used, the cross-sectional shapes and dimensions of anchorage concrete may be determined and anchorage elements placed by using that method.

If unbonded prestressing tendons or external tendons are used, tension fluctuations are transmitted directly to anchorages. It is therefore necessary to use structures capable of smoothly diffusing anchorage reaction forces and transmitting them to girders in a reliable way. It is also necessary to ensure safety from local stresses such as bearing pressure stress, splitting stress and back tensile stress and from bending stress and shear force. When tendons are anchored, usually many tendons are concentrated in small anchorage zones. It is advisable, therefore, to adequately reinforce such tendons not only individually but also as groups.

15.9.7 Minimum amount of reinforcement

(1) Minimum amount of reinforcing steel for prestressed concrete members shall be 0.1% of the gross area of the concrete section. The reinforcing steel mentioned here is deformed bar and bonded tendon.

(2) For the reinforcing steel meeting the provisions of Section 15.7.2(2), the diameter of the bars shall not be less than 9mm, and they shall be arranged in a manner that the spacing between bars is not more than 300mm.

(3) Across joints in precast members, these provisions of minimum amount of reinforcement may not be applied.

[Commentary] (1) Cracks due to shrinkage and temperature gradient may occur in concrete in prestressed concrete members even at the stage when the prestress has not been introduced. Therefore, at any section, a minimum amount of reinforcing steel, equal to 0.1% of the gross area of the concrete section, shall be provided to essentially prevent occurrence of such detrimental cracks.

For prestressed concrete members using the post-tensioning or pre-tensioning system, it is recommended that the amount of total amount of steel, including the bonded tendons should be less than 0.15% of the gross area of the concrete section (Section 13.4.1(1)). This provision, however, does not need to be followed in connection with prestressed concrete girders or prestressed concrete slabs constructed by the pretensioning method if it has been verified, for example, through studies on manufacturing methods, that harmful cracks do not occur in such girders or slabs.

External tendons must not be included when calculating the minimum amount of steel because external tendons are not bonded to concrete members.

(2) The 'reinforcing steel' referred to in this clause includes deformed reinforcing bars, pre-tensioning prestressing tendons, or grouted post-tensioning prestressing tendons.

(3) Since the provision for minimum amount of reinforcement is made primarily to control harmful cracks due to shrinkage or temperature gradient, the provision may not be applied in the case of joints of precast members.

15.10 Precast Concrete

15.10.1 General

(1) Concrete members or products produced in advance at a factory or at a field production facility, shall be considered as precast concrete.

(2) Precast concrete shall be designed to ensure that it meets the required criteria for performance and safety as well as economy, when used as a single unit, or as part of structure.

[Commentary] (2) Usages of precast concrete can be as a single item, or several members fabricated together or jointed together with cast-in-place concrete. It is necessary therefore for both of single item use and the whole assembly of precast concrete to have adequate safety during construction and service periods; or to possess enough functionality under ordinary usage; and to have enough during target design life.

The Japanese Industrial Standard (JIS) is enacted on many of factory products among precast concrete. Upon designing factory products, specified values for cracking strength, ultimate strength, etc. shall be satisfied. The provisions for durability may adopt easier conditions when factory products can be replaced easily by new ones. It is also necessary to design structures taking into account the harmony with surrounding environment. For this purpose, coloring and/or special surface treatment may be applied to factory products.

15.10.2 Shrinkage and creep of precast concrete

Values adopted for the shrinkage strain and creep coefficient for factory made precast concrete products should in general be experimentally verified.

[Commentary] Behavior of shrinkage and creep of precast concrete depends on various factors, such as mix proportion, methods of casting and curing. Their values shall be those confirmed by experiments.

Influence of accelerated curing is large, and shrinkage and creep become smaller compared with the cases of otherwise at the same age. It is reported in the case of steam curing that the shrinkage and creep after the cure decreased by 20 to 40 % compared with those without steam curing. In the case of autoclave curing, the effect is particularly large making respective values to 1/4 to 1/6.

15.10.3 Relaxation ratio of prestressing steel

(1) In the case of precast concrete made by pretensioning, the type of prestressing steel, the tensile force initially applied to the prestressing steel and the methods of production and curing, shall be taken into consideration when determining the apparent relaxation ratio of prestressing steel for calculating the loss of prestressing.

(2) In the case of precast concrete made by post-tensioning, values as shown in Table 5.3.1 may be used as the apparent relaxation ratio of prestressing steel for calculating the loss of prestressing.

[Commentary] (1) Relaxation progresses considerably prior to transfer of prestress for pre-tensioning system. Particularly under high temperature such as during steam curing, there is tendency of increase in amount of relaxation and there even are reports that the values at 65˚C are three times of that at the normal temperature. On the other hand the relaxation is small after returning to the normal temperature and there are reports that no difference between the two cases id found in the long run. Therefore, the apparent relaxation ratio should be determined, as a rule, taking into account the difference, such as by type of prestressing steel; period until the prestressing and method of concrete curing.

Further, low relaxation prestressing steel has especially smaller amount of increase under higher temperature than other types of prestressing steel.

After applying prestress, the apparent relaxation ratio of prestressing steel as shown in Table 5.3.1 which is for the case of post-tensioning system, may be adopted. When steam curing is applied as stated above, there is a tendency of relaxation being reduced after prestressing and the values shown in Table 5.3.1 may be altered after confirming through experiments.

(2) For post-tensioning system, high temperature curing is not applied after prestressing and

there is no difference from ordinary prestressed concrete. Therefore the values provided in Section 5.3.7 shall be adopted.

15.10.4 Loads

Precast concrete shall be designed considering not only ordinary design loads but also loads that may act during storage, transportation, fabrication, jointing and others.

[Commentary] Upon designing of precast concrete, loads for design of ordinary structures including those which may apply during construction and design life, shall be considered. Apart from these, loads during storage, transportation, fabrication, joints and others shall also be considered in order to avoid occurrence of critical cracks and to ensure safety. Reinforcing shall be necessary in some cases.

15.10.5 Unit weight

In general, the unit weight of precast concrete should be determined using experiments.

[Commentary] Unit weight of precast concrete is affected by various reasons, such as aggregates; mix proportions, amount of steel and method of casts. Therefore, the value confirmed by experiments is generally adopted. In case without experiments, the design values of unit weight of precast concrete to be used in designs may adopt those shown in Table C15.10.1.

Table C15.10.1 Unit weights of factory products

Compaction method	Unit weight (kN/m^3)
Vibrating compaction	25
Centrifugal compaction Pressurized compaction	26

15.10.6 Connection

(1) In cases when precast concrete is used in a part or the whole of a structure, the structural analysis shall be carried out, determining the load transfer capacity of connections with the consideration of the connection method.

(2) It shall be confirmed that the connection secure the required strength and the durability based on the connection method.

(3) Connections should, in principle, be provided at locations where the effect of loading is small

[Commentary] (1) The method stated below is usually adopted as the connection method of precast concrete.

i) The method to lap and anchor the reinforcement, furthermore to grout concrete or mortar

ii) The method to connect by prestressing force

iii) The method to connect steel products projected from precast concrete by tightening the bolt or welding

i) is the method that transforms a part of normal concrete of single cast and cast-in-situ into precast concrete. A large gap is required to secure the desired development length of reinforcement. However, the connection can secure the same level of stiffnesses and durability as in precast concrete.

ii) is connection method usually used in precast concrete, the connection shall be designed based on Section 15.10.7.

iii) is the connection method adopted for the shield segment and others. The advantage is not to require the form and curing, because of no casting concrete or mortar in the connection. However, it is necessary to arrange in highly accurate position and dimensions at the manufacturing, assembling, and connecting, and to note sufficiently partial reduction of stiffnesses, durability and fire resistance.

(2) When the connection is special structure, it shall be confirmed by the experiment for this structure to suit the purpose.

15.10.7 Joining by prestressing force

(1) In cases when independently fabricated precast elements are joined by prestressing, the location and details of these joints shall be fully examined, and it shall be ensured that the structure or member has the required strength.

(2) Joint structures, etc., shall be as specified in Items (i) to (iv) below.

(i) The angle between the face of the joint and the resultant compression force acting on the joint should, in general, be 90°. This angle shall not be smaller than 45 degrees.

(ii) Any special measures to be taken for preparation of the surface of the joint, should be clearly specified on the drawings.

(iii) Use of multiple keys shall be made to ensure better contact. Appropriate reinforcement shall be provided at the ends of a member near joints or in regions adjacent to keys, as these locations are potential weak-spots.

(iv) Examination shall be carried out for the water-tightness of the joint.

(v) Tendons across joints of precast members generally are arranged in regions without tension reinforcement so that when internal cable is used, the tendons should be bonded by grouting and some of these are arranged near the tension edge. In cases when only unbonded tendons or external cable are provided, care should be taken in choosing the method of calculation of failure strength and other details.

(3) Verification related to safety shall ascertain that Items (i) and (ii) below are met.

(i) Design shear capacity of the joint shall exceed the actual shear force. The design

shear capacity, V_{cwd}, of the joint may be calculated according to Section 9.2.2.5 considering the shear capacity of joint key.

(ii) key shall be safe against the actual bearing stress.

[Commentary] (1) When independently fabricated precast elements are joined by prestressing, the structural characteristics of the joint depend on several factors such as location of the joint, properties of the materials used for the joint, level of prestress. Therefore, depending upon the required condition of the joint, care should be taken in choosing the structural type and materials of joint.

(2) (i) When the angle between the face of the joint and the direction of resultant force acting on the joint is not equal to 90°, shear force acts along the joint. In such cases, unless effective countermeasures are taken, the precast elements may move relative to each other along the face of the joint. When that angle is between 70° and 55°, appropriate measures such as chipping should be taken. When that angle is between 55° and 45°, a joint key shall be provided on the face of the joint.

(ii) Widely used methods of joining precast concrete members include the method of directly joining members by use of adhesive and the method of joining members by using cast-in-place mortar or concrete. It is necessary to indicate on the design drawings joint details including the methods of treatment of the faces in contact with the joints of precast concrete members described in the preceding section.

When mortar is used in the joint(s), cracking often occurs in the joint mortar. Further, since mortar tends to have relatively large shrinkage and creep, the width of the joint should be as small as possible. From the viewpoint of workability, the width between 10mm and 60mm may be used. In cases when the mortar is applied on the joint face a width between 10mm and 20mm is recommended, in which the joint should be fastened using prestressing before the mortar hardens. A width between 30mm and 60mm is recommended in cases when the joint is fastened by prestressing after the strength of the cast-in-situ mortar reaches the required value.

When concrete is used in the joint, the width of the joint should be small so that the amount of in-situ concrete used can be kept small. In such cases, the joint width may be kept at a level that is higher than that determined on the basis of considerations such as reinforcement layout and casting of concrete. When the reinforcing bars are used as tension reinforcement, the joint width should be determined so as to provide the required lap splice length specified in Section 13.7.

Further, in the early stages after concrete casting, more cracks often occur in the joint region than in other portions. When the examination for the serviceability limit state is carried out for structures, where cracking is considered detrimental, conditions a) or b) below should preferably be satisfied.

a) When mortar or adhesives are used for the joint, adequate compressive stress should remain in concrete around the joint at the serviceability limit state.

b) In cases when concrete is used for jointing and tension reinforcement is provided to control crack width in the concrete in the region of the joint, the tensile stress in concrete due to bending and axial forces for evaluation of the serviceability shall not exceed the flexural strength of concrete. In cases when effective tensile reinforcement is not provided, provisions of a) above should be adhered to.

(iii) When adhesives are used at the joint, the opposite faces should be firmly in contact with each other. For this reason, appropriate methods such as match-casting method are used to manufacture precast members. To facilitate re-construction the desired shape, a joint key is preferably provided as a guide.

Joint key is provided to not only serve as a guide for re-construction, but also to increase the shear capacity. Especially, in case that segments are erected one by one such as cantilever erection method, the axial force at joint is comparatively small. Therefore, joint keys shall bear almost shear force. It should be noted that the shape of the concrete shear key is decided to ensure shear failure but bearing failure. The shape given in Fig. C15.10.1 may be used as a reference in deciding the shape of the shear key. In case a steel key is used, the details should be decided with consideration to the mode of failure.

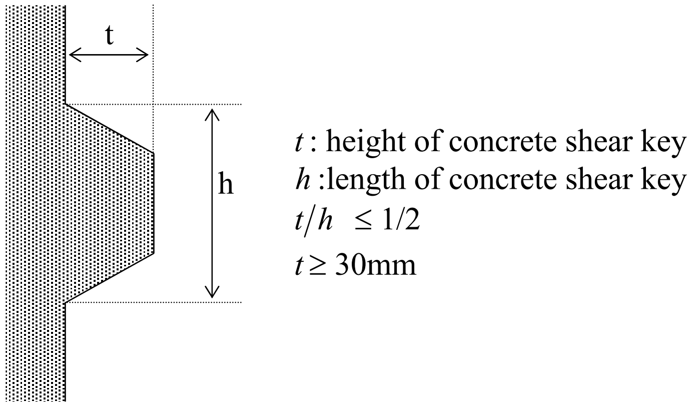


Fig. C15.10.1 Shape of concrete shear key

(iv) Joints often lack watertightness so that evaluation for watertightness should be taken according to Section 10.6. For joints in structures where presence of water is considered detrimental, appropriate steps to ensure waterproofing should be taken.

(v) Cracks tend to concentrate near the joint of precast members in cases tensile reinforcement is not provided. However, the width of cracks near the joint does not become too large provided the tendons are bonded and some of tendons are provided around the tension edge. If the latter tendons are not provided, the crack-width may become large, resulting in a reduction in the ultimate strength. Experimental studies have shown that the reduction in ultimate strength can be controlled to a large extent by ensuring that compressive failure of concrete does not occur. However, as it is not clear how much reinforcement is necessary, a careful consideration of all aspects should be taken.

(3) (i) As tensile stress does not occur at joints for serviceability verification, friction resistance by prestressing force can be expected. Therefore, verification for shear force may not be generally necessary.

Shear forces for safety verification at joints may be taken into account in the following manner:

Design strength in shear V_{cwd} may be calculated by Eq. (C15.10.1) by setting β_M=0 in Eq.(9.2.21).

$$V_{cwd} = V_{cwd,c} = (\tau_c \times A_{cc} + V_k)/\gamma_b \qquad \text{(C15.10.1)}$$

$$\tau_c = \mu \times f'_{cd}{}^{b} \times \sigma_{nd}{}^{1-b} \qquad \text{(C15.10.2)}$$

$$\sigma_{nd} = -(1/2)P'_c / A_{cc} \tag{C15.10.3}$$

where, σ_{nd} : average compressive stress which acts to shear plain vertically

P'_c : axial force which acts in compression zone of the member cross section

A_{cc} : area of shear plane in compression zone

b : coefficient that indicates plane configuration (0−1), which is 1/2 in case of joint of precast member by adhesive agent

μ : average friction coefficient of solid contact, which may be 0.45

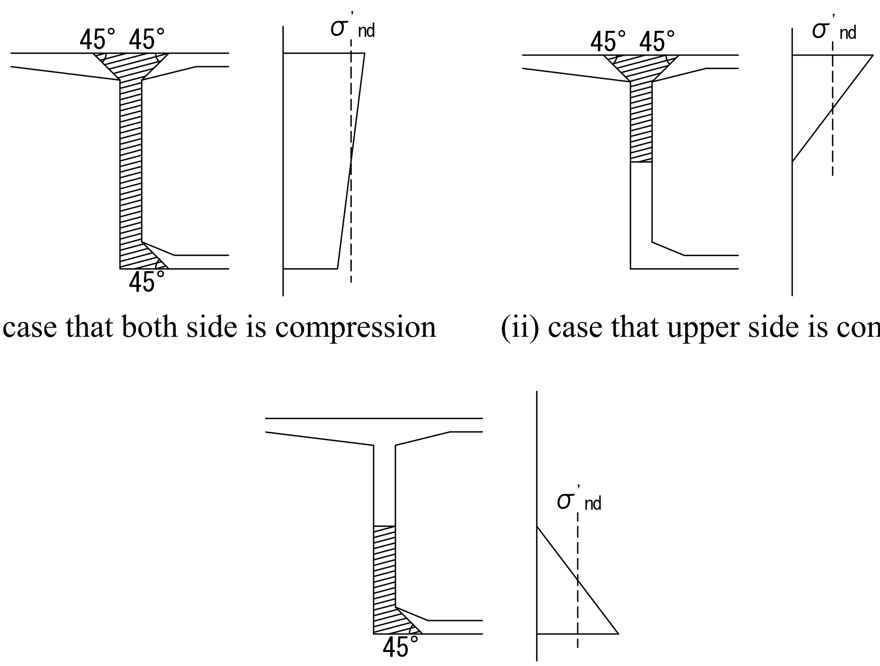
V_k : shear capacity by shear key

for multiple key or corrugated key,

$V_k = 0.1A_k \times f'_{cd}$, A_k : area of compressive side of shear key

γ_b : generally it may be 1.3

Area of shear plane A_{cc} should be the area indicated by Fig. C15.10.2, for shear force is transmitted through only web part.



(i) case that both side is compression (ii) case that upper side is compression

(iii) case that lower side is compression

Fig. C15.10.2 How to estimate the compressive area at shear plain

In cases when a steel shear key is used, V_k may be calculated by Eq. (C15.10.4).

$$V_{ks} = N \times A_k \times f_{vk} \tag{C15.10.4}$$

where, V_{ks} : shear force that steel shear key can carry

N : number of steel shear key

A_k : area of steel shear key (Fig. C15.10.3)

f_{vk} : limit value of shear stress that shear key can carry

f_{vk} may be 240 N/mm^2 at ultimate limit state when its material is FCD450.

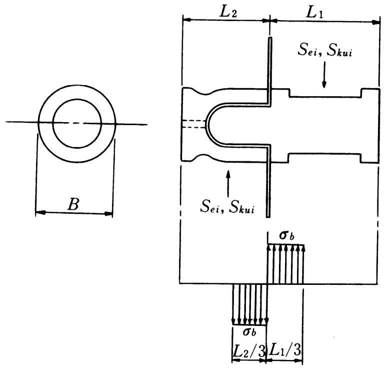


Fig. C15.10.3 Bearing part of steel shear key

(ii) Bearing stresses acting at the shear key may be calculated as follows:

i) in the case of using steel shear key

$$\sigma_a = (V_k / N)/(B \times (L/3)) \tag{C15.10.5}$$

where, σ_a : bearing stress that acts to single steel shear key at execution or ultimate limit state

V_k : shear force at execution or ultimate limit state

N : number of steel shear key

B : outer diameter of steel shear key

L : embedded depth of steel shear key

ii) in the case of using concrete shear key (Fig. C15.10.4)

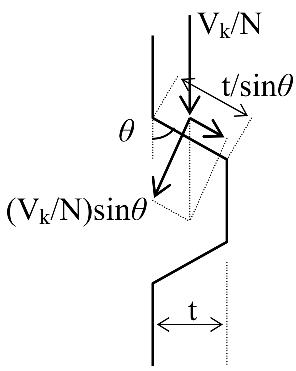


Fig. C15.10.4 Calculation method of bearing stress of concrete shear key

$$\sigma_a = (V_k / N) \cdot \sin\theta / (B \cdot t / \sin\theta) = (V_k / N) \cdot \sin^2\theta / (B \cdot t) \quad \text{(C15.10.6)}$$

where, σ_a : bearing stress that acts to single concrete shear key at execution or ultimate limit state

V_k : shear force at execution or ultimate limit state

N : number of concrete shear key

B : width of shear key

t : height of shear key

θ : angle between shear key and vertical line

Bearing strength of shear key is calculated according to Section 5.2.1(4).

15.10.8 Concrete cover

(1) Concrete cover for precast concrete shall not be less than the diameter of reinforcing bars.

(2) Minimum concrete cover for mass-produced factory products shall be as shown in Table 15.10.1.

Table 15.10.1 Minimum concrete cover (mm)

Classification of factory products: Replacement	Classification of factory products: Compaction method	When a member is exposed to atmosphere, or contacts ground or water directly, or durability is needed to be taken into account	When a member is not exposed to atmosphere, or is embedded in cast-in-place concrete, or durability is not needed to be taken into account
Difficult to replace	Vibrating compaction	20	10
	Centrifugal compaction	15	10
Relatively easy to replace	Vibrating compaction	12	8
	Centrifugal compaction	9	8

Note : When a product contacts water or atmosphere containing acid or other harmful material, or when there is a possibility of abration, the cover shall be larger than the number specified above, or appropriate measures shall be taken.

(3) Minimum concrete cover for precast concrete exposed to the effect of chloride shall conform to the provisions of Chapter 8, or appropriate alternative measures shall be taken.

[Commentary] (1) and (2) The provisions are ruled taking into account; i)the Japanese Industrial Standard (JIS) is applied to many of mass-produced factory products among precast concrete, and ii) for precast reinforced concrete and precast prestressed concrete, when compared with the ordinary reinforced concrete and prestressed concrete structures, the water to cement ratio is smaller, more compaction is conducted, dimensions of formworks are more accurate and better quality control is maintained for reinforcement distribution, sizes of members are smaller making occurrence of cracks less likely, and replacing is easier. However, the values ruled here are minimum ones and it is necessary to deeply consider the type of precast product, method of curing;

importance as a structure; condition on service environment; design life; and others, when determining the minimum concrete cover. Also, when the thickness of factory products is particularly thin to apply the values shown in Table 15.10.1, and it is clear that it will not affect practically durability, then the concrete cover may be reduced slightly from the ones shown in Table 15.10.1. The minimum concrete cover, even in this case, shall be not less than the diameter of reinforcing steels.

When centrifugal compacting is applied, the compacting around reinforcement is better done than vibrating compaction and since it has been confirmed by past experiments, that a centrifugally compacted concrete product has better durability, concrete cover may be reduced compared with vibrating compaction. Also pressurized compacting may follow the case of centrifugal compacting.

When artificial lightweight aggregates are used, it is preferable to add 5 mm to the values shown in Table 15.10.1.

It is necessary to determine the concrete cover to secure enough durability for comparatively large-scale precast products such as girder of bridges, box culverts and segments, since they are of great importance and their surrounding environments of use very much. This is also necessary since JIS is not applied on large-scale precast products. In those cases, concrete cover shall preferably be conformed the provisions of Section 13.2, with taking into consideration, particularly, the importance, surrounding environments, designed life and others.

(3) When precast concrete is used in sea water or in atmosphere above the sea level as well as when anti-freezing or deicing agents are used, it is necessary to make the concrete cover larger than the values shown in Table 15.10.1 since chloride will seep in from the surface and accelerate corrosion of reinforcement. Further, it is necessary to make the concrete cover, as a rule, not less than the maximum size of coarse aggregates. Instead of making concrete cover, as a rule, not less than the maximum size of coarse aggregates. Instead of making concrete cover larger, reinforcement with anti-corrosion treatment or painting on surface of the concrete may be applied however ample examinations must be made on applications.

15.10.9 Clear distance between reinforcing bars

(1) Clear distance between reinforcing bars shall not be less than the diameter of reinforcing bars and not less than 5/4 the maximum size of coarse aggregate.

(2) In cases when precast concrete is made using the pretension method, the clear distance between prestressing tendons shall not be less than 5/4 the maximum size of coarse aggregate. However, in cases when adequate bond between concrete and prestressing tendons is needed at the end of a member, it shall not be less than 3 times the diameter of prestressing tendons.

(3) Clear distance between sheaths shall not be less than 5/4 the maximum size of coarse aggregate.

[Commentary] (1) to (3) As precast concrete are made under sufficient production control, the spacing between reinforcing bars may be smaller than that for ordinary concrete structures. When reinforcing bars, prestressing steels or sheaths are bundled together and production control is identical with cast-in-place concrete, the provisions of Section 13.3 and 15.9.4 shall be applied. However, the minimum clear distance between the bundled steel materials determined by the maximum size of coarse aggregate may still be 5/4 the maximum size of coarse aggregate.

CHAPTER 16 COMPOSITE STEEL AND CONCRETE STRUCTURE

16.1 General

(1) This chapter lays down specifications for design of structural members made using a combination of concrete and structural steel. Provisions are given for design of such members, including the limit states defined for checking durability, safety, and seismic performance procedure for examination of such limit states and structural details that are prerequisite for the examination.

(2) The provisions given here apply to following types of composite (i) steel reinforced members, (ii) concrete-filled steel columns, and (iii) steel-concrete sandwich members.

[Commentary] (1) The provisions of this chapter apply to members in which steel and concrete are mechanically composed to resist stress resultant. Such composite members include those where the structural steel is embedded inside concrete and those where the steel reinforcement is placed outside concrete.

Reinforced and prestressed concrete members made using conventional steel reinforcement are not considered 'composite members' as far as the provisions in this chapter are concerned. Apart from the three types mentioned in this chapter, several types of composite member have been proposed to date, and it can be expected that newer proposals will continue to be made in the future also.

Of the many possible types of composite members, the provisions in this chapter, in principle, cover only those general types that have been used in earlier construction, and for those, for which the methods of checking performance and methods of design are largely established. The design of other composite members, including detailing, may be carried out using the publication 'Guidelines for Performance Verification of Steel-Concrete Hybrid Structures (Draft)' of the Japan Society of Civil Engineers, the provisions of this chapter as a reference, and on the basis of appropriate experimental or analytical results. For parts relating to steel structures may be made to publications such as the 'Standard Specification for Steel and Composite Structures' of the Japan Society of Civil Engineers.

(2) The provisions of this chapter may be used for the design of the following types of composite members:

1) Composite beams: reinforced concrete slabs placed upon steel beams and interconnected with shear connectors.

2) Steel reinforced concrete members: members including reinforced concrete beams and columns that contain rolled steel sections, steel plates or built up sections such as I-section, in their sections.

3) Concrete-filled steel columns: column members consisting of steel tubes filled with concrete

4) Composite walls: wall members consisting of laterally continuous rows of steel columns covered by concrete or wall members consisting of laterally continuous rows of steel columns filled by concrete.

5) Composite slabs: slabs made of a combination of steel plates and plates and conventional reinforced concrete slabs interconnected with shear connectors.

6) Steel-concrete sandwich members: slab or wall members consisting of a concrete infill between two steel plates.

Representative examples of composite structural members are shown in Fig. C16.1.1.

Characteristics of composite members covered by this chapter are as follows:

(i) The structural steel reinforced concrete members are members that consist of rolled H-sections or Warren truss members incorporated into conventional reinforced concrete, and in which both combine to resist external forces. Such members can be categorized superimposed-type structures, which utilize the steel section with full web or reinforced concrete type structures where a combination of conventional steel reinforcement and structural steel is used, reinforcement depending on the arrangement of the structural steel and its bond with the concrete. In principle, conventional steel reinforcement, as main and shear reinforcement is arranged around the structural steel sections. If such reinforcement is not provided, adequate measures should be taken to protect the concrete against delamination and, if necessary, distribution of crack should be examined.

(ii) Concrete-filled steel columns are structural forms consisting of circular or rectangular steel tubes with concrete infill.

(iii) Sandwich members are structural forms consisting of two steel plates interconnected by steel and with the space between them filled with concrete.

Figure C16.1.2 shows some of the structural forms described above. The concrete should be placed within the spaces between steel plates or inside tubes in a manner that no voids are left. For the design and construction of sandwich members, reference may also be made to "The Specification for Design and Execution of Steel-Concrete Sandwich Structures (Draft)" published by the Japan Society of Civil Engineers.

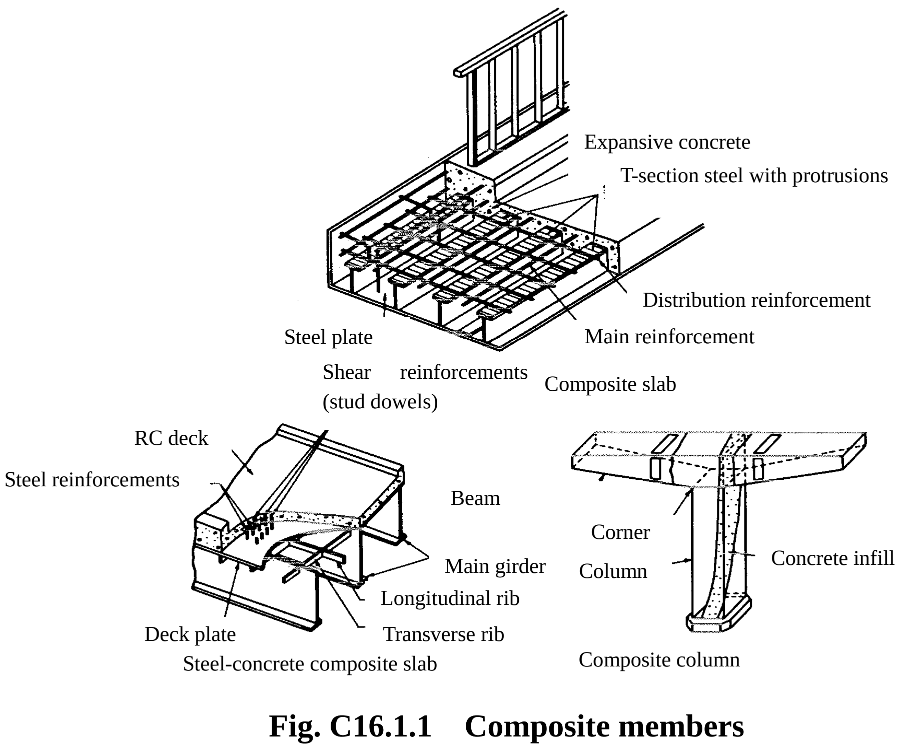


Fig. C16.1.1 Composite members

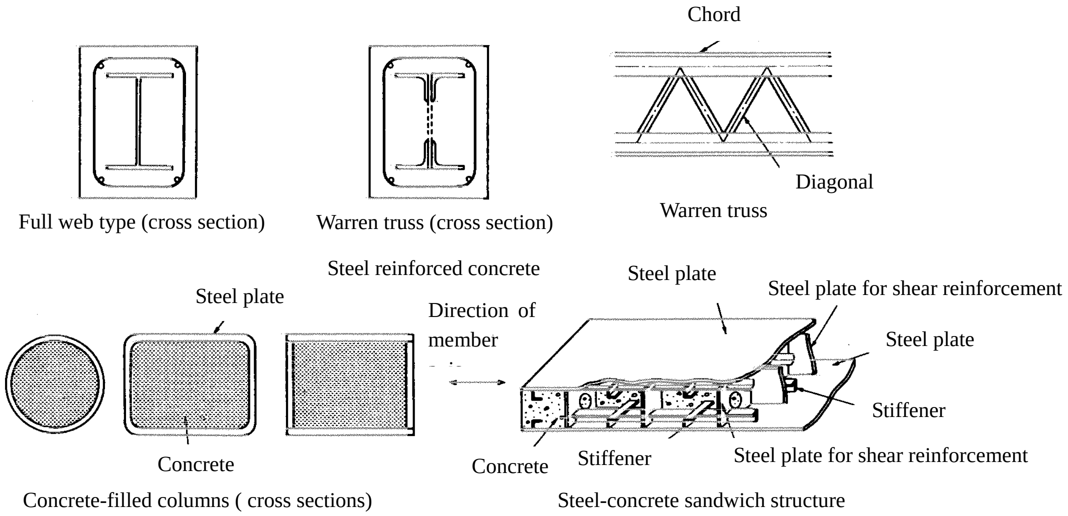


Fig .C16.1.2 Composite members covered

16.2 General requirements for composite structures

Composite members defined in this chapter shall satisfy the following requirements:

(1) There is a perfect bond between the concrete and structural steel, and the bond remains throughout the period when the structure is in service.

(2) Structural steel embedded in concrete does not buckle.

(3) The durability of composite members should be comparable to that of conventional reinforced concrete members. In cases when structural steel is placed outside the concrete in the composite members, the steel should be provided with an appropriate anti-corrosion coating, etc. depending upon the environment to which the structure or member is exposed.

(4) For structural steel arranged outside the concrete, appropriate fire-resistant cover etc. should be provided if the structure or member is likely to be exposed to very high temperatures such as in the event of fire.

[Commentary] (1) When considering the ultimate limit state of conventional reinforced concrete, a perfect bond (i.e. the average strain is the same in both materials at a given point) between the steel reinforcement and the concrete may be assumed. For composite members, however, the ultimate strength may be estimated either with the assumption that the steel and concrete have a perfect bond and their strains are continuous, or even without making that assumption.

(2) All steel embedded in concrete should be embedded in a manner that they do not buckle. When the steel plates (or sections) lie on the outside, enhancement of local buckling strength and/or the post buckling strength may be expected by means such as providing shear connectors or stiffeners.

(3) When structural steel is embedded in concrete, the provision of sufficient concrete cover should be ensured, along with appropriate means to disperse cracks and restrict the width of cracks, in a manner similar to the measures taken in the case of conventional reinforced concrete. In contrast, when such structural steel lies outside, the steel surface shall be painted or treated to ensure adequate durability depending on the specific environmental conditions. In cases where there is direct contact with seawater, the provision of heavy-duty corrosion protective coating or cathodic protection may be specially considered. When the structural steel lies on the outside of concrete, and it can be ensured that the enclosed concrete is completely sealed, the provision for restricting crack width does not apply from the point of view of durability.

16.3 Design Method

16.3.1 Selection of steel

In addition to steel generally used, steel developed especially for composite structures may be also used as the steel in plates and bar reinforcement.

[Comments] To have an adequately proportioned cross-section in the composite structure at both the safety and serviceability limit state, steels which have similar mechanical properties, such as yield strength and yield strain, should be used in the main elements of the composite member. Steel plates with protrusions or perforations may be used to augment the bond between the steel and concrete, or to disperse cracking. The provision of the present specification does not apply when materials such as carbon fibers or high strength steel with a yield stress in excess of 700-800 N/mm^2 are used.

16.3.2 Verification method of performance

(1) The structural performance shall be verified depending upon the type of the composite member.

(2) It shall be ensured that composite members meet the performance requirements both during erection and while in service.

(3) Durability of composite members may be verified according to Chapter 8 of this Specification. In cases the steel is provided with an appropriate anti-corrosion coating, etc. depending upon the environment to which the structure or member is exposed, examination for steel according to Chapter 8 need not to be carried out.

[Comments] (1) and (2) There are many different types of composite members, and it is difficult to lay down a single procedure to examine the limit states to cover all of the members. Therefore, the Specification allows and prescribes the use of separate methods for the examination depending upon the type of the member. Efficiency of erection can be enhanced and steel can be more efficiently used if steel is used as formwork or supports during erection, and then remains in position as part of a composite member after completion. It is necessary to examine the performance during the construction work and in service separately as (a) the loads acting during the construction and in service, and, (b) the required performance of the members, in these stages, can be different.

(3) Durability of steel-concrete composite member is necessary to be verified appropriately for the steel arrangement. If steel is embedded in concrete, an adequate concrete cover must be

provided. Measures must also be taken to distribute cracks, and crack width must be verified in accordance with Chapter 8 as in the case of reinforced concrete members. If steel is placed outside the concrete, it is necessary to take measures to meet durability requirements such as coating the steel surface. If the structure is contact with seawater, the use of heavy-duty coating or cathodic protection should be considered on an as-needed basis. In cases where steel is placed outside the concrete, if the concrete is completely closed, it is not necessary to follow the rules applicable to reinforced concrete members with respect to crack width limits from the viewpoint of durability. When steel is arranged in steel-concrete composite member, verification needs to be carried out according to Chapter 8 as in the case of reinforced concrete.

16.3.3 Shear connector

Adequate shear connectors shall be provided in the form of stud dowels, or steel shapes, etc.

[Commentary] Use of appropriate shear connector is required when shear forces need to be transmitted between structural steel plates and concrete. Though stud dowels are frequently used as shear connectors, similar role can also be expected using structural steel or reinforcing bars other than those shown in Fig. 11.3.1. Several types of shear connectors have been developed for bridge decks. In principle, the amount and layout of shear connectors should be decided to ensure that shear forces are adequately transmitted and no buckling of the structural steel plates occurs. Layout of the shear connector should be such that Eq.(C 11.3.1) is satisfied.

$$\gamma_i H_d \Big/ \sum_{i=1}^{n_{sc}} V_{scdi} \leq 1.0 \qquad \text{(C.16.3.1)}$$

where, γ_i : structure factor

H_d : design shear force acting per unit width between steel and concrete over a particular segment

V_{scdi} : design shear transfer capacity of each individual shear connector

n_{sc} : Number of shear connector within the area of calculating H_d

Ease of welding the shear connector and the thickness of the steel plates involved should be kept in mind when designing the shear connectors in composite members. Diaphragms or steel plates with protrusions, etc. may also be used as shear connectors though their actual shear strength should be determined through appropriate experiments, etc.

16.3.4 Examination of limit states during erection

Appropriate examination shall be carried out at the design stage to ensure the adequate strength of the structural steel and to ensure that the criteria for limits states for strength, deformation and buckling of the structural steel are satisfied. All the above criteria shall be met at both stages – before and after the steel and concrete is interconnected and for all the structural steel involved, including plates or built-up section, or rolled sections.

[Commentary] Steel is sometimes used as formwork for placing concrete or as a support during erection. The limit states during construction should be examined for both the stage before placing concrete and the stage when concrete is in fresh state after being placed. Stress having a similar distribution to fluid pressure is transferred from fresh concrete to the steel plates. In the case of high-flowable concrete, the distribution is rather close to that of hydrostatic pressure. It should be ensured that under the action of such distributed loads, the steel does not deform excessively and that the stresses generated are within acceptable limits. Thus, the properties of concrete used, rate of placing, atmospheric and concrete temperatures, and placement height should be taken into consideration when planning the construction work.

The stress induced in the steel plates during erection should in principle be superimposed when examining steel plate buckling. However it does not need to be superimposed in examination of ultimate limit state.

16.4 Examination of completed structures containing steel sections used during erection

The effect of structural steel to be used solely for the purpose of erection on the overall stiffness and the failure mode of the member after completion shall be appropriately considered.

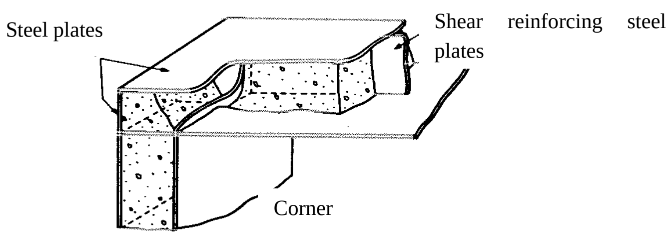
[Commentary] Structural steel sections, plates and built-up sections to be used solely for the purpose of erection are sometimes left in a member after the structure is completed. Since the presence of such steel is not accounted for at the time of design, the actual capacity of the member is larger than the estimation. However, this may also make a change in the overall stiffness and failure mode of the member, and these should be appropriately considered.

16.5 Structural Performance of Joints and Corners

(1) Joints of members and corners of frames shall be designed to appropriately transmit bending and torsional moments, and axial and shear forces, to adjacent members (beams, columns, etc.).

(2) Connection bases shall be designed to appropriately transmit all forces in the columns to the foundation(s).

[Commentary] (1) The actual distribution of stresses at the joints between members and in the corners of frames is complicated due to abrupt change of cross sections. Steel should be provided and arranged at the corners to appropriately transmit forces. If necessary, haunches should be used to provide for the corner. Figure C16.5.1 shows an example of the corner of a sandwich member.



Example of corner in sandwich member

Fig. C16.5.1 Corner structure

Column bases should be designed to ensure appropriate transmission of forces from the column to the footing, and prevent the column itself from being pulled out.　The capacity for transmitting forces can be checked using appropriate tests, non-linear analysis or equations, of which accuracy and applicability is already confirmed, or using the Part 6 of the Standard methods in this specification.

16.6　Effect of Shrinkage and Creep of In-Filled Concrete

The effect of drying shrinkage of concrete used to fill structural steel may, in principle, be neglected.　The effect of creep should appropriately be examined. If the structural system changes from that at the erection stage to that after completion, statically indeterminate forces produced by the creep of concrete shall be accounted for.

[Commentary]　In sandwich members and concrete-filled columns, the concrete infill is located in an airtight environment and therefore the drying shrinkage may be considered to be small. Consequently, drying shrinkage may be neglected.　In cases where creep, drying shrinkage, or autogenous shrinkage is expected to pose a difficulty, their effects should be appropriately examined.

16.7　Steel Reinforced Concrete Members

16. 7. 1　Classification of structural types

Provisions of this section shall be applicable to the following steel reinforced concrete members：

(i) Superimposed-type, which is characterized by presence of steel sections with full web or similar.

(ii) Reinforced concrete type, which is characterized by presence of rolled steel used as together with conventional reinforcing bars.

(iii) Erection type, in which structural steel rolled sections, steel plates or built-up sections are used solely for the purpose of erection, and their presence may be neglected in the structural analysis.

[Commentary]　Since it is difficult to provide definite specifications for design of various types of structural steel reinforced concrete, they have been classified into three types as above considering experiences in the past.　Appropriate provisions have been made separately for each type. In case of (iii), when steel plates or built-up sections are not taken into account in structural analysis, the design may be carried out in the same manner as for conventional reinforced concrete structures.

16.7.2 Examination of safety

16.7.2.1 Examination of limit state of failure of cross section

(1) Capacity of a superimposed-type composite member in flexure and under the action of flexure and axial forces shall be taken as the sum of the individual capacities of the conventional reinforced concrete components and that of the rolled steel sections, steel plates and/or built-up sections, computed separately. Similarly, the capacity of a reinforced concrete type composite member shall be computed in a manner similar to conventional reinforced concrete members, accounting for the presence of rolled steel sections, steel plates and/or built-up sections as equivalent reinforcing bars.

(2) Capacity of a superimposed-type composite member in shear shall be taken as the sum of the individual capacities of the conventional reinforced concrete component and that of the rolled steel sections, steel plates and/or built-up sections, computed separately. The shear capacity of a reinforced concrete type composite member shall be computed in a manner similar to conventional reinforced concrete members, accounting for the presence of rolled steel sections, steel plates and/or built-up sections as equivalent reinforcing bars.

[Commentary] (1) When moments are dominant in a superimposed type composite member having a symmetrical cross section, it may be assumed that moments are carried only by the conventional reinforced concrete together with rolled steel sections, steel plates and/or built-up sections, whereas the axial compressive forces are carried only by the conventional reinforced concrete. When axial compressive forces are dominant, it may be assumed that the axial compressive forces are carried by conventional reinforced concrete together with rolled steel sections, steel plates and/or built-up sections, whereas the moments are carried by rolled steel sections, steel plates and/or built-up sections alone.

(2) Shear capacity of rolled steel sections, steel plates and/or built-up sections, in a composite member can be computed using one of the following equations (refer to Fig. C16.7.1.):

Full-web girder type:

$$V_{sy} = f_{vyd} z_w t_w / \gamma_b \tag{C16.7.1}$$

where, V_{sy} : shear capacity provided by web of steel shapes, plates or fabrications

f_{vyd} : design yield shear strength of steel shapes, plates or fabrications

z_w : web height of steel shapes, plates or fabrications

t_w : web thickness of steel shapes, plates or fabrications

Warren truss type:

$$V_{sy} = f_{wsd} A_{sw} z_s \sin\theta / \gamma_b \tag{C16.7.2}$$

V_{sy} : shear capacity provided by Warren truss

f_{wsd} : design yield strength of the diagonals of steel shapes, plates or fabrications

A_{sw} : area of the diagonals of steel shapes, plates or fabrications

θ : angle between the axis of the diagonal and that of the chord

z_s　:　lever arm of steel shapes, plates or fabrications

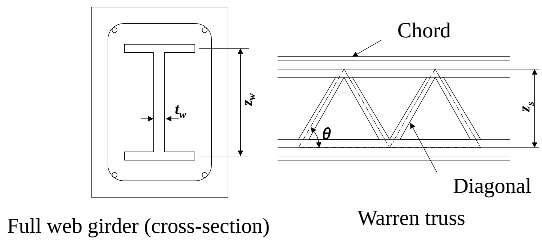


Fig. C16.7.1　Structure of steel shape plates and/or built-up section

16.7.2.2　Examination of limit state of fatigue failure

In cases where examination for fatigue limit state is required, the examination shall be performed separately for both rolled steel sections, steel plates and/or built-up sections and for conventional reinforcing bars.

[Commentary]　Presence of welds or forming holes in the rolled steel sections, steel plates and/or built-up sections, generally reduces the fatigue strength.　If the steel is in wet condition the fatigue strength may be further reduced.　Examination for the fatigue limit state should also account for such reduction in the fatigue strength of steel.

16.7.3　Examination of serviceability

(1) When examining the serviceability limit state in terms of crack width, it should be, as a rule, carried out in the same way as for conventional reinforced concrete, accounting for the presence of rolled steel sections, steel plates and/or built-up sections as equivalent reinforcing bars.

(2) Examination of displacement and deformation may be carried out as for conventional reinforced concrete members.

[Commentary]　Since the bond strength of the rolled steel sections, steel plates and/or built-up sections with the surrounding concrete is likely to be smaller than that of deformed reinforcing bars, there could be a tendency for the crack width to become larger as the ratio of the area of rolled steel sections, steel plates and/or built-up sections to the gross cross-sectional area of the section increases.　Methods of dispersing cracks including welding studs to the steel or using steel plates with protrusions should be used depending upon the requirements.　The permissible tensile stresses in steel has been imposed primarily because it is not appropriate to apply methods of crack width computation normally used for conventional reinforced concrete directly in the case of structural steel reinforced concrete. In general, tensile stresses in steel should not exceed the values given in Table C16.7.1.

Table C16.7.1 Upper limit of tensile stress in steel

A_{ss} / A_s	Upper limit of tensile stress (N/mm^2)
Not more than 30%	180
50%	160
70%	140

A_{ss} : cross sectional area of tension steel shapes, plates or fabrications

A_s : total cross sectional area of tension reinforcing bars and tension steel shapes, plates or fabrications

16.7.4 Structural details

(1) Maximum and minimum amounts of steel

i) For structural steel reinforced concrete members, the maximum area of the longitudinal steel including rolled steel sections, steel plates and/or built-up sections, and conventional reinforcement shall not exceed 8 % of the area of a concrete section. The minimum area shall not be less than a half of the minimum amount specified in Chapter 13. The minimum amount as tension reinforcement shall be provided in accordance with Chapter 13.

ii) The minimum shear reinforcement shall be in accordance with Chapter 13, and shall be provided in the forms of stirrups, ties, or hoop reinforcements enclosing all longitudinal steel. The area of stirrups, ties, and hoop reinforcement shall be not less than half the minimum value specified in Chapter 13.

(2) Cover

Concrete cover for structural steel in steel reinforced concrete should generally be 100mm. The cover for conventional reinforcing bars shall be conformed to that of conventional reinforced concrete.

(3) Connection of steel and end treatment

Structural steel such as rolled steel sections, steel plates and/or built-up sections shall be connected at sections where the capacity of the cross section has an enough margin for stress resultant. Connections of steel shapes, plates or fabrications shall be designed to possess at least 75 % of the strength of material. Care shall be taken that the cross section having such connections does not become the critical section.

Tension and compression steel in the free end of a cantilever and similar situations shall be sufficiently anchored.

(4) Spacing of reinforcing bars and steel

The conventional reinforcing bars and the rolled steel sections, steel plates and/or

built-up sections shall placed in a manner that the clear distance between the bars, etc. for the concrete to pass is neither less than 40 mm nor less than 4/3 the maximum size of coarse aggregate.

(5) Splices and anchorages of reinforcing bars

i) Splices for the conventional reinforcing bars and connections of rolled steel sections, steel plates and/or built-up sections shall not be located at the same cross section.

ii) Provisions for anchorages and splices of conventional reinforcing bars shall be in accordance with Chapter 13 of this Specification.

[Commentary] (1) Presence of excess amounts of steel affects the ease of placement of concrete, which in turn affects the quality of concrete. Splitting cracks may occur in such cases. The maximum amount of steel in a given member is specified assuming that construction is carried out in accordance with the minimum specified standards and that the constructability of concrete used meets the requirements of the Standards Specifications for Concrete Structures “Materials and Construction”. Provision for a minimum area of conventional reinforcing bars along the axis of the member has been made to prevent excessive cracking or spalling of concrete.

(2) A concrete cover of 100mm has been specified for rolled steel sections, steel plates and/or built-up section, taking into consideration various factors including workmanship, and also to prevent spalling of concrete. However, it is desirable to provide additional cover when using wide steel sections. Concrete cover for rolled steel sections, steel plates and/or built-up section should be taken as the clear distance between the surface of the rolled steel sections, steel plates and/or built-up section and the surface of concrete.

(3) In members such as cantilevers, simply supported beam, or, at the head of a column etc., the ends of structural steel such as rolled steel sections, steel plates and/or built-up section should be appropriately anchored in a manner similar to conventional reinforcing bars. However, since the bond strength between structural steel and surrounding concrete could be smaller than that in conventional reinforced concrete, use of special measures such as connecting tension and compression steel members in a composite section using other members is recommended. The members used for interconnecting should have stiffness comparable to that of the tension and compression members connected. However, in such cases, it is desirable that loading tests be carried out to check the shapes and dimensions of interconnected members.

16.8 Concrete Filled Steel Columns

16.8.1 Examination of safety

16.8.1.1 Examination of limit state of failure of cross section

(1) For flexure and axial forces, the ultimate capacity shall be computed as for a conventional reinforced concrete member, accounting for steel as equivalent reinforcing bars.

(2) Shear capacity shall be computed assuming that only the steel tubes carry shear.

16.8.1.2 Examination of limit state of fatigue failure

Examination of the fatigue limit state for the steel tubes shall be performed according to Chapter 9 of this Specification.

16.8.2 Examination of serviceability

(1) In general, examination for the crack width need not be carried out.

(2) Examination for displacement and deformation shall be carried out in a manner similar to that for conventional reinforced concrete members.

[Commentary] (1) In concrete filled steel columns, steel tube is arranged all around the concrete and as long as it can be ensured that this steel is sound, and the watertightness and airtightness are maintained, it may be assumed that any occurrence of cracking in the concrete will not have a large effect on performance of the member, such as its durability and/or the appearance. Thus, it is important to ensure that the steel tube does not lose its function of completely protecting the in-filled concrete on account of its interaction with the environment by corrosion. Any holes made in the steel to facilitate erection should be appropriately sealed.

16.8.3 Examination for placing concrete

When concrete is placed into steel tube, deformation and stresses on the surrounding steel due to lateral pressure induced by the concrete shall be appropriately examined

[Commentary] The lateral pressure of the concrete before it hardens acting on the steel tube is estimated taking into account various factors including the rate and height of concrete placement, air and concrete temperatures, etc and then the stress and deformation of the steel tube are calculated. If any construction of structure is to be carried out above the column, sufficient time should be taken to allow the concrete to cure. When using concrete having standard workability as prescribed in Specifications ”Materials and Construction”, concrete should be placed using tremies or pipes ensuring that any segregation does not occur. The concrete should be appropriately compacted by vibrator, etc and placed gradually steadily starting from the bottom of the column. High-flowable concrete may be used in cases it is difficult to compact the concrete sufficiently.

16.8.4 Structural details

(1) Maximum and minimum amounts of steel

For concrete filled steel columns, the maximum and minimum amounts of rolled steel sections, steel plates and/or built-up section shall be such that the proportion of axial forces to be borne by the structural steel is between 20% and 80% of the total axial compression forces borne by composite member. The minimum thickness of steel plate should, in principle, be 8 mm.

(2) Manholes and openings

All manholes and openings shall be sealed by welding with steel plates after

completion of concrete placing.

(3) Treatment of inside of steel tubes

The steel surface of steel tube in contact with concrete, shall be unpainted, and any oil, grease, loose rust or mill scale shall be removed before the concrete is cast.

(4) Vents

Appropriate air vents shall be provided, especially in corners, etc., to prevent air from getting trapped and enable concrete to fill all space.

16.9　Sandwich Members

16.9.1　Examination of safety

16.9.1.1　Examination of limit state of failure of cross section

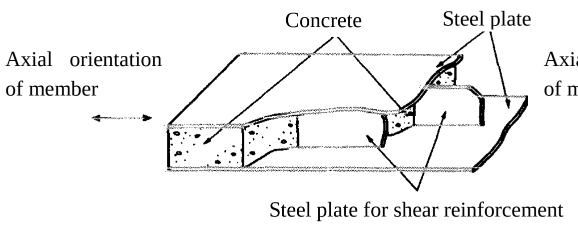
(1) For flexural and axial forces, the ultimate capacity shall be computed as for a conventional reinforced concrete member, accounting for tension steel plates as equivalent reinforcing bars.

(2) Shear capacity shall be computed on the basis of shear resisting mechanisms accounting for the orientation and intervals of shear reinforcing steel plates.

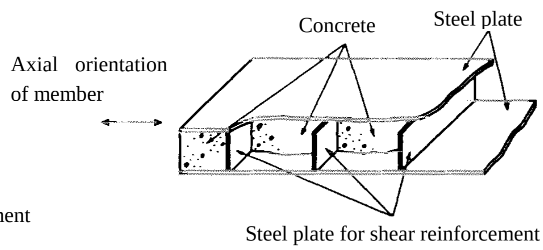
[Commentary]　The provisions for sandwich members assume that concrete fills all space between the steel plates in the sandwich member.

(1) Stiffeners of tension steel plates, and shear reinforcing steel plates oriented along the member's axis may be considered as tension reinforcing steel of the sandwich member.　Local buckling of compression steel plates and shear reinforcing steel plates oriented along the member axis should be appropriately accounted for when the ultimate capacity in compression is calculated.

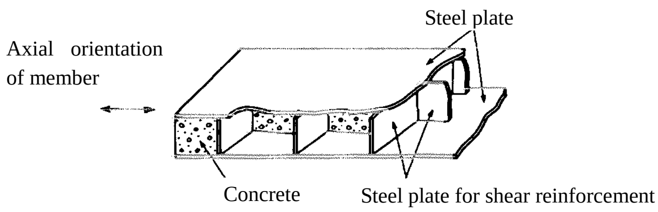
(2) Steel-concrete sandwich members resist shear forces by means of a truss-like mechanism. Since the layout and shape of tension reinforcing structural steel and shear reinforcing structural steel in sandwich members are different from conventional reinforced concrete the computation of the shear capacity should take this difference into account.　In the computation of shear capacity, both cases – failure of concrete compression diagonals, and, yielding of shear reinforcement steel plates, should be examined.　Care should be taken as different methods are required to be used depending upon the relative orientation (refer to Fig. C16.9.1) of the shear reinforcing structural steel, i.e. when the shear reinforcing steel is oriented only in the axial direction, when it is oriented perpendicular to the axial direction, and cases where it is oriented in both directions.　Examination for the ultimate capacity of the welded point should be carried out, since sandwich members tends to have many welding.



Steel plate for shear reinforcement oriented in axial direction



Steel plate for shear reinforcement oriented perpendicular to axial direction



Mix of steel plate for shear reinforcement oriented in axial direction and steel plate for shear reinforcement oriented perpendicular to axial direction

Fig. C16.9.1 Shear reinforcement concepts

As far as the torsional capacity of sandwich members is concerned, no clearly established methods are yet available, and therefore the capacity may be estimated using only the torsional moment capacity provided by the structural steel. However, the torsional moment capacity can be estimated based on the experimental results of loading test.

16.9.1.2 Examination of limit state of fatigue failure

Examination for the fatigue limit state, if required, shall be carried out in accordance with Chapter 9 of the Specification.

[Commentary] Examination should account for a possible reduction in the fatigue strength of the steel by welds or forming holes in rolled steel section, steel plates or built-up sections. Further reduction in the fatigue strength on account of dampness should also be accounted for.

16.9.2 Examination of serviceability

(1) In general, examination for the crack width need not be carried out.

(2) Examination for displacement and deformation shall be carried out in a manner similar to that for conventional reinforced concrete members.

[Commentary] In steel-concrete sandwich members, structural steel is arranged all around the concrete. As long as it can be ensured that this steel is sound, and the water-tightness and air-tightness are maintained, it may be assumed that any occurrence of cracking in the concrete will not have a large effect of the member. Thus, it is important to ensure that the surrounding structural steel does not lose its function of completely protecting the in-filled concrete on account of its interaction with the environment by corrosion. Special attention should be paid to ensure that there are no holes that are harmful to durability at locations where the steel plates are welded. Holes made the steel to facilitate erection purposes should also be sealed. Where shear connectors are provided, the occurrence of cracking is more concentrated than in conventional reinforced concrete member, and the load at which crack occurs is smaller.　This must be taken into account when computing the bending moment that produces cracking.

16.9.3　Structural details

(1) Minimum plate thickness

The minimum structural steel plate thickness used in sandwich members should be 8 mm.

(2) Vents

In order to prevent air becoming entrapped during concrete placement, air vents must be provided at appropriate locations.　The location and the size of the holes should be decided in a manner that their presence does not weaken the structure.　When rolled steel sections are used for shear reinforcement, appropriate gaps or holes should be provided, in the sections to facilitate placement and compaction of concrete.

(3) Openings in steel plates

Openings in the structural steel plates shall be as small as possible.　Appropriate reinforcement shall be provided around the openings to ensure adequate strength of the structural steel plates.　Openings made in the plates to facilitate erection should be sealed using additional steel plates and welding appropriately.

[Commentary] (1) A minimum plate thickness has been set at 8 mm to take into account weldability, etc.　If there are no problems for fabrication or design, steel plates of 6 mm thickness may be used.

(2) It should be ensured that concrete passes through the steel plates without obstruction.　In cases when a vibrator or similar equipment cannot be used, concrete having high-flowability should be used, though it should be ensured that no segregation occurs.　Use of self-compacting concrete is also recommended

Standard Methods

PART 1 STRUCTURAL ANALYSIS OF MEMBERS

CHAPTER 1 GENERAL

1.1 Scope

(1) This Specification for Design provides a method of performance verification using the linear analysis method specified in Chapter 7 of "Design: General Requirements." When making structural analysis by nonlinear analysis, this Specification may be consulted.

(2) For reinforcing bar arrangement where members or structures are designed in accordance with this Specification, it should in principle adhered to the relevant stipulations in "Design: Standard Methods Part 5."

[Commentary] (1) This Specification presents a simple structural analysis method based on the linear analysis method for the performance verification of members or structures, and describes verification methods and structural details for individual members or structures. When structural analysis is carried out using nonlinear analysis, this Specification may be consulted. The general requirement for modeling and structural analysis should be referred to Chapter 7 "Design: General Requirements.

(2) The design methods for members and structures specified in this Specification are based on the assumption that the requirements specified in "Design: Standard Methods Part 5" should be satisfied. Where using the design methods specified in this Specification, not only the details of ordinary reinforcing bar arrangement specified in "Design: Standard Methods Part 5" but also the details of reinforcing bar arrangement for related members should be observed.

CHAPTER 2 BEAMS

2.1 General

(1) Member forces in beams shall be computed by linear analysis taking into account structural systems, loading conditions, and so forth.

(2) Where member forces are obtained by linear analysis, flexural, shear and torsional stiffnesses of members may generally be computed using gross concrete section.

[Commentary] (1) Generally beams are classified by the shape of their cross sections, e.g. rectangular, T- or I- shaped, or box- beams, or by their support conditions, e.g. simply supported, continuous, fixed or cantilever beams. These beams are usually built separately and connected to each other with cross beams.

Structural analysis of beams means computation of member forces such as moments and shearing forces in a beam, by replacing the actual loads and structures with appropriate structural models. Member forces can be obtained by selecting a structural analysis method that suits loading conditions, structural systems, span-width ratio, flexural and torsional stiffnesses of beams and slabs, and the required accuracy of design computations. Structural analysis is determined corresponding to the selected analysis method.

(2) Member forces may be computed by linear analysis using gross concrete section, since it does not significantly affect the distribution of member forces in the whole structure provided the reinforcement in the sections is within a normal range. Even in a statically indeterminate structure where the member forces are dependent on member stiffnesses, the presence of steel or the decrease of stiffness due to cracking is only relative and thus, has little influence on the distribution of the member forces.

In a structure made up of different kinds of members, such as reinforced concrete columns and rolled steel beams, the effect of steel stiffness shall be accounted for in the analysis. For the structural analysis of statically indeterminate structures, the effect of deformations due to the changes of temperature, shrinkage or creep shall be taken into account in accordance with the provisions in Sections 5.2 and 5.3.

2.2 Span Length

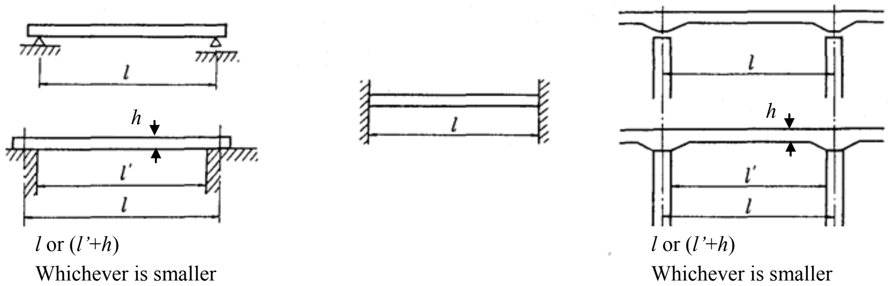
(1) The distance between centers of supports shall be taken as the span length in design of simply supported beams. In case of wide supports, the sum of the clear span and the depth of member at midspan shall be taken as the span length.

(2) For beams built monolithically with rigid walls or beams, the clear span may be taken as the span length.

(3) For continuous beams, the center-to-center distance of supports shall be taken as the span length.

[Commentary] (1), (2) and (3) The provisions in this section are made to ensure that conservative estimates are used for span lengths in the design of simply supported, fixed and continuous beams.

Span lengths to be used for simply supported, fixed and continuous beams are shown in Fig. C2.1.1 (a),(b) and (c), respectively.



(a) Simply supported beams (b) Fixed beams (c) Continuous beams

Fig. C2.1.1 Span lengths of beam

2.3 Effective Compression Flange Widths of T- Beam

(1) Effective compression flange widths of T- beams for flexure may be determined in accordance with Eq. (2.3.1) or (2.3.2).

(i) For beams with slabs on both sides of the web (see Fig.2.3.1(a)),

$$b_e = b_w + 2(b_s + \frac{l}{8}) \qquad \textbf{(2.3.1)}$$

where, b_e **shall not exceed the distance between the centers of slabs on both sides.**

(ii) For beams with a slab on one side of the web only (see Fig.2.3.1(b)),

$$b_e = b_l + b_s + \frac{l}{8} \qquad \textbf{(2.3.2)}$$

where, b_e shall not exceed the sum of b_l and 1/2 the clear span length of the slab. In the above computations, l shall be taken as the span length for a simply supported beam, the distance between points of inflection for a continuous beam, or twice the clear span length for a cantilever beam. The value of b_s shall not be greater than the depth of haunches.

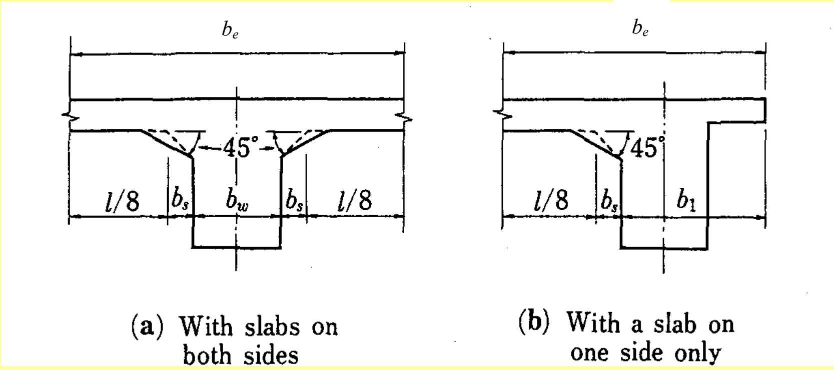


Fig. 2.3.1 Effective compression flange widths T-beams

(2) Effective compression flange widths for axial loads may be taken as overall widths of slabs.

(3) Effective compression flange widths of T-beams used for computation of the indeterminate forces may be taken as overall widths of slabs.

[Commentary] (1) When beams having compression flanges such as T- beams or box- beams are subjected to loads, the distribution of longitudinal stresses in the flanges may be taken as shown in Fig. C2.3.1. For the sake of design simplicity, the stress is computed assuming that it is uniformly distributed over a flange width of b_e, which is called the effective compression flange width.

Though the effective widths should, in principle, be determined taken into consideration loading conditions, dimensions of beams, span to flange-width ratio and support conditions, the method becomes practically inconvenient. For simplicity, this section stipulates λ shown in Fig. C2.3.1, called the effective width on one side, to be equal to $l/8$, based on the past research results relating to effective flange widths for loads similar to uniformly distributed loads. For a continuous beam, l in the equation for b_e may be taken as shown in Fig. C2.3.2. When the flange width is especially large, horizontal shear at the intersection of the flange and the web shall be examined for safety.

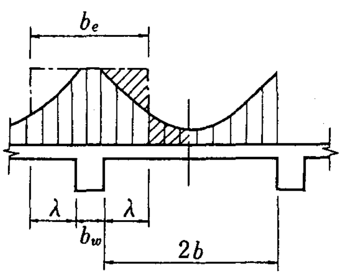


Fig. C2.3.1 Effective width

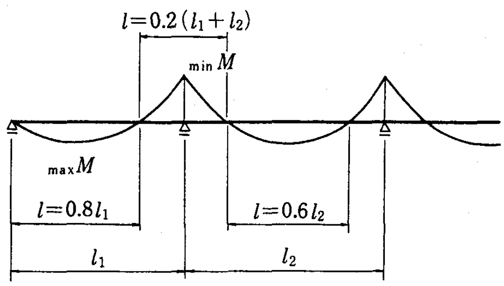


Fig. C2.3.2 Values of *l* for continuous beam

(2) On the basis of past research, this section permits considering the overall flange width as the effective width for axial loads. In the case of prestressing forces in a prestressed concrete member, the forces should be broken down into axial forces and moments due to eccentric loading, and the effective width for each used respectively.

(3) When deformation due to shrinkage or creep of concrete is restrained in a statically indeterminate structure, indeterminate forces are produced due to the restraint. For computation of such forces, this section permits overall widths to be taken as effective widths in the case of T and box type beams as past research and experience has shown adequate safety.

2.4 Isolated Beam

Isolated beams, unless analysis for lateral stability is carried out, shall comply with the following requirements (i) to (iv).

(i) Isolated rectangular beams shall be laterally supported at spacing not exceeding 15 times the width of beam.

(ii) Isolated beams shall be laterally supported at spacing not exceeding 25 times the width of web.

(iii) Isolated beams shall have a flange thickness not less than 1/2 the width of web.

(iv) Effective compression flange widths of isolated beams shall not exceed 4 times the width of web.

[Commentary] In the design of isolated beams, it is required to take into account stress due to lateral loads which may occur other than due to ordinary vertical loads. When using isolated beams with narrow width, total stability of the member shall be checked. Lateral buckling of concrete in compression may occur, if lateral spacing of isolated beams is excessively large. Provisions (i) to (iv) are so specified as to ensure safety, unless specific analysis is conducted.

2.5 Continuous Beam

As for continuous beams, the negative moments at intermediate supports, may be reduced in accordance with Eq. (2.5.1) (see Fig. 2.5.1).

$$M_d = M_{od} - rv^2/8 \qquad \textbf{(2.5.1)}$$

providing, $M_d \geq 0.9 M_{od}$

where, M_d **: reduced design moment at intermediate support**

M_{od} **: design moment at intermediate support calculated as support**

$r = R_{od} / v$

R_{od} : **design support reaction force at intermediate support**

v : **hypothetical distribution width of support reaction force along centroidal axis of member**

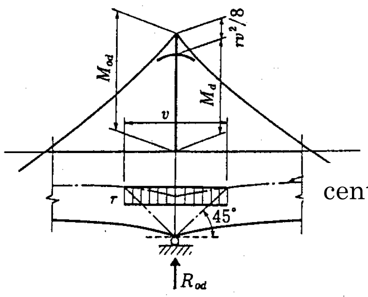


Fig. 2.5.1 Design flexural moment at intermediate support

[Commentary] This section permits the reduction of the moment as specified, since it is assured by finite element analysis and other methods that the distribution of negative moment over the support of continuous beams is graded due to the effect of the width of bearing, depth of beam, and existence of cross beams.

2.6 Structural Details

(1) Thickness of the flange in T-beams and the top and the bottom slabs in box beams shall not be less than 80 mm.

(2) Thickness of webs in T-beams and box- beams shall not be less than 100 mm.

[Commentary] For important structures such as bridges, it is recommended that a slab thickness be not less than 100 mm, and a web thickness not less than 150 mm.

2.7 Deep beam

(1) In cases when the span length to overall depth ratio of beams is less than the value indicated below, and a detailed examination of the beams is not carried out, the beams shall be designed as a deep beam in accordance with provisions of this section.

(i) simply supported beam $l/h < 2.0$

(ii) continuous beam with two spans $l/h < 2.5$

(iii) continuous beam with three or more spans $l/h < 3.0$

where, l **: span of beam**

h **: height of beam**

(2) In the examination for flexure, axial tension reinforcement may be computed in accordance with Section of 9.2.1 of "Design: General Requirement" using the maximum flexural moment obtained by the ordinary beam theory. Reinforcement located within a distance of 0.2 h **from the extreme fiber in tension zone of a cross-section shall be considered as axial tension reinforcement and shall be anchored in accordance with Section 7.2 of "Design: Standard Part 5".**

(3) Shear should, in general, be checked at the ultimate limit state of failure at the cross section, where the design shear capacity V_{dd} **may be computed using Section 9.2.2.2 of "Design: General Requirement".**

[Commentary] (1) When the depth of beam is large in comparison with the span length and thus the stress distributions of the beams are different from ordinary beams, such beams are called deep beams. Scope for beams to be treated as deep beams has been defined referring to current codes of various countries and results of past researches. Provision shows only a guideline, and need not be applied if a more detailed examination is carried out.

(2) Behaviors of deep beams are similar to those of tied arches in a way that axial tension reinforcement in deep beams operates in the same manner as tension tie members in tied arches after the formation of diagonal cracks. Therefore, failure of a deep beam occurs, either when reinforcement as tension tie members in tied arches yields (flexure failure), or when concrete as arch rib reaches compression failure (shear failure). Provision has been made from the fact that for the yielding of reinforcement the computation in accordance with Section 7.2 of "Design: Standard Part 5" based on moment obtained by ordinary beam theory also gives reasonable value.

Because axial tension reinforcement is considered to be tension tie members in tied arches, all amount of reinforcement required for maximum moment should extend beyond the support and should never be anchored in the spans.

2.8 Corbel

(1) A cantilever beam shall be considered a corbel and treated in accordance with the present section in cases when the ratio of the distance between the loaded point and the column face to the overall depth is less than 1.0 (see Fig.2.8.1), unless specific analysis is carried out.

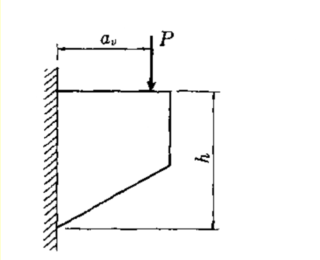


Fig. 2.8.1 Corbel

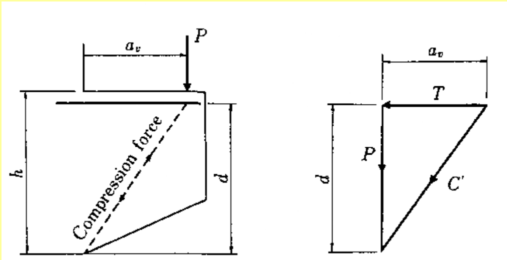


Fig. 2.8.2 Analytical model for corbel

(2) Corbels may be analyzed as a truss system comprising of a horizontal tension member and a diagonal compression member as shown in Fig. 2.8.2.

(3) In the examination for flexure, axial tension reinforcement shall be provided to resist the horizontal tensile forces calculated in Clause (2).

In cases when the axial tension reinforcement is arranged in two or more layers, the reinforcing bars shall be placed within a distance of $d/4$ from the upper surface of corbels (see Fig. 2.8.3), where d is the effective depth of corbel at the column face.

(4) In general, the check for shear should be carried out at the ultimate limit state of failure at the cross section, and the design shear capacity, V_{dd}, may be calculated using Section of 9.2.2 of "Design: General Requirement". In this equation, a_v and d shall be taken as the distance between the loaded point and the column face, and the effective depth of corbel at the column face, respectively.

(5) Effective depth of the corbel at the loaded point shall not be less than 1/2 the effective depth at the column face (see Fig. 2.8.3).

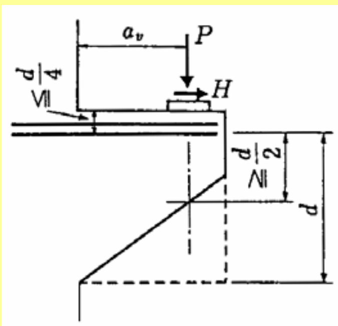


Fig. 2.8.3 Effective depth at the loading point and primary tension reinforcement in corbel

[Commentary] (1) Corbels can be considered the same as deep beams, since corbels have the similar distribution of stresses and the resisting mechanism to those in deep beams. But, considering particular characteristics and detailing in corbels, the provision specifically applied to corbels have been made separately. Geometrically corbels are defined as cantilever beams having small ratio of overhanging length from the column face to the overall depth of beam, and the provision stipulates the value of such ratio as not exceeding 1.0 for the purpose of design.

(2) From past experimental results and researches by finite element analyses, it has been permitted to assume the resisting mechanism by means of truss analogy as shown in this section.

(4) Failure of corbels is defined as, yielding of axial tension reinforcement, or the compression failure of the web concrete acting as the diagonal in compression of a truss. As the compression failure of the web concrete of corbels is similar to that in deep beams, the equation applied to deep beams also applies to the calculation of the shear capacity of corbels.

(5) (i) When the effective depth of the corbel at the loaded point is insufficient, failure as shown in Fig. C2.8.1 occurs and there is no formation of the resisting mechanism of the truss system. Therefore, the minimum effective depth at the loaded point is stipulated to prevent this kind of failures.

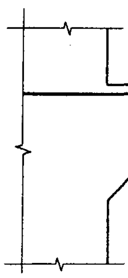


Fig. C2.8.1 Example of failures due to insufficient effective depth at the loading point

CHAPTER3 COLUMNS

3.1 General

Design of columns shall be carried out on the basis of the axial force, moment, shear force and so forth as calculated from structural analysis, taking into consideration the shape and rigidity of members, relative stiffness ratio of adjoining members, occurrence of construction joints, loading conditions, and so forth.

[Commentary] Column is a member mainly subject to axial compression force, but it is generally subject also to moment, shear force and so forth simultaneously with the axial compressive force according to the relative stiffness ratio to the members to be adjoined, the construction of joints, loading conditions, and so forth. This provision prescribes the column as a member mainly governed by the axial compressive force. And when the column is greatly affected also by moment or shear force, it shall be examined as a beam in accordance with Chapter2.

3.2 Slenderness ratio

(1) Slenderness ratio shall be taken as the ratio of the effective length to the radius of gyration of columns.

(2) Effective length of columns shall be determined according to the degree of fixity at both ends.

(i) For columns laterally supported at both ends, the length of axis line of structures shall be taken as the effective length of columns.

(ii) For columns with one end fixed and the other end free, the effective length shall be taken as 2 times the length of axis line of the structure.

(3) Gross cross-section of concrete may be used for calculating the radius of gyration.

[Commentary] When lateral displacements of both ends of a column cannot occur, the effective length of the column shall be determined according to the degree of fixation against rotation at both ends. When both ends of column are laterally confined by beams or the like in a multistory rigid frame structure or others, both ends of column may be considered as hinges and the length of axis may be used as the effective length of column for the purpose of simplifying calculations. A column with one end fixed and the other end free indicates the deformation equal to that of a column with the length of 2 times longer and both ends hinged. Thus, the effective length of column with one end fixed and the other end free shall be 2 times the length of the column.

3.3 Short Column

A column with slenderness ratio not exceeding 35 may be designed as a short column and the effect of lateral displacement may be neglected.

3.4 Long Column

A column with slenderness ratio exceeding 35 shall be designed as a long column, considering the effect of lateral displacement. Secondary moments due to lateral displacement, shall be calculated considering factors such as slenderness ratio, shape of section, type of load, restraining condition at column ends, characteristics of material, amount and arrangement of reinforcement, eccentricity due to construction error, shrinkage and creep, and so on.

[Commentary] It is necessary to design a long column based on a rational analysis of structure containing the effect of lateral displacement. The relation between stress and strain of concrete is nonlinear, so that the relation between the moment that act on each section and the deformation is nonlinear. For this reason, in general, the deformation curve of column is supposed, the method that makes repeatedly calculation until the deformation curve due to internal force suits the supposed deformation curve is adopted. The advantage of this method is to be accurate and to have a generality to be effective for various boundary conditions.

The long column composed by reinforced concrete is also designed by the method that the effect of secondary moment is approximately considered except general preceding method. When slenderness ratio is roughly no greater than 100, it can be designed by approximate formulae admitted in general.

3.5 Tied reinforced Column

The minimum lateral dimension of tied reinforced columns shall not be less than 200mm.

[Commentary] Tied reinforced column is a reinforced concrete column having longitudinal reinforcement hooped by lateral ties.

The limit for the minimum lateral dimension of a column has been provided herein because the principal columns are greatly related to the capacity of structures, and the capacity of the column is greatly affected by defects of concrete induced by various causes created during and after construction, especially when the minimum lateral dimension of the column is small.

3.6 Spiral reinforced Column

(1) Characteristic compressive strength of concrete used for spiral reinforced columns shall not be less than 20 N/mm^2.

(2) Diameter of the effective cross section, which refers to diameter of the circle drawn using the centerline of the spiral, of spiral reinforced columns shall not be less than 200 mm.

[Commentary] Spiral reinforced column is a reinforced concrete column having longitudinal reinforcement entwined by spiral bars. Column with longitudinal reinforcement bars hooped by circular ties instead of spirals may be treated as spiral reinforced column, if reinforcement is completely anchored.

(1) Spiral reinforced column can enhance the compressive strength of concrete by confining the lateral strain of concrete by its spirals and thus is used when it is necessary to support a large load with a column having a relatively small cross section. In this case, concrete with strength as large as possible is used and further reinforced with spirals. Therefore, spiral reinforced column using a small strength concrete is unable to fully achieve its features. Because of this, the minimum strength of concrete has been set forth.

(2) This provision has been prescribed for the same reason as the case of tied columns set forth in Section 3.5. Diameter of effective cross section is considered because concrete located inside the spirals works effectively against failure load of spiral reinforced column.

CHAPTER4 RIGID FRAMES

4.1 General

(1) A structure made of beams and columns or slabs and walls held together monolithically shall be analyzed as a rigid frame.

(2) Rigid frames shall be designed considering the relative stiffness of the connected members, the loading conditions, degree of fixity of the support, and so forth. The effect of the ground shall also be appropriately examined depending upon the condition.

[Commentary] (1) Besides ordinary beam and slab type rigid frames, flat slab, gantry constructions, culverts and so forth should also be analyzed as rigid frames.

4.2 Structural Analysis

(1) The axis line of a rigid frame shall coincide with the centroids of cross sections of respective members. The influence on centroidal lines due to presence of haunches may be neglected unless they are especially large.

(2) Rigid frame structural analysis shall be carried out considering the presence of rigid areas at the connections and at haunches.

(3) Rigid frame structural analysis may, in general, be carried out considering only flexural deformations. However, in cases the ratio of the thickness and the length of the member is not less than 0.3, the analysis shall be carried out taking into considerations shear deformations of the members.

(4) The moment considered for examination of cross-sections at the ends of members shall be in accordance with Fig.12.3.1. However, as far as a haunch is concerned, only the portion where the slope is gentler than 1:3, shall be considered effective, in the determination of the cross section.

(5) The shear forces for examining cross sections at the ends of members shall be the shear force at the upper and lower faces of the beam in the case of column analysis. And, it shall be the shear force at the position, which is at a distance of 1/2 the depth of the cross section at the front face of column from the front face of column in the case of beam analysis. However, as for haunch, only the portion, which has a slope gentler than 1:3, shall be considered as effective in calculation of cross section.

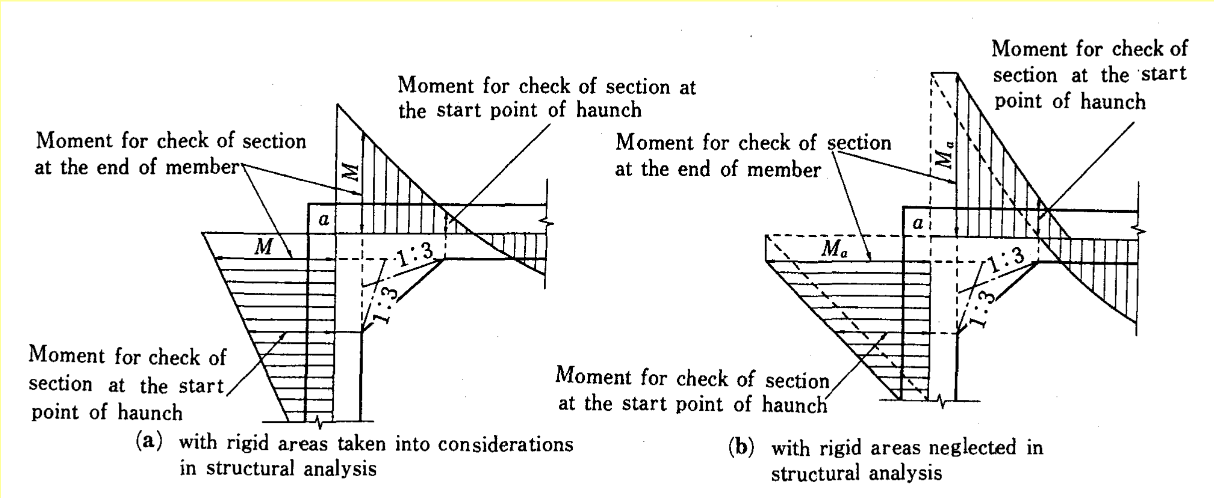


Fig. 4.2.1 Moment for check of sections at the end of member

[Commentary] (1) When the haunch is especially large, a polygonal line shall be considered as axis, and the change of cross section shall be taken into consideration.

(2) Structural analysis of a rigid frame shall be made, as a rule, taking into considerations the rigid zones by estimating the effects of connection parts of members and haunches. Rigid zones shall be generally determined by the methods from i) to iii) stated below.

i) When there is no haunch, the rigid zone shall be the inner portion starting from the cross section which is distant at 1/4 the depth of member inside from the ends of members (see Fig. C12.3.1(a)).

ii) When a member has a haunch sloped not less than 30 degrees from its axis, the rigid zone shall be the inner starting from the cross section whose depth is 1.5 times the depth of the member. However, when the slope of haunch is not less than 60 degrees, the rigid zone shall be the inner portion starting from the cross section which is distant at 1/4 the depth of member inside from the starting point of haunch (see Fig. C4.2.1(b)).

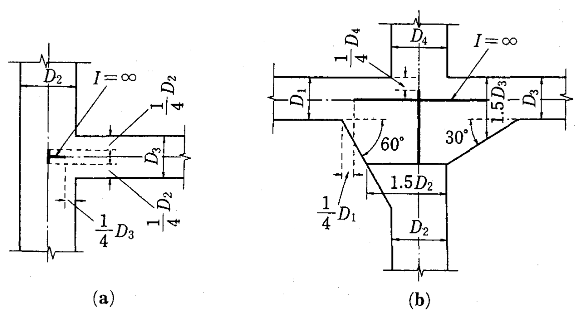


Fig. C4.2.1 Rigid area

iii) When two or more points are determined by i) and ii) above mentioned because of the difference between right and left haunches, the point that creates a larger rigid area shall be selected.

(3) In the rigid frame, only the flexural deformation may be generally considered in the structural analysis of a rigid frame. However, when the height of member is large compared with

the length of member, then the shear deformation of member shall not be neglected in comparison with flexural deformation, and so, in this case, shear deformation shall preferably be taken into considerations in the structural analysis. Generally speaking, a structural analysis shall be made considering shear deformation when the ratio of height to length (the length of axis) of member is not less than 0.3.

(4) When a structural analysis has been made considering rigid areas, then the moment to be used for checking the section at the end of member shall be as shown in Fig. 4.2.1(a). When an analysis has been made neglecting the rigid areas, then it shall be as shown in Fig. 4.2.1(b).

4.3 Structural details

In principle, haunches should be provided at the corners of rigid frames.

[Commentary] It is generally possible to cope with large moment and shear force at the corners relatively easy by increasing the height of cross sections through providing haunches at the corners, and the haunches are effective to smoothly transfer stresses at the corners. Because of this, it has been prescribed as a rule to provide haunches at corners. However, haunches sometimes cannot be provided because of structural reasons or other reasons related to appearance. In this case, it is preferable to provide large chamfers.

CHAPTER5 ARCHES

5.1 General

(1) In principle, axis of an arch should coincide with the thrust line of the loads.

(2) The shape of the cross-section of an arch rib shall be determined taking the span-to-rise ratio, axis of the arch, strength of concrete, and method of construction into consideration.

(3) The strength of the foundation of an arch rib shall be high enough to resist the reactions at the end of the arch rib.

[Commentary] (1) The thrust line of arch due to loads is the line which connects acting points of resultants in the section of arch rib. As for the shape of arch axis, normally used is quadratic parabola, hyperbolic (exponential) curve or quadratic curve. Circle, two-centered circle or multi-centered circle is often used as the shape of arch axis where the span is small, only because the forms and the temporary supports can be simplified.

(2) Sectional shape and dimensions of arch rib are determined by referring to actual examples of structures built in the past or by assuming them with approximate formulas and making repeated calculations.

Construction method sometimes becomes an important factor for determining the cross-sectional shape of arch rib, so the adequate constructing conditions must be comprehended during design stage.

5.2 Structural Analysis

(1) The line connecting the centroids of cross-sections of the arch rib shall be taken as the axis of the arch.

(2) Effects of shrinkage and temperature change of concrete shall be taken into consideration in calculating member forces.

(3) Effects of changes in the cross-section of the arch rib shall be taken into consideration in calculating statically indeterminate forces.

(4) The effect of settlement or displacement in a foundation, whenever required, shall be appropriately taken into consideration.

(5) In cases when the slenderness ratio of arch rib is not less than 35, the effect of movement of arch axis should be taken into consideration in principle in calculating

member forces.

(6) Safety of the arch-rib against in-plane and out-of-plane buckling shall be confirmed.

(7) Examination of the arch-rib in the out-of-plane direction may be carried out in accordance with Section 12.2, assuming that the arch rib is a straight column subjected to an axial force equal to the horizontal reaction acting at the end of arch rib. In this case, the length of the column should be taken as equal to span of the arch, in principle.

[Commentary] Structural analysis of arch can be made in various ways including the method in which the member forces are calculated by the frame structural analysis theory approximating the arch rib member to a polygonal line and combining it monolithically with members such as vertical members. Therefore, a linear analysis shall be applied as a rule from the viewpoint of practicality, when calculating member forces to be used in the examination of each critical state.

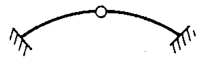
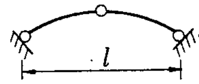
(2) This has been especially prescribed because in an arch the effects of temperature change and shrinkage of concrete are significant compared with those of beams.

(4) Foundation at the end of arch is desired to be sufficiently strong. But when there is the possibility of the foundation displacement, the effect of displacement of foundation shall be taken into consideration during design stage.

(5) When designing an arch rib, small displacement theory may be employed in calculating member forces that arch span is small or the slenderness ratio is not greater than 35. However, this deformation shall be taken into consideration as a rule, because the deformation in arch cannot be neglected in such cases that arch span is large or its slenderness ratio is not less than 35.

The slenderness ratio of arch member shall be calculated by Eq. (C5.2.1) using the equivalent length of member (l_e) of arch member calculated by using δ given in Table C5.2.1.

Table C5.2.1 Values of δ

f/l	0.10	0.15	0.20	0.24	0.30	0.35	0.40	0.45	0.50
Fixed arch 	0.360	0.375	0.396	0.422	0.453	0.495	0.544	0.596	0.648
1-hinge arch 	0.484	0.498	0.514	0.536	0.562	0.591	0.623	0.662	0.706
2-hinge arch 	0.524	0.553	0.594	0.647	0.711	0.781	0.855	0.915	1.059
3-hinge arch 	0.591	0.610	0.635	0.670	0.710	0.781	0.855	0.956	1.059

Note : f/l=Reciprocal number of span to rise ratio.

$$\lambda = l_e \cdot \sqrt{\frac{A_{1/4} \cdot \cos\theta_{1/4}}{I_m}} \qquad \text{(C12.4.1)}$$

where, λ : slenderness ratio

$l_e = \delta \cdot l_l$: equivalent length of member

$A_{1/4}$: cross-sectional area of arch rib at 1/4 point of l (m^2)

$\theta_{1/4}$: slope angle of arch axis at 1/4 point of l

I_m : average moment of inertia of arch rib section (m^4)

l_l : span length considering the degree of fixation of foundation (m)

(6) The arch rib is a member that subject to large axial compressive force due to horizontal reaction. Not only examination for stress and sectional strength of arch rib, but also safety against buckling of in and out-of-plane direction of arch rib shall be confirmed. Structural analysis for buckling of out-of-plane direction shall be made in accordance with (7).

5.3 Structural Details

In cases when an arch rib has a box type cross section, diaphragms shall be provided at the connections with support columns (see Fig.12.4.1).

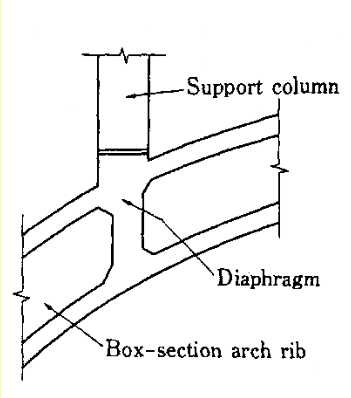


Fig. 12.4.1 Diaphragm of box-section arch rib

CHAPTER 6 DESIGN OF PLANAR MEMBERS

6.1 Scope and Definitions

Provisions of Section 12.5 shall apply to design of planar members, such as slabs, footings, shells and walls, whose width in directions perpendicular to the principal axis of the member is much larger than its height or thickness.

[Commentary] Slabs including flat slabs may be taken to be thin, plane-sided members that resist moments and shear due to transverse loads. Footings used in foundations of structures are much thicker than ordinary slabs, and have a different purpose compared to that of a slab. Shells are three dimensional spatial structures made up of one or more curved or folded plates. Walls are consisting of vertical plane.

6.2 Structural analysis of planar Members

The structural analysis shall be carried out in accordance with the provisions of Chapter 7 of "Design: General Requirement", considering the effect of (i) transverse load, (ii) in-plane load, or, (iii) a combination of transverse and in-plane loads.

6.3 Design of Slab

6.3.1 Structural analysis

(1) Flexural moment, shear force, torsional moment and support reaction in slabs should, in principle, be computed using thin-plate theory taking into account the support conditions, geometry and loading conditions. However, generally accepted approximate methods may also be used.

(2) Examination of the ultimate limit state of failure at the cross section for slabs may be carried out using a method based on plastic analysis, provided the method has been confirmed to be safe.

(3) In cases the support condition for slabs are likely to change during construction or while the structure is in-service, the slabs and their supports shall be appropriately designed for satisfactory performance under those changes.

(4) The center-to-center distance between supports shall be taken as the span length for the purpose of analysis. In case of slabs with long supports the sum of the clear span and the slab thickness at midspan shall be taken as the span length. For slabs built

monolithically with rigid walls or beams, the clear span may be taken as the span length.

(5) Loads acting on slabs shall be taken as distributed over areas of the slab in a shape similar to the loaded area, at a distance of half the slab thickness from the periphery of the loaded area. In cases when the slabs are overlaid with concrete or asphalt-concrete, the full thickness of the overlayer shall be added to the above distance. In cases of overlays with soft material, 3/4 of the thickness of the overlay shall be considered as the distribution thickness.

[Commentary] (1) In principle, the thin-plate theory may be adopted for structural analysis of slabs since the theory of bending of slabs with small deflections is generally applicable. For this analysis, the span length and the distribution width of concentrated load shall be in accordance with (4) and (5) respectively.

Reinforced concrete slabs are generally built monolithically and the properties of the section per unit length in two orthogonal directions are similar. When applying the thin-plate theory linear elastic analysis for isotropic slabs may be used on the assumptions that flexural stiffness in two orthogonal directions is equal and remains unchanged after cracking. In cases when a slab is built integrally with beams and the distances between beams in the two orthogonal directions are not equal, the flexural stiffness in the two orthogonal directions may be significantly different, it is recommended that such effects should be accounted for in the analysis.

Member forces in slabs, such as flexural moment and shear force, may be calculated using methods of numerical analysis such as the finite element method, on the basis of the thin-plate theory making appropriate assumptions such that the actual configuration, etc. are represented in the boundary conditions. Such calculations, however, tend to be complicated, and established methods may be used for approximate analysis. Available charts and tables may also be used. For slabs having a complex shape, member forces may be evaluated using experimental results obtained from testing of scale models.

(2) For slabs under serviceability limit state, the deformation remains sufficiently elastic. Therefore, linear analysis should be used considering the simplicity of design. For slabs under ultimate limit state of failure at the cross section, the redistribution of moments become more significant and the member forces calculated by linear analysis tend to differ from the actual values. In such cases, plastic or nonlinear analysis in accordance with the provisions of Chapter 7 of "Design: General Requirement" may be carried out, provided it could be assured that plastic hinges can freely rotate.

In a continuous slab, or in other similar cases, flexural moment calculated at supports using linear analysis may be increased or decreased in accordance with Section 7.3.2.1 of "Design: General Requirement". For practical purposes, for the plastic analysis in slabs, yield-line theory and strip method may be used. In both methods, the effect of torsional moments can be eliminated and there are fewer limitations in the analysis compared to linear analysis. On the other hand, it should be borne in mind that the principle of superposition cannot be used in this case and the analysis may be complicated depending on load conditions. Due consideration should be given to the above features when using plastic analysis.

(3) For instance, the support conditions for precast concrete slabs at the time of lifting during construction may be different from when they are in-service after installation. Further, a change in the support condition may occur when slabs deteriorated after being in service for a long time are repaired or strengthened.

In cases when a deteriorated beam–slab system is replaced by a new slab, it is not necessary that the new or repaired slabs are supported in a manner similar to that prior to deterioration. In such cases, in addition to the slabs, possible changes in the support conditions for the beams and/or columns shall also be considered.

(4) The provisions in this Section for the spans to be used in design of simply supported or continuous slabs and slab monolithic with walls, etc. are based on considerations that have been found to be empirically safe.

The span of a slab may be taken as the smaller of the following two distances:

(i) the distance between the centers of supports.

(ii) the sum of the clear span and the thickness of the slab at midspan.

For slabs built to be monolithic with beams or walls which may be considered as fixed, the span may be taken as the clear span between faces of the walls or beams. In cases when the slab is supported on bearings, the span shall be taken as the distance center-to-center of the bearings. In cases when the length of bearing is small, the effects due to bearing stress shall be examined.

(5) A concentrated load with rectangular contact area (measuring $t_1 \times t_2$,) acting directly on the surface of the slab, may be resolved into an equivalent rectangular uniform load acting over an area of [$(t_1 + t)$ x $(t_2 + t)$], where t is the thickness of the slab. The dimensions $(t_1 + t)$ and $(t_2 + t)$ may be taken to be the 'distribution widths' at a distance of 1/2 the slab thickness (from the edge of the concentrated load), assuming a load distribution at 45°. When an overlay, having a thickness 's', is present over the slab surface, the distribution width shall be taken as $(t_1 + 2s + t)$ and $(t_2 + 2s + t)$, respectively. In cases of overlays with soft material, the distribution width shall be taken as $(t_1 + 1.5s + t)$ and $(t_2 + 1.5s + t)$ (see Fig. C12.5.1).

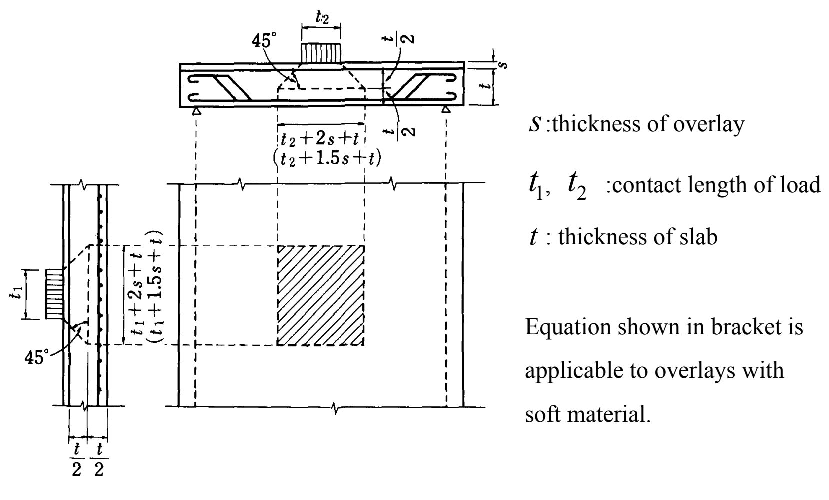


Fig. C6.3.1 Distribution width of concentrated load

6.3.2 Examination for applied member forces

(1) Examination for flexural moment

Examination for moments in slabs may be carried out as in the case for a beam, in accordance with the relevant requirements. In cases when the direction of principal moment does not coincide with the direction of reinforcement, sufficient flexural strength shall be provided in all directions.

(2) Examination for shear

Calculation of shear capacity of slabs should be carried out in accordance with the followings:

(i) The shear capacity may be calculated in accordance with the provisions for linear members assuming that a slab near a support acts as a wide beam and resists shear at the cross section in one direction.

(ii) For circular slabs subjected to uniform loading, the extra shear resistance due to circumferential reinforcement may be included.

(iii) Examination for punching shear around a concentrated load or near a support should be carried out assuming that the surface resisting shear is conical or pyramidal in shape.

(3) Examination for torsional moments

Torsional moments in slabs should be considered as equivalent shear forces at the end of the slab and as equivalent bending moments at other position of the slabs.

[Commentary] (1) Examination for flexure in slabs shall, in general, be carried out at each point for two orthogonal directions considering the slab to be a beam with unit width and accounting for the moment distribution in slabs. Even in a wide slab, where moments in one direction may be dominant, moments also arise in the transverse direction, and therefore, appropriate distribution reinforcement should also be provided in the transverse direction.

Reinforcing bars are, in general, provided in the two orthogonal directions of a slab. In most cases, reinforcing bars are provided in the direction of the principal moment at the point where the maximum moment occurs. In regions of the slab away from the area of the maximum moment and in the neighborhood of the edges of the slab, it could be subjected to unsymmetrical loads. Further, near openings the direction of principal moment differs significantly from the direction of reinforcing bars because of presence of torsional moment.

Slabs with two-way flexural and torsional moments may be designed as plates subjected to membrane forces in accordance with the provisions of Section 9.2.2.4 of "Design: General Requirement", using the method specified in 6.5(2) of the Specification, which resolves these moments into membrane effects.

When using plastic analysis for the examination of the ultimate limit state, adequate serviceability shall be ensured. Therefore, the degree of moment redistribution shall be limited as far as the result of linear analysis is applicable. Further, when assuming formation of plastic hinges, it shall be ensured that adequate plastic deformation can be guaranteed.

(2) As provided above, when designing a slab section, examination should be carried out for moments and shear forces assuming the slab to act as a beam. Further, in slabs subjected to a concentrated load, the examination for punching shear should be carried out.

However, available researches have shown that slabs such as an underground LNG tank, whose width is larger than their thickness, have greater shear resistance than that of ordinary slender beams. For example, experiments have shown that shear resistance of circular slabs is larger than that of ordinary beams because of the restraint to radial expansion provided by the reinforcing bars arranged in the circumferential direction. Experiments have also shown that the shear strength of simply supported and uniformly loaded circular slabs, whose diameter is larger than ten times the effective depth, may be computed in accordance with the provisions of Section 9.2.2.4 (commentary) of "Design: General Requirement", which has been derived from Eq.(6.3.3) and modified to account for the resistant due to circumferential reinforcements, which results in radial compressive forces.

(3) Torsional moments at other positions than the end of the slab are the elements of main bending moments and should be examined as described in (1) above.

6.3.3 Examination for slabs having different shapes

6.3.3.1 One-way slab

(1) For simply supported one-way slab subject to a concentrated load, the maximum moment per unit width may be computed as in the case of a beam with an effective width given in Eqs. (6.3.1) and (6.3.2).

(i)when $c \geq 1.2x(1-x/l)$ **(see Fig. 6.3.1(b)),**

$$b_e = v + 2.4x(1-x/l) \qquad \textbf{(6.3.1)}$$

(ii)when $c < 1.2x(1-x/l)$ **(see Fig. 6.3.2)),**

$$b_e = c + v + 2.4x(1-x/l) \qquad \textbf{(6.3.2)}$$

where c **: minimum distance between edge of the load and free edge of slab**

x **: minimum distance between the concentrated load and support**

l **: span length**

u, v **: distribution width of concentrated load**

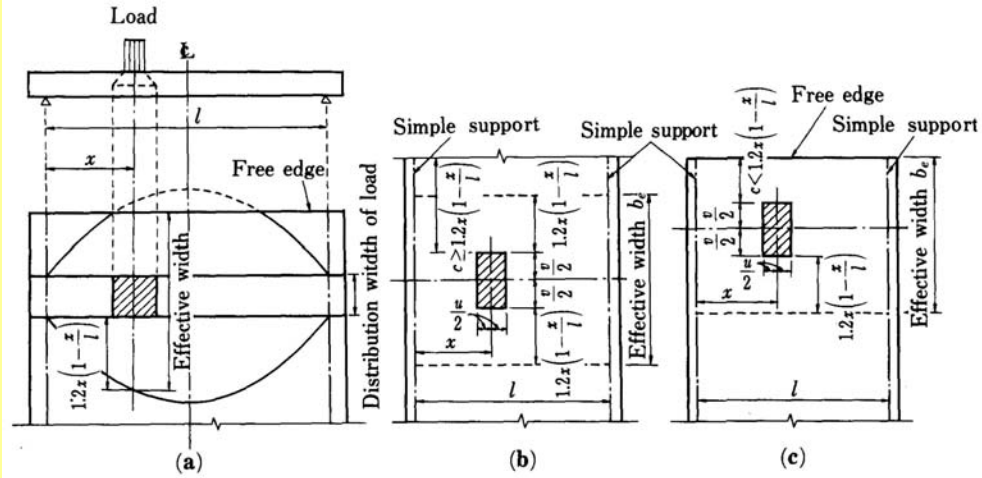


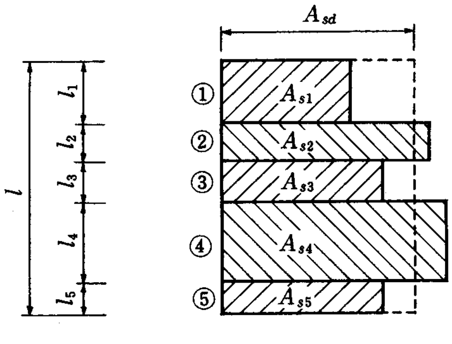
Fig. 6.3.1 Effective width of one-way slab

(2) When the design of slabs in shear is based on the method applied for beams, the effective width shall be taken in accordance with the provisions of 1).

[Commentary] (1) The method given in this section gives the approximate moment in a slab supported along two parallel sides perpendicular to the span direction due to a concentrated load. Here, the moment per unit width may be computed as in the case of a beam using the effective width (measured parallel to the edge of support) as given here (see Fig. 6.3.1).

In cases multiple concentrated loads act on a slab in a manner that their effective widths overlap, the effect of all the loads shall be adequately accounted for in the design of each region. However, in an application where the requirement of the reinforcement varies for different regions, the following simplification may be made.

In cases when the variation in the requirement for the different regions is not significant, the reinforcement per unit width, A_{sd}, may be distributed as shown in Fig.C.12.5.2 in accordance with the theory of plasticity, except in the case of skewed slabs for which a separate treatment may be required.



, $A_{sd} \times l \geqq l_1 \times A_{s1} + l_2 \times A_{s2} + l_3 \times A_{s3} + l_4 \times A_{s4} + l_5 \times A_{s5}$

$A_{s1}, A_{s2}, \cdots A_{s5}$: amount of reinforcement per unit width for each region
$l_1, l_2, \cdots l_5$: effective width for each load

Fig.C.6.3.2 Example of reinforcement provided in slabs subject to multiple concentrated loads

For continuous slabs, the following approximate method may be adopted.

(a) The maximum moment per unit width at midspan, M_c, shall be calculated as follows.

$$M_c = Km_c$$

where, $m_c = \dfrac{\text{midspan moment in a continuous slab calculated as a continuous beam}}{\text{total width of slab}}$

$K = \dfrac{\text{moment per unit width of simply supported slab with effective width}}{\text{moment per unit width of simply supported beam}}$

The span used in the calculation of K shall be taken as follows.

end span $0.8l_1$, where l_1 is the length of end span

center span $0.6l_2$, where l_2 is the length of center span

(b) The maximum moment per unit width at support, M_e, shall be calculated as follows.

$$M_e = K' m_e$$

where, $m_e = \dfrac{\text{moment at support in a continuous slab calculated as a continuous beam}}{\text{total width of slab}}$

To calculate the support moment at the first intermediate support, the coefficient K' should be taken as the average of K at the end span and at the first intermediate span. Also, for the second intermediate support, average value of K at the first and second intermediate spans should be used.

In cases when a one-way slab is fixed at opposite ends, and is subject to the action of a concentrated load, the effective width shall be computed using Eq. (C6.3.1) and (C6.3.2) for positive moment at center span and negative moment at fixed end, respectively.

$$b_e = v + x(1 - x/l) \qquad \text{(C6.3.1)}$$

$$b_e = v + 0.5x(2 - x/l) \qquad \text{(C6.3.2)}$$

6.3.3.2 Two-way slab

(1) In cases of two-way slabs, with the ratio of shorter to longer span not exceeding 0. 4, subject to uniformly distributed loads, member forces may be computed as in the case of one-way slab on the assumption that the loads are carried only along the shorter span.

(2) For two-way slabs other than in 1), member forces in slabs shall be computed using the thin-plate theory or other approximate methods.

[Commentary] (1) When two-way slabs with a ratio of shorter span to longer span l_x/l_y, not greater than 0.4 are subjected to uniformly distributed loads, the magnitude of moment in x direction is nearly equal to that calculated as a one-way slab in the short span direction. In this case, the moment may be computed as a beam with unit width of slab spanning in short span direction. It is necessary to provide distribution reinforcement for the moment in y direction accordingly.

6.3.3.3 Cantilever slab

(1) For a cantilever slab subject to a partially distributed load, the maximum moment per unit width of the slab may be computed using the effective width, b_e, as defined below.

For load applied in the center (Fig.6.3.2(a))

$$b_e = v + 2x \qquad \textbf{(6.3.3)}$$

For load applied at the edge

$$b_e = v + x \qquad \textbf{(6.3.4)}$$

where, x : distance between support and load

v : distributed width of load

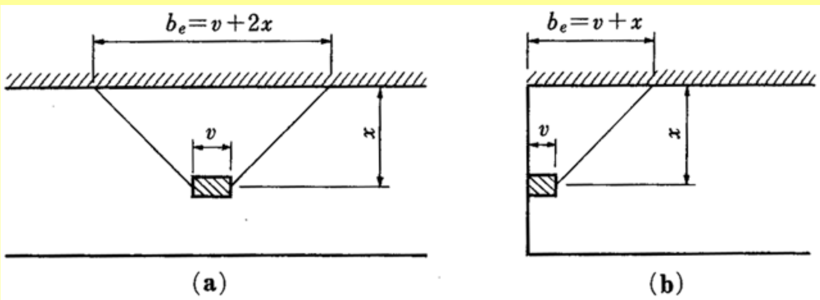


Fig. 6.3.2 Effective width of cantilever slab

(2) For a cantilever slab subject to uniformly distributed load, the maximum moment in the direction of the span may be computed as in the case of a beam, in accordance with the relevant requirements assuming the cantilever slab to act as a cantilever beam.

[Commentary] (1) When a wide cantilever slab is subject to a concentrated load P perpendicular to the slab surface at a distance x from the fixed end as shown in Fig. C6.3.3(a), the influence lines for moment M on the edge of support are given in Fig. C6.3.3(b). This Section is so specified that the effective width for the approximate calculation leads to a satisfactory margin of safety.

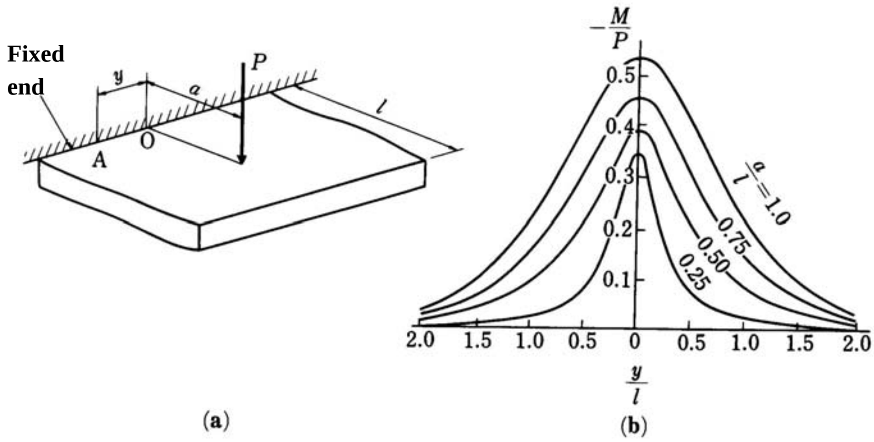


Fig. C6.3.3 Influence lines for moment at point A on fixed end

Hooks shall be provided for primary reinforcement in a cantilever slab.

Where the thickness of slab is not enough to anchor a reinforcing bar shall be bent down at the free end and extended along the bottom of slab. (Fig. C6.3.4)

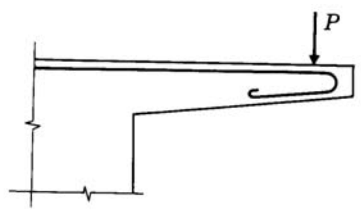


Fig. C6.3.4 Development of primary reinforcement in cantilever slab subject to large concentrated load at free end

6.3.3.4 Skewed slab

(1) For skewed slabs, member forces shall be computed taking into account the degree of the skew angle and the width of slab.

(2) For simply-supported skewed slab with the skew angle not less than 45°, member forces may be computed using the following simplified methods (i) and (ii) (see Fig. 12.5.4).

(i) For $b/l_i \leq 0.75$

Positive moment shall be evaluated as in the case of a one-way slab using the skewed span length, l_i**.**

(ii) For $b/l_i > 0.75$

Positive moment shall be evaluated as in the case of a one-way slab using the straight span length, l_n**.**

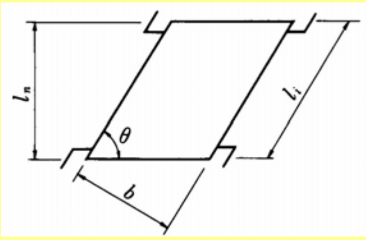


Fig. 6.3.3 Skewed slab

[Commentary] (1) For the calculation of the member forces in skewed slab, the methods shall consider the degree of the skew angle and the width of slab. For skewed slabs with skew angle less than 45° analysis by such as finite element method or grid theory shall be employed.

(2) When the value b/l_i of skewed slab is small, it is pointed out that the simplified calculation method specified in this Section leads to a rather conservative estimate of design moment assuming the skewed length as the span length. This method, however, allows the use of the skewed span length for safe estimations.

6.3.3.5 Flat slab

(1) General

Provisions of this Section shall apply to the flat slab constructions, where the slab is supported directly or through drop panels by the columns. The following definitions of technical terms in flat slab construction have been used.

Column line: Line connecting the centers of columns

Column strip: A strip having a width of $0.25l_x$ or $0.25l_y$ on either side of column line

Middle strip: A strip having a width of $0.5l_x$ or $0.5l_y$, sandwiched between column strips

where l_x : length of span measured between center to center of supports in a direction of the column lines

l_y : length of span perpendicular to the direction of l_x

(2) Structural analysis

(a) Member forces of flat slab may be calculated as substituted rigid frames which can approximate the behavior of the portions of flat slab as divided by the center lines between adjacent column lines or by the center lines of middle strips. In this case, examination shall be carried out for each of the two orthogonal directions in the plane of slab.

(b) The following approximate method may be adopted for the analysis of flat slabs reinforced in two directions. In this case, however, the structural details as given in Section 6.3.4 of "Design: Standards Part 5"shall be satisfied.

(i) For vertical loads, calculations shall be carried out in accordance with a) to c) below.

a) Slabs in flat slab construction shall be considered as made up of two groups of orthogonally intersecting beams divided by column lines in two orthogonal directions – x and y. A rigid frame made up of these beams and columns shall be considered in each direction, and calculations shall be carried out by applying complete loads in worst conditions to the frame. In the x direction of the rigid frame, the span and width of the beam shall be taken as l_x, and l_y respectively. The depth of the beam shall be taken to be t, which is the thickness of the slab. It is the same for the y direction (see Fig. 12.5. 7). In the case of multistory rigid frames, only the flexural resistance of columns directly connected to slabs being considered may be taken into account.

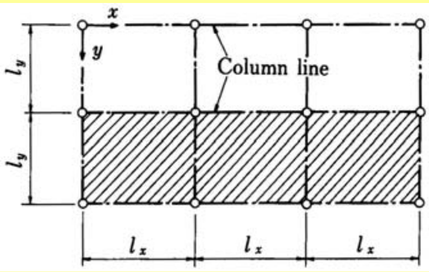


Fig. 6.3.4 Frames of flat slab

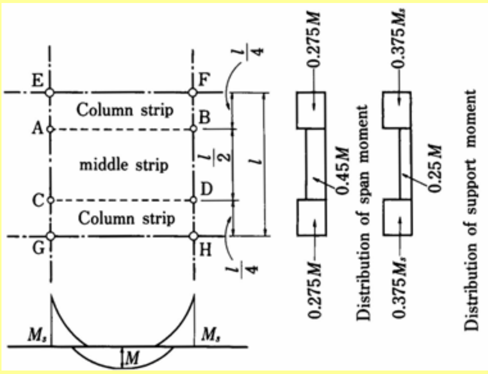


Fig. 6.3.5 Distribution of moment

b) Slabs shall be divided into a middle strip ABDC having width of l/2 and the column strips ABFE and CDHG each having width of l/4. 45 % of positive or negative span moment calculated as rigid frame shall be uniformly distributed to the middle strip,

and the remaining 55 % uniformly to the column strips on both sides. 25 % of negative support moment shall be uniformly distributed to the middle strip, and the remaining 75 % uniformly to the column strips on both sides (see Fig. 6.3.5). In case when the edges of the flat slab are supported throughout their length, 3/4 of the moment of the ordinary middle strip may be used for the portion with the width of 3l/4 from the supporting edge.

c) Columns shall be considered as vertical members of a rigid frame.

(ii) For horizontal loads, calculations shall be carried out assuming the rigid frame as considered above. However, the width, b, of the beam in the rigid frame shall be given by Eq.(6.3.5)

$$b = (l + c)/2 \tag{6.3.5}$$

where, l: span length in each direction

c: with of the column head (diameter in the case of circular column)

Moment of beam obtained shall be distributed in the proportion of 0.7 and 0.3 to the column and the middle strip, respectively.

(3) Design for the connections between slab and columns

(a) Connections between the slab and columns shall be checked against punching shear.

(b) In cases when moments are transferred between the slab and column at their connection, in addition to punching shear, the bending and torsional moments, and shear force acting at the face of the connection shall be accurately determined. The dimensions and materials of the connection shall be so determined to safety the limit state requirements against the above forces.

(4) Structural details

(a) Thickness of slab should, in general, not be less than 150mm. However, this condition may not apply to cases when the load is small, e.g. in the case of roof slabs.

Ratio of effective depth of slab to the larger span should, in general, not be less than 1/32. Even in cases when the applied loads are small, e.g. in roof slabs, the ratio should not be less than 1/40.

(b) Width of column shall be larger than 1/20 of the distance between the centers of columns in the same direction as that of the width, 1/15 of the height of story and 300mm, whichever is the largest (see Fig. 6.3.6).

(c) Dimensions of column head shall conform to Fig. 6.3.6. When calculating the stresses in the column head, only the portion above the 45° line from the horizontal line shall be considered effective.

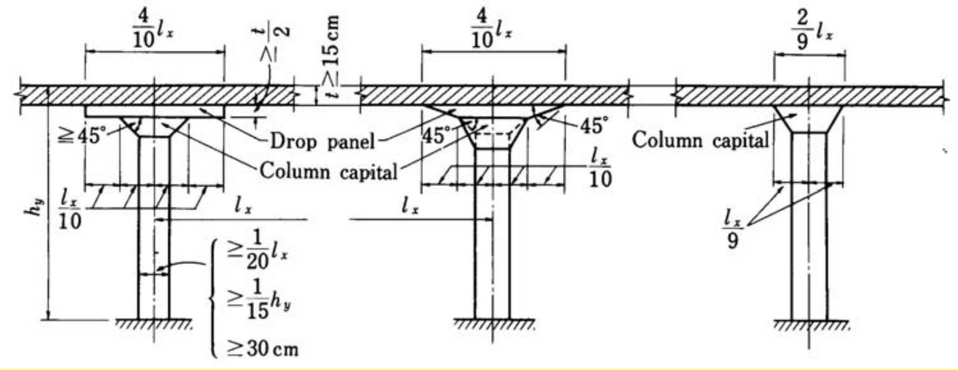


Fig. 6.3.6 Flat slab system

[Commentary] (1) When designing flat slab as a rigid frame, the rigidity of column and slab may be determined based on the rigidity of actual cross sections.

Moment of slab may be obtained by properly distributing the moment of beam, calculated from the substituted rigid frame, to column strips and middle strip. Moment may be uniformly distributed within column strips and middle strip.

(2) For the practical application the simplification has been made in the distribution method of moment specified here, based on the results from the theoretical calculation.

Edges of flat slab supported throughout their full length means that flat slab is supported by walls or beams. In this case, the moment acting on slab strip adjoined to and in parallel to the supporting edge is smaller than that of other ordinary slab strips. Therefore, 3/4 of the moment of ordinary middle strips may be taken into account for this portion.

(4) The provisions for the dimensions of each portion specified here have been prescribed taking into considerations not only the requirements as flat slabs but also the conditions for approximate analyses of them. The method specified in Fig.12.5.9 is applicable to not only y direction but also x direction.

6.3.4 Structural details

Minimum thickness of slabs shall be 80mm.

[Commentary] This Section specifies the minimum thickness of 80mm for the normal in-situ concrete slab because in a thin slab the defects due to inadequate workmanship may affect the strength of the slab. Nevertheless, in determining the slab thickness, due consideration shall be given to control the deformation of slab to be sufficiently small, not to damage the serviceability and appearance of structure and not to give adverse effect on the overlayers.

In a thin slab, it is generally difficult to arrange shear reinforcement effectively. For slab with the thickness not less than 200 mm, however, shear reinforcement may be provided to resist shear force.

6.4 Design of footing

6.4.1 General

Provisions of this Section shall apply to design of isolated footings, wall footings and combined footings, which are used as foundations of structures. The following definitions of technical terms related to footings have been used.

Isolated footing: A footing that distributes forces transmitted through a single column or a pedestal and is not combined with other footings

Wall footing: A footing that distributes forces transmitted through a wall

Combined footing: A footing that distributes forces transmitted through more than single column or pedestal, or, that connecting isolated footings with members.

6.4.2 Structural analysis

(1) Footings shall be designed taking into account not only the forces transmitted through columns, walls, etc. but also loads due to self-weight and soil surcharge (or overburden). The reaction from the ground for spread foundations, the pile reaction for pile foundations and buoyancy shall also be considered.

(2) In principle, footings should have sufficient thickness so as to behave as a rigid body.

[Commentary] (1) Footings shall be designed by taking into account the loads from superstructure, self weight of footing, surcharges due to soils, subgrade reaction, pile reaction, buoyancy and so on. The effect of the surcharge may be unsafe or safe depending on the location of the design cross section. Therefore, footings shall be designed by the possible loading conditions on safety side.

(2) The existing design has been formulated on an assumption that a footing is connected rigidly with foundations and columns or walls. This implies that footing shall have enough rigidity compared with foundations or columns and walls. Therefore, a footing should have enough depth to be handled as a rigid body. The decision whether a footing can be treated as a rigid body or not may be evaluated by its relative rigidity to soil or foundation. A footing may not need to be a rigid body, if it is analyzed precisely.

The required depth to handle a footing as a rigid body may be obtained in the following way.

(a) In case of isolated footing and combined footing, average depth of a footing shall not be less than about one fifth of the long side of the footing.

(b) In case of wall footing, average depth of a footing shall not be less than about one fifth of the remaining width of footing after deducting the thickness of wall.

When it may be on the unsafe side to handle the footing of pile foundation as a rigid body, the footing shall be analyzed by taking into consideration its elastic behavior.

6.4.3 Examination for flexural moment

(1) Computation of design flexural moment and critical cross section

(i) Isolated footing and wall footing

a) Design flexural moment in a footing shall be computed assuming it to be a cantilever. The critical cross section shall be taken as the vertical plane at the face of rectangular column or wall or at a location one tenth of the diameter inside of the circular column.

b) Flexural moment at the critical cross section shall be computed as the moment generated by all forces acting over the entire area of footing at the face of column or wall.

(ii) Combined footing

a) The cantilever portion of a combined footing shall be treated in a manner similar to isolated footings. The portions between columns in a combined footing shall be treated as a rigid monolithic frame.

(2) Effective width at critical cross section

Effective width at the critical cross-section of footing for flexural moment shall be obtained using Eq. (6.4.1)

For spread foundation: $b_e = b_0 + 2d \leq B$ **(12.5.8)**

In the case of pile foundations, it shall be appropriately determined considering the pile arrangement and configuration of the footing.

where, b_e **: effective width**

B **: total width of footing**

b_0 **: width of column or pedestal**

d **: effective depth of member for flexure**

(3) Design capacity of member at critical cross section

The design capacity of the member at the critical cross-section shall be calculated in accordance with the method described in Chapter 9 of "Design: General Requirement".

[Commentary] (1) Flexural moment of isolated footing and wall footing may be computed by considering the critical cross section as a support of cantilever.

When the cross section of a column is neither square nor rectangular, it shall be substituted by square or rectangular having the same area and center as the original one for determining the critical cross section. For circular column, the critical cross section may be determined to be the vertical plane at one tenth of diameter inside of it (see Fig. C6.4.1(b)).

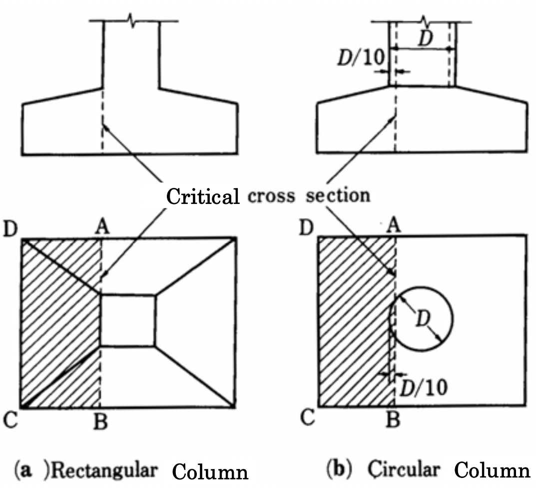


Fig. C6.4.1 Critical cross section for flexural moment

Flexural moment at the critical cross section AB as shown in Fig. C6.4.1 shall be computed as the flexural moment by all the forces acting on the entire area of ABCD.

In a footing with two-way reinforcement arrangement, although the forces acting over corners are counted twice for the cross sections at right angle to each other, they shall not be reduced.

The footing in between columns should be, as a rule, designed as a monolithic rigid frame consisting of column, beam, footing and foundation structure.

(2) Effective width of footing for flexural moment is determined by assuming 45 degree distribution of forces from the edge of columns or walls.

This effective width shall be replaced by Eq. (C6.4.1) when the tensile pile reaction makes the upper axial reinforcement.

$$b_e = b_0 + d \leq B \tag{C6.4.1}$$

6.4.4 Examination for shear

(1) Critical cross section and effective width

(i) Near the columns or walls, the critical cross section shall be at section A-A which is $h/2$ **away from face of column or wall (see Fig. 6.4.1)**

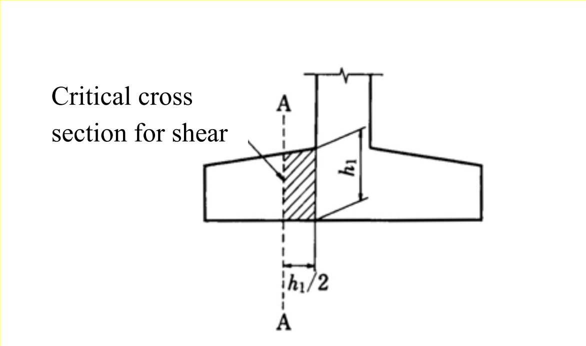
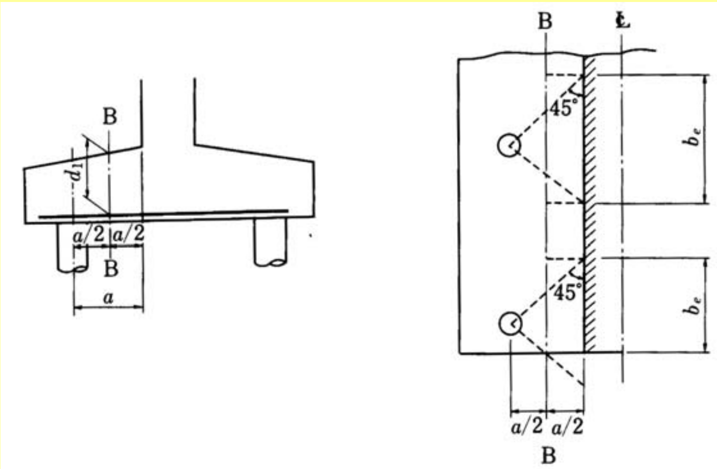


Fig. 6.4.1 Critical cross section of footing for shear adjacent to column or wall

(ii) In cases when a pile foundation has a single row of piles on one side and the distance between the center of piles and the face of column or wall is less than twice the height of the footing at the face of column or wall, the critical cross section shall be taken as the section B-B, which is at a distance $a/2$ **from the face of column or wall (see Fig.6.4.2).**



where, a **: distance between center of piles and face of column or wall (**$a \leqq 2h$**)**
b_e**: effective width**

Fig.6.4.2 Critical cross section for shear and effective width of pile foundation with single row of pile on one side

(iii) Effective width b_e **of footing shall be the entire width of the footing at the critical cross section. However, for wall footings of pile foundations with a single row of piles on one side, the effective width,** b_e**, per pile shall not exceed two times the distance between the center of pile(s) and the face of wall.**

(2) Design shear capacity at critical cross section

Design shear capacity at critical cross section shall be computed in accordance with the provisions of Section 9.2.2.4 of "Design: General Requirement."

[Commentary] (2) Based on experiments, the effective width for shear design is provided in an ordinary footing, where the ratio of distance between the face of column or wall and the center of pile to the effective depth of member at the critical cross section is, in many cases, less than 2.0. However, when the width of column is relatively small compared with the width of footing, the effective width should be determined in a different way.

In general, the behavior of footing is similar to that of deep beams and it may be sometimes difficult to arrange vertical stirrups because of spatial restriction. Attention should be paid to arrangement of vertical stirrups because they may in some case initiate diagonal cracking.

In case where the ratio of the distance a between the center of ground reaction and overburden load and the face of column or wall to effective depth d is less than 2.0($a/d < 2.0$) in spread foundations where footing is directly supported, verification may be made using design compressive shear capacity of deep beam V_{dd} as the design limit value. In the case where $a/d \geq 2.0$, verification should be made using design shear capacity V_{yd} of linear member as the design limit value. As the effective width of wall footing for shear force, total width of wall footing may be used. However, where the width of column is relatively small compared with the width of footing, the effective width should be determined in a different way. The case where the footing is not directly supported referred to the case where the top face of footing is subjected to tensile stress (see Fig. C6.4.2).

In the case where a pile foundation has several rows of piles, verification for shear force shall be carried out at each pile head. In general, A-A cross section h1/2 away from the outside of the row of piles may be used as the critical cross section (see Fig. C6.4.3).

In the case where a pile foundation has a single row of piles on one side and the distance between the center of pile and the face of column or wall is less than twice the height of the footing at the face of column or wall, the design punching shear capacity should be calculated for each pile and the combined total of shear capacity should be used as the design limit value. As the punching shear capacity per pile, the design compressive shear capacity V_{fdd} calculated using Eq. (C6.4.2) may be used (see Fig. C6.4.4). The distance to the edge of the footing, however, should exceed $b_e/2$. If the effective width exceeds the total width of the footing, the width of the footing should be used as the effective width. In the case where multiple piles have overlapping effective widths, the effective width should not exceed 1/2 of the distance to the adjacent pile.

$$V_{fdd} = (\beta_d + \beta_w)\ \beta_p \cdot \beta_a \cdot f_{fdd} \cdot b_e \cdot d_1 / \gamma_b \qquad \text{(C6.4.2)}$$

where, V_{fdd}: Design compressive shear capacity (N)

$f_{fdd} = 0.19\sqrt{f'_{cd}}$ (N/mm^2)

$\beta_d = \sqrt[4]{1000 / d_1}$ where $\beta_d > 1.5$, β_d is taken as 1.5

$\beta_p = \dfrac{1+\sqrt{100 p_v}}{2}$ where $\beta_p > 1.5$, β_p is taken as 1.5

$\beta_a = 5/[1 + \{1.1(a_1 - R/2)/d_1\}^2]$

$\beta_w = 4.2\{\sqrt[3]{100\,p_w} \cdot \{1.1(a_1 + R/2)/d_1 - 0.75\}/\sqrt{f'_{cd}}\}$ where $\beta_w < 0$, β_p is taken as 0

b_e : Effective width $(=0.8\,a_1 + R)$ (mm)

d_1 : Effective height at the face of the column or the wall (mm)

a_1 : Distance from the face of the column or wall to the center of the pile

R : Pile diameter (mm)

p_v : Ratio of tension reinforcement $p_v = A_s/(b_e \cdot d_1)$

A_s : Cross sectional area of tension reinforcement (mm^2)

p_w : Ratio of shear reinforcement $p_w = A_w/(b_1 \cdot s_1)$

where $p_w < 0.002$, p_w is taken as 0.002

A_1 : Cross sectional area per vertical stirrup (mm2)

b_1 : Spacing of vertical stirrups perpendicular to the member axis (mm)

s_1 : Spacing of vertical stirrups in the direction of the member axis (mm)

f'_{cd} : Design compressive strength of concrete (N/mm^2)

γ_b : Member factor, for which 1.2 may generally be taken

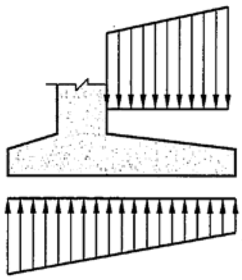


Fig. C6.4.3 Critical cross section for shear force near pile head

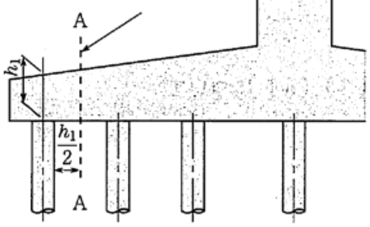
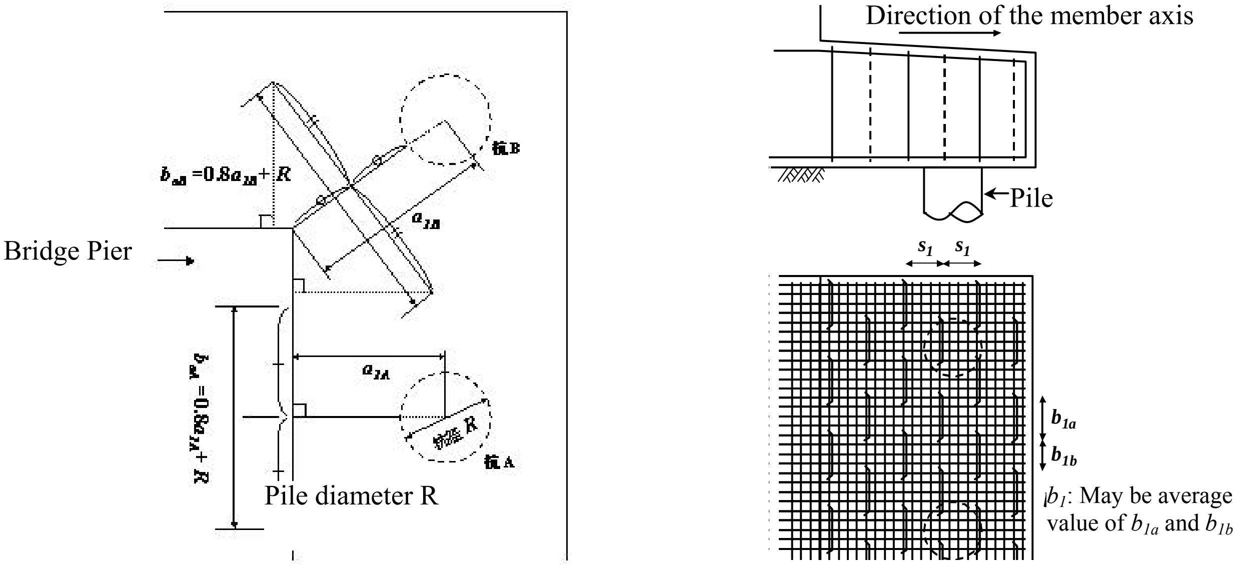


Fig. C6.4.2 Case where the footing is not directly supported



(a)Effective width b_e (b) Spacing of stirrup

Fig. C6.4.4 Verification of punching shear of pile

6.4.5 Examination for punching shear

(1) Examination for punching shear shall be carried out in accordance with the provisions for planer members. However, for pile foundations with a single row of piles on one side, the examination of punching shear may not be necessary in cases when the distance between the center of the piles and the face of the column or wall is less than twice of the effective depth of footing at the face of column or wall.

(2) When the distance between centers of piles is small, examination for punching shear shall be carried out by calculating the resistant length as a pile group.

[Commentary] In footings free edge is sometimes very close to piles. This is quite different from the case of punching shear for ordinary slabs because of small resistant area. Therefore punching shear of footings shall be examined. However, for pile foundation with single row of piles on one side, when the ratio of distance between the face of column or wall and the center of pile to the effective depth of member at critical cross section is less than 2.0, the relevant experiments show that the shear failure plane of footing passes the equally divided plane between the outer edge of pile and the face of column or wall and the failure mode is similar to that of punching shear. However, the examination for punching shear may be eliminated, because experiments show that the calculated shear capacity for beam with the effective depth specified 12.5.7.3(2) well describes the failure loads.

Depending on pile arrangements, due attention should be paid to cases which require examination for punching shear by column in isolated footing.

6.4.6 Examination for pull-out shear

(1) In cases when a footing is supported by piles and pullout forces act through anchors, the examination for pull-out shear shall be carried out in a manner similar to that for punching shear.

(2) When the distance between centers of piles is small, the critical cross section shall be determined by taking into consideration the angle of diagonal crack.

(3) When shear reinforcement is provided, its effect may be appropriately considered in the computation of the pull-out shear capacity.

[Commentary] (1) When the design for pull-out shear is conducted in accordance with the case for punching shear, the peripheral length u of loading face shall be computed in accordance with the provisions of Section 9.2.2.3 of "Design: General Requirement".

(2) When the distance l between the edge of loading face and the center of the piles is less than the effective depth d, the critical cross section shall be, in general, at 1/2 of distance between the edge of loading face and the center of the piles.

(3) When pull - out force is applied to footing, the mechanism of failure is clearly similar to the case of punching shear. Some experimental results with shear reinforcements show them to be similar to the cases of directly transmitted shear. However, there are not many experimental data for cases with shear reinforcement and formula has not yet been established to-date. Therefore, when the effect of shear reinforcement is confirmed by an experiment or other method, pull-out shear strength may include this effect.

6.5 Design of Shell and Wall

(1) In general, structural analysis of shells and walls shall be carried out in accordance with i) to iii).

i) In shell design the selection of curve shapes shall consider the mechanical characteristics of the shells. However, analysis or structural models may be simplified as far as the three-dimensional elastic behavior of shell structure can be properly approximated.

ii) Shear walls subject to in–plane horizontal loads may be designed as frame structures consisting of columns, beams and walls as a web.

iii) In general, linear analysis may be used to evaluate member forces.

(2) When shells are subject to flexural and torsional moments and axial forces due to transverse and in-plane loads, they may be examined as planar members subject to in-plane loads or beams subject to flexural moment and axial force by resolving moments into in-plane loads (axial forces) in accordance with Section 6.3.6 and 6.2 respectively.

(3) The examination for shells in shear shall be similar to that of beams.

(4) In general, structural details of shells and walls shall be determined in accordance with i) to iii)

i) End members supporting shell should have larger rigidity than that of the shell, and also the plane subject to concentrated loads should have supporting members.

ii) The effective width of shells acting together with supporting or end members shall be taken in accordance with the provisions of Section 2.3

iii) Thickness of a shear wall shall not be less than 100mm, nor less than 1/25 of the length of unsupported side of wall, and also not less than 1/30 of the shorter side in the c.

[Commentary] (1) i) Shells treated in this Section shall resist member forces due to both in-plane and transverse loads and shall have a structural shape to which the theory of shell can be applied. The folded plate forms a curve if it is divided into small elements. In such a case it may be considered as a shell.

The design of large shell structures shall include the effects of reduction in rigidity due to cracks and of the progress of deformation due to creep as well as buckling.

In the analysis of shell structures, member forces and displacement shall be calculated in such a manner that the force equilibrium as well as the compatibility are satisfied even after deformation. However, only limited cases can evaluate solutions strictly satisfying these conditions theoretically. In practice the simplification may be allowed as far as the behavior of shell structure can be properly approximated. For instance, membrane theory may be applied to the ordinary portion of the shells, while local bending effect should be taken into account in the vicinity of boundary.

<u>(1) ii)</u> Wall is a vertical plate with horizontal length not less than 4 times its thickness and it is used as members from (a) to (d) as explained below according to stress conditions (loading conditions).

(a) Walls subject to vertical loads: These members have properties similar to those of columns.

Walls subject to vertical loads may be designed as columns with a rectangular cross section as shown in Fig. C12.5.10 in accordance with Section 12.2.

(b) Walls subject to loads normal to the plane: These members have properties similar to those of slabs, such as retaining walls. Since the placing direction of concrete is different from that of slabs, their structural details governed by construction methods and so forth are different from those of slabs.

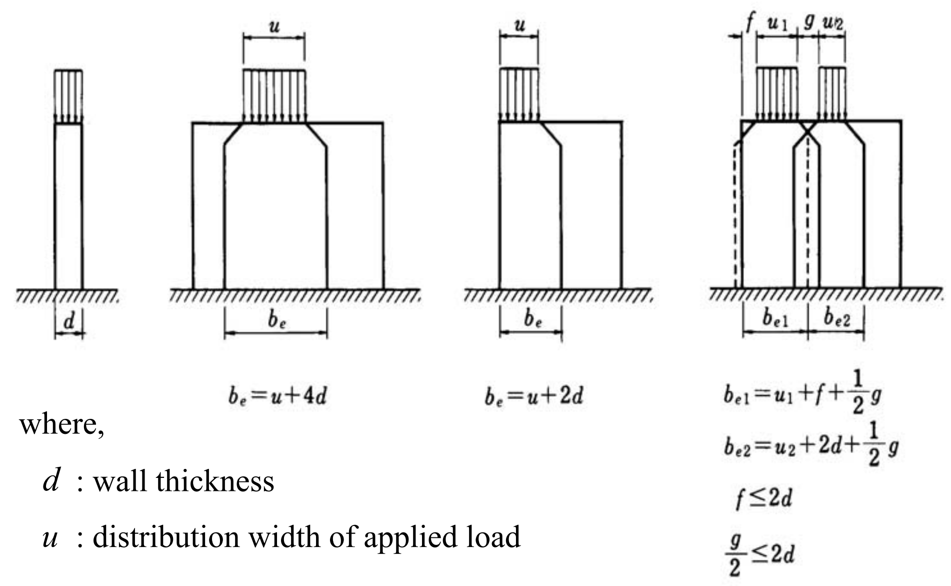


Fig. C12.5.10 Walls subject to vertical loads

(c) Walls subject to bending in plane: When walls not uniformly supported at bottom but fixed only at both ends, are loaded at its span, they are subject to bending in plane. These walls have the properties similar to those of beams, such as deep beams.

(d) Shear walls: These walls support loads mainly through the shear resistance of walls, such as earthquake resisting walls. This Section specifies the design of the shear wall as shown in Fig. C12.5.11.

If shear cracks and so forth occur in the wall, the reduction in rigidity shall be examined. In addition, the calculation of deflection or other deformations shall consider not only effects of moment but also shear force.

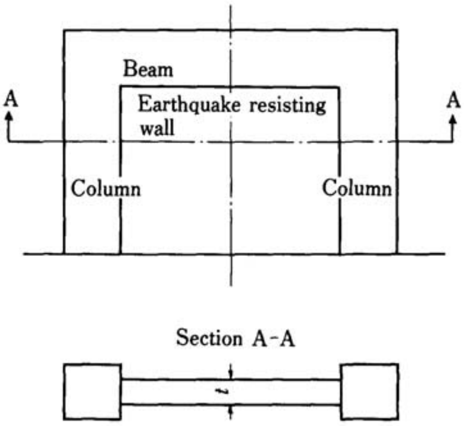


Fig. C12.5.11 Shear wall

(2) When shells are subject to membrane forces (N_x, N_y, N_{xy}),flexural moments (M_x, M_y) torsional moment(M_{xy}) and out-plane shear (V_x ,V_y)as shown in Fig. C.6.5.3, they may be modeled as comprising three layers, two outer layers and one central zone (Fig.C.6.5.4). Fig.C6.5.5 illustrates that flexural and torsional moments may be resolved into statically equivalent membrane forces. It also shows the lever arm z (distance between centroid of compression force and that of tensile reinforcements) taken as the distance between the upper and lower reinforcements. This simplification is, in general, conservative in resolving moments into membrane forces. Shells subject to combined loadings can then be replaced as for planar members subject to in-plane loads only and the ultimate limit state of failure at cross section for them is verified according to Section 9.2.2.4 of "Design: General Requirement". This design method is applicable to cases when N_1 is tension in principal membrane forces ($N_1 \geqq N_2$)and reinforcements should be less than 50% of balanced ratio at any section of shells.

When orthogonally reinforced concrete plates are subject to axial forces and in-plane shear, these membrane forces are resolved into one set of principal forces N_1 and N_2 at angle α to the reinforcement direction as shown in Fig. C.9.2.15 of "Design: General Requirement". As far as neither the yielding of reinforcement nor compression failure of concrete of shells occurs, so called “Seismic Performance 1” specified in the Chapter 11 of "Design: General Requirement" is satisfied. Shells subject to membrane forces shall follow Section 9.2.2.4 of "Design: General Requirement" in calculation of design membrane forces and capacity respectively.

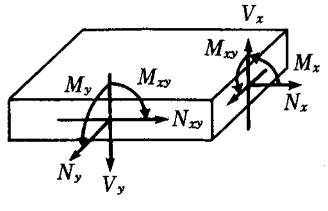


Fig. C12.5.12 Applied forces and moments in shells

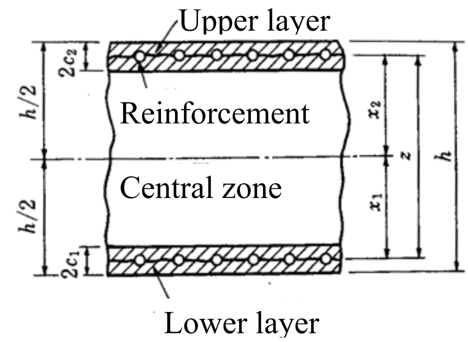


Fig. C12.5.13 Upper and lower layer model of shells

$$N_x^T = \frac{N_x}{2} - \frac{M_x}{z} \qquad N_x^B = \frac{N_x}{2} - \frac{M_x}{z}$$

$$N_y^T = \frac{N_y}{2} - \frac{M_y}{z} \qquad N_y^B = \frac{N_y}{2} - \frac{M_y}{z}$$

$$N_{xy}^T = \frac{N_{xy}}{2} + \frac{M_{xy}}{z} \qquad N_{xy}^B = \frac{N_{xy}}{2} + \frac{M_{xy}}{z}$$

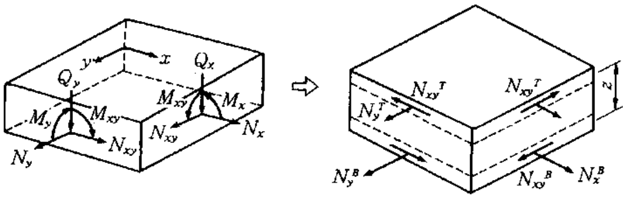


Fig. C12.5.14 Moments resolved into equivalent membrane forces

(4) i) Shells are thin in comparison with other types of concrete structures. Therefore, if shells such as domes are subject to concentrated loads, they should be supported by more rigid members, in order to prevent excessive deformation and to assure smooth flow of stresses.

(4) ii) When the end member or supporting member carry flexural stresses, shells on one side or both sides of them may also resist the flexural stresses. The effective width of shell utilized in calculations is specified here. This width may be the same as the effective width of slab as T-beam flange.

(4) iii) The minimum wall thickness is specified. Construction work may be difficult and the rigidity against flexural buckling or others may decrease, if the wall thickness is small compared with the dimensions of the plan. Generally, the thickness should not be less than 150mm.

PART 2　SEISMIC DESIGN

CHAPTER 1　GENERAL

1.1　Scope

This Specification for Design presents a method for satisfying Chapter 11 of "Design: General Requirements."　This Specification is applicable to the seismic design of reinforced concrete structures that satisfy the following conditions.

(i) Structures in which the first mode of vibration is predominant

(ii) Structures with limited major plasticity areas

[Commentary]　“Design: General Requirements” describes a flow from setting waveforms of time history accelerations on the surface of the engineering bedrock to calculating the response by conducting coupled time history response analysis using an analysis model incorporating the structure and the soil, or conducting time history response analysis in which the time history acceleration at a designated depth obtained by response analysis of soils is input to an analysis model of a structure on land, and checking by comparing the response value of the member with the limit.

In this Specification, the results of implementation of the above verification flow using the models of soil and structure that have already been developed are presented as spectra at the designated seismic coefficient at the yield point.　Then, the Specification describes a design method based on the seismic coefficient method using the designated seismic coefficient at the yield point, and a design method using nonlinear spectra, as seismic design methods for the soil and structure similar to their models.

This Specification describes a design method that meets the requirements for seismic performance verification described in “Design: General Requirements” in a limited scope of application.　This Specification is applicable to structures in which the first mode of vibration is predominant and with limited major plasticity areas.　For the structures to which this Specification is inapplicable, the seismic performance verification should be made in accordance with “Design: General Requirements.”

Even beyond the scope of application, the design method described in this Specification may be used for designing the cross section or making assumptions for reinforcement arrangement for performance check stipulated in “Design: General Requirements.”

1.2　Seismic Performance That Should Be Made Available

This Specification stipulates that the following seismic performance should be made available.

(i) Seismic performance 1 should be made available against level 1 ground motions.

(ii) Seismic performance 2 should be made available against level 2 ground motions.

[Commentary] For the design methods described in this Specification, the relationship between the design ground motion and seismic performance has been specified as described above. In cases where other type of relationship between the design ground motion and seismic performance than that specified here is specified, check should be made in accordance with "Design: General Requirements." In this Specification, the limit seismic performance is defined as described below.

Seismic performance 1: Yield strength and yield displacement of the member

Seismic performance 2: Shear carrying capacity and ultimate displacement of the member

CHAPTER 2 DESIGN BY SEISMIC COEFFICIENT METHOD

2.1 General

This chapter is applicable to seismic design using the seismic coefficient method.

[Commentary] This chapter describes a standard seismic design using the seismic coefficient method using the required yield point seismic coefficient spectra. A flowchart of the seismic design method using the required yield point seismic coefficient spectra is shown in Fig. C2.1.1.

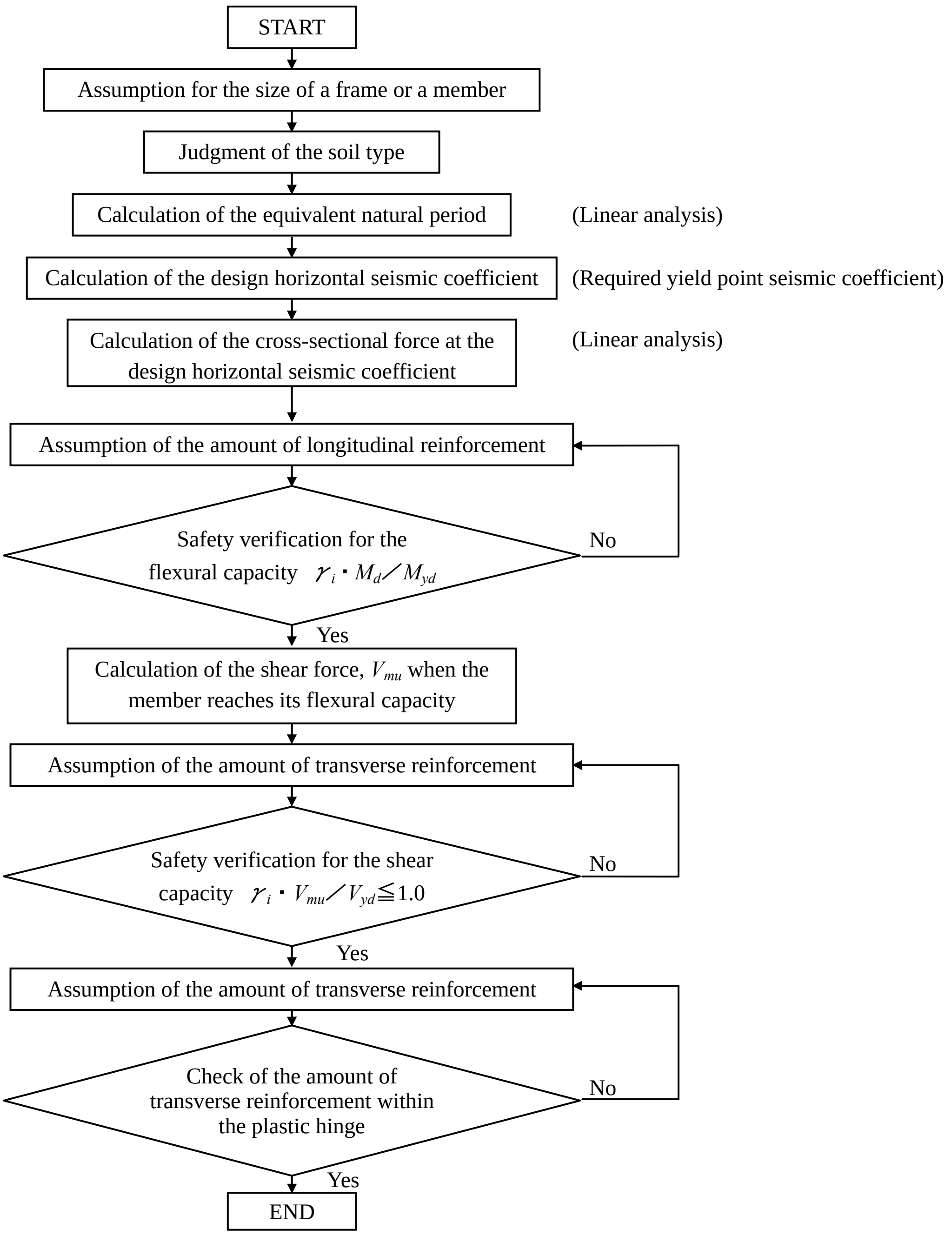


Fig. C2.1.1 Flow of seismic design using the required yield point seismic coefficient spectrum

2.2 Loads

2.2.1 General

(1) Seismic effects on structures should be considered under permanent, variable and accidental loads. For permanent and variable loads used for seismic performance verification, refer to "Design: General Requirements."

(2) As seismic effects, inertia forces of the mass of the structure and of the imposed mass should be considered.

(3) The inertia forces generated by an earthquake may be obtained by multiplying the mass of the structure and the imposed mass by the design horizontal seismic coefficient.

(4) Seismic ground motions in two orthogonal horizontal directions should generally be considered independently.

(5) The design horizontal seismic coefficient should be determined based on the required yield point seismic coefficient spectra according to the soil type and the equivalent natural period of the structure.

(6) The required yield point seismic coefficient spectra should be developed considering the vibration characteristics of the surface soil at the construction site, and the equivalent natural period and deformability of the structure.

[Commentary] (6) Numerous structures are modeled while fixing the flexural-shear capacity ratio (V_{yd}/V_{mn}) and other parameters, and the deformability of the structure is verified based on the response obtained by the time history response analysis under level 2 ground motions. Based on the results of the verification, the designated seismic coefficient at the yield point should be determined so as to meet the seismic performance requirements and the requirements against level-1 ground motions. A schematic diagram of the required yield point seismic coefficient spectrum is given in Fig. C2.2.1.

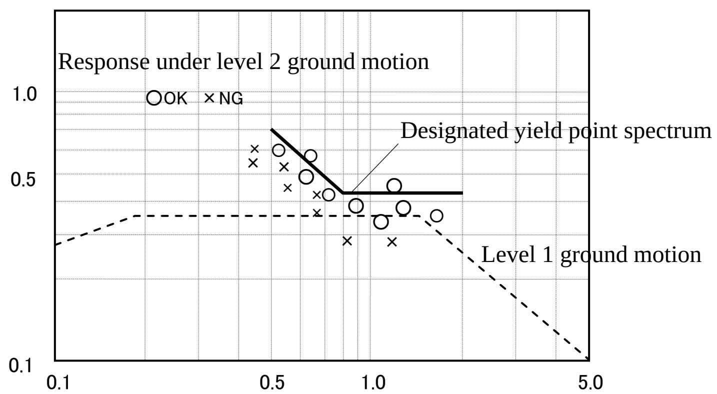
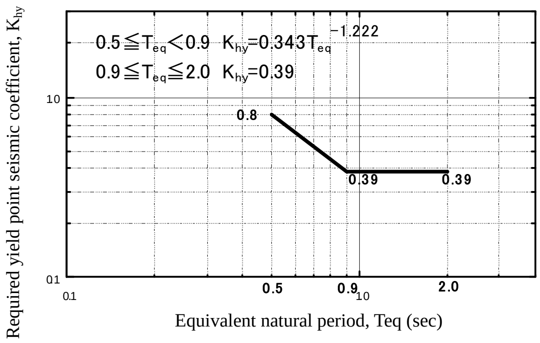


Fig. C2.2.1 Schematic diagram of the required yield point seismic coefficient spectrum

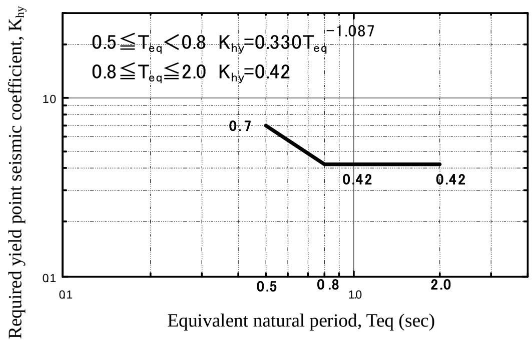
Examples of the required yield point seismic coefficient spectrum are shown in Fig. C2.2.2. These are obtained for the structure on soils except liquefied or irregular grounds in which the flexural-shear capacity ratio (V_{yd}/V_{mn}) of the member at the plastic hinge set as 2.0 where the column in a structure was damaged mainly. The seismic ground motions shown in Figs. C6.4.2 to

C6.4.4 of "Design: General Requirements" were used in the calculation.　Rigid-frame viaducts and bridge piers with a firmly fixed spread foundation or pile foundation with underground beams were examined.　The equivalent natural period of the study structure T_{eq} ranged from 0.5 sec to 2.0 sec.

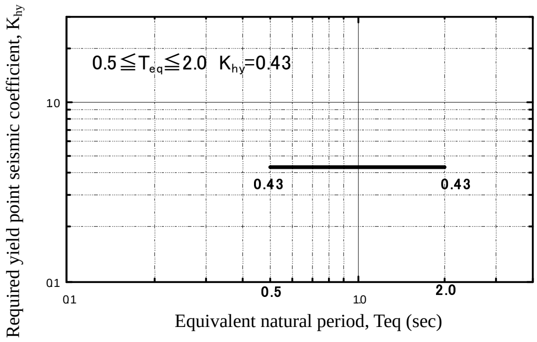
In cases where spiral reinforcing bars are arranged inside the longitudinal reinforcing bars in the cross section at the member that became plastic, the required yield point seismic coefficient spectrum should be developed in accordance with the provisions in this section.



Natural period of subsurface ground (~0.25sec, rock)



Natural period of subsurface ground (0.25~0.5sec, normal ground)



Natural period of subsurface ground (0.5sec~, soft ground)

Fig. C2.2.2 Examples of required yield point seismic coefficient spectrum

2.2.2　Calculation of equivalent natural period

Equivalent natural period should be calculated using the stiffness of the structure obtained by a static linear analysis for the structure considering the nonlinear characteristics of the member and the soil.

[Commentary]　Equivalent natural period may be calculated using equation (Eq. C2.2.1).

$$T_{eq} = 2.0\sqrt{\frac{W}{K}} \qquad \text{(C2.2.1)}$$

where,　T_{eq}　: Equivalent natural period of the structure (sec)

W　: Equivalent weight of the structure (kN)

Bridge pier: $W = W_u + 0.3W_p$

Rigid-frame viaduct: $W = W_u + 0.4W_p$

W_u : Weight of the superstructure. Self weight in the portion subjected to main vibration and an imposed weight may generally be specified. (kN)

W_p : Weight of the portion of the structure above the soil surface considered in the seismic design except W_u (kN)

K : Secant stiffness at the yield point of the structure. $K = \frac{P}{\delta}$ (kN/m)

P : Unit lateral force (kN)

δ : Lateral displacement for the lateral force P (m).

2.3 Calculation of Response Value

2.3.1 General

When calculating the response value by the seismic coefficient method using the required yield point seismic coefficient spectrum, the design cross-sectional force may be calculated from static linear analysis using structural analysis models for structural members and soil which are modeled by linear elements and a spring, respectively.

[Commentary] This section presents a general method for calculating response values by the seismic coefficient method using the required yield point seismic coefficient spectrum. The method presented in this section should be adopted in the case where it can be assumed that only the inertia force of a structure causes seismic effects on the structure. Therefore, in cases where no effects of soil displacement due to an earthquake can be ignored, the verification using the response displacement method in which the soil displacement is subjected to the structure is additionally required.

2.3.2 Modeling of structure

(1) In cases where the structure is of a continuous shape, the entire system of the continuous structure should be modeled. In cases where individual structures have nearly the similar natural period, mode of failure and horizontal load carrying capacity, the structures may be modeled separately by dividing the system according to the design vibration unit.

(2) The structure and soil should be integrally combined into a model. In cases where the plasticity of the foundation or soil has small effects, however, the foundation may be replaced with a spring for modeling.

(3) For the stiffness of members and soil, the stiffness of the gross cross section may be reduced considering the nonlinear characteristics at the yielding of the structure according to the study items.

[Commentary] "Design: General Requirements" stipulate that response values should be calculated using an analysis model that takes the nonlinear characteristics of materials into consideration. The objective is to trace the process in which members become plastic based on the time history. The structural analysis described in this section is aimed at calculating the displacement and cross section force at the yield point of the structure. Members are assumed to behave beyond the yield point like the members of the structure used as a model when developing the required yield point seismic coefficient spectrum on condition that adequate transverse reinforcement is arranged. Based on this assumption, linear analysis is applied for structural analysis. Herein, the members suffer the reduction of stiffness due to the effects of cracking at the yield point. The stiffness of members should therefore be evaluated properly. When using the required yield point seismic coefficient spectra shown in Fig. C2.2.2, the stiffness reduction coefficient may be specified as shown below for members. For bridge piers or viaducts, stiffness is reduced only for members in which major nonlinear characteristics are considered. For other members, the stiffness of the gross cross section is considered.

Bridge piers: 0.3

Rigid-frame structures: 0.5

2.3.3 Calculation of design response value

For the design cross-sectional force for structures, the cross-sectional force obtained from the static linear analysis under the inertia equivalent to the design lateral seismic coefficient should be adopted.

[Commentary] For the design cross-sectional force for structures, the cross-sectional force that can be obtained in structural analysis in which the inertia obtained based on the design lateral seismic coefficient is applied should be used. The cross-sectional force obtained here satisfies the yield strength of the structure and serves as a goal when designing members for which plasticization is considered. As for members in which no plasticization is considered, the nonlinearity of the entire structure for which the required yield point seismic coefficient was assumed should be ensured. In addition, the cross-sectional force which is properly incremented according to the yield strength or flexural strength of the member that becomes plastic should be used for design in accordance with Section 2.4.

2.4 Seismic Performance Verification

2.4.1 General

The design cross-sectional force of each member should be used as the response value in the seismic performance verification.

[Commentary] In the verification of seismic performance of structures by the seismic coefficient method, the yield strength of structures and the shear carrying capacity of members should be checked using the design cross-sectional force, and the deformability of the members should be ensured.

2.4.2 Examination for bending moment

(1) Safety of the member in which plasticization is allowed against bending moment should be verified whether or not the design flexural yield capacity, M_{yd} satisfies the conditions expressed by equation (2.4.1) against the design bending moment at the design horizontal seismic coefficient, M_d.

$$\gamma_i \cdot M_d / M_{yd} \leq 1.0 \qquad \textbf{(2.4.1)}$$

where, M_d **: Design bending moment**
M_{yd} **: Design flexural yield capacity**
γ_i **: Structural factor, which may be set at 1.0.**

(2) Safety of the member for which plasticization is not allowed against the design bending moment should be verified using equation (2.4.1) where the design bending moment at an actual yield point seismic coefficient and the restorability of the members in which plasticization is allowed is considered.

[Commentary] (2) In order to prevent the occurrence of multiple plastic hinges, as a prerequisite for applying the required yield point seismic coefficient spectrum, this provision shall be used for the member for which no plasticization is allowed. In this provision, the cross-sectional force of the member at the actual yielding shall be used for simplicity. The seismic coefficient at the yield point of the member in which plasticization is allowed cannot be fully identical to the design horizontal seismic coefficient because of the arrangement of reinforcement but exceeds the design horizontal seismic coefficient. In this Specification, the horizontal seismic coefficient when the member in which plasticization is allowed actually yields is referred to as the actual yield point seismic coefficient.

This Specification assumes that if the variance between the design horizontal seismic coefficient and the actual yield point seismic coefficient is small in design, the cross-sectional force is nearly proportional to the seismic coefficient, and stipulates that the actual yield point seismic coefficient may be calculated using equation (C2.4.1).

The actual yield capacity of the member in which plasticization is allowed should be calculated using the actual yield strength of the reinforcement considering all the longitudinal reinforcement arranged in the cross section as followed in Section 2.4.3 (1).

(Actual yield point seismic coefficient) = (Actual yield capacity of the member for which plasticization is allowed)/(Cross-sectional force at the design horizontal seismic coefficient) × (Design horizontal seismic coefficient) (C2.4.1)

2.4.3 Examination for shear force

(1) Safety of the member in which plasticization is allowed against shear forces should be verified whether or not the design shear carrying capacity, V_{yd} satisfies the condition expressed by Eq. (2.4.2) against the shear forces in each cross section of the member when the flexural capacity, M_u in the member is reached.

$$\gamma_i \cdot V_{mu} / V_{yd} \leq 1.0 \qquad \textbf{(2.4.2)}$$

where, V_{mu} **: Shear force in each cross section of the member when the flexural**

capacity in the member is reached. $V_{mu} = M_u / L_a$

M_u **: Flexural capacity which is calculated considering all the longitudinal reinforcement arranged in the cross section.　As the nominal value of the tensile yield strength of steel, a material modification factor ρ_μ of 1.2 multiplied by the JIS requirement for lower limit of tensile yield strength may be used.**

L_a **: Shear span.　A half of the member length for rigid-frame viaducts.　As for bridge piers, the height from the bottom of the column to the point in the superstructure that is subjected to the inertia.**

V_{yd} **: Design shear carrying capacity which is calculated in accordance with "Design: General Requirements."**

(2) Safety of the member in which plasticization is not allowed against shear forces should be verified whether or not the design shear carrying capacity, V_{yd} **satisfies the condition expressed by Eq. (2.4.3) against the design shear force calculated considering the actual yield strength of reinforcement and the effect of structural characteristics of the member in which plasticization is allowed.**

$$\gamma_i \cdot V_d / V_{yd} \leq 1.0 \qquad \textbf{(2.4.3)}$$

where, V_d **: Design shear force that is calculated considering the actual yield strength of reinforcement and the effects of structural characteristics.**

V_{yd} **: Design shear carrying capacity.**

γ_i **: Structural factor which may be set at 1.0.**

[Commentary]　(1) The member in which plasticization is allowed responds as the deformation progresses after yielding through the displacement at the maximum flexural capacity, and eventually the ultimate displacement is reached.　Therefore, as the design shear force that is assumed the maximum shear acting on the member, the shear forces in each cross section of the member when the flexural capacity in the member is reached is used for the verification.　Herein, since the calculating the maximum shear acting on the member is required, the actual yield strength of reinforcement to calculate the flexural yield capacity should be used.　According a proven procedure, the JIS requirement for lower limit tensile yield strength multiplied by a material modification factor of 1.2 as long as JIS-conforming reinforcement is used as the actual yield strength of reinforcement.

(2) Since both the difference of the actual yield strength to the nominal strength of reinforcement and the structural characteristics affect on the design shear force, a rate of the increases in the design shear force at the design horizontal seismic coefficient should be determined by a preliminary examination with detailed information.　The rate of the increase in the design shear force should generally be set as around 1.5 for a RC beam in single-layer rigid-frame structures and as around 2 for piles.　For the piles with the reduction of the axial force, the design shear carrying capacity should be calculated considering with the twice of the reduction of the axial force.

2.4.4 Examination for the amount of hoops in portion that becomes plastic

The amount of hoops should be determined so that the prerequisites for developing the required yield point seismic coefficient spectrum should be satisfied.

[Commentary] In order to provide the member that becomes plastic with the deformability that is assumed when developing the designated seismic coefficient spectrum at the yield point, hoops should be arranged in the area where members become plastic if the designated seismic coefficient spectrum at the yield point shown in Fig. C2.2.2 is used.

$$V_{yd} / V_{mu} \geq 2.0 \qquad \text{(C2.4.4)}$$

where, V_{mu} : Shear force in each member cross section of the member when the flexural capacity in the member is reached. $V_{mu} = M_u / L_a$

M_u : Flexural capacity which is calculated considering all the longitudinal reinforcement arranged in the cross section. As the nominal value of the tensile yield strength of steel, a material modification factor ρ_μ of 1.2 multiplied by the JIS requirement for lower limit of tensile yield strength may be used.

L_a : Shear span. A half of the member length for rigid-frame viaducts. As for bridge piers, the height from the bottom of the column to the point in the superstructure that is subjected to the inertia.

V_{yd} : Design shear capacity which is calculated in accordance with "Design: General Requirements." For column members, the design shear carrying capacity should be calculated with the consideration of the axial force at the design horizontal seismic coefficient.

CHAPTER 3　DESIGN USING NONLINEAR SPECTRA

3.1　Scope

This chapter describes provisions for seismic design using nonlinear spectra.

[Commentary]　This chapter presents a standard method of seismic design using nonlinear spectra. A flowchart of steps of the seismic design method using nonlinear spectra is shown in Fig. C3.1.1.

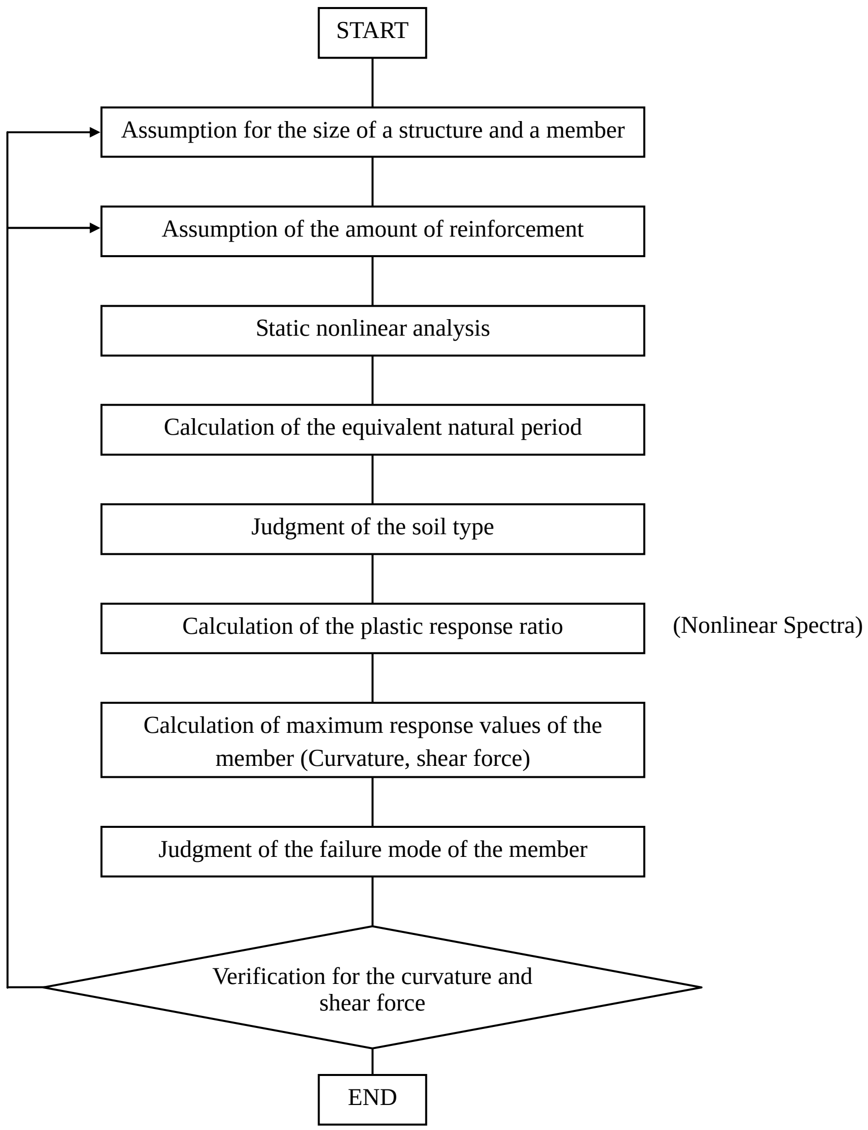


Fig. C3.1.1　Steps of the seismic design method using nonlinear spectra

3.2 Loads

(1) Seismic effects on structures should be considered under permanent, variable and accidental loads. For permanent and variable loads used for seismic performance verification, refer to "Design: General Requirements."

(2) Seismic ground motions in two orthogonal horizontal directions should generally be considered independently.

(3) As seismic effects, inertia forces of the mass of the structure and of the imposed mass should be considered.

[Commentary] (1) Used as the design loads for the seismic performance verification are permanent, variable and accidental loads. The variable loads should be treated as subordinate variable loads and combined with the seismic effects as the accidental loads. The live loads among the variable loads may be given an appropriate value in the range from zero to the expected value of the maximum during the design service life. The value should be set properly considering the fact that the response value depends on the change in stiffness of the structure due to the thermal effect or other factors and also considering the relationship between the seismic effects and the interval of recurrence.

(2) The linear spectra shown in this Specification are for the seismic response of structures of which two-dimensional models can be developed. The seismic effects should therefore be considered independently in two orthogonal horizontal directions.

(3) Earthquakes have such effects as inertia, soil displacement, earth pressure and hydrodynamic pressures during an earthquake, and lateral flow due to soil liquefaction. This Specification presents seismic design methods in cases where only the inertia is taken into consideration. Other seismic effects should be taken into consideration separately, if any.

3.3 Calculation of Response Values

3.3.1 General

When calculating response values using nonlinear spectra, the response values of structure may be calculated from the load-displacement curves obtained by static nonlinear analysis using structural analysis models for members and soil which are modeled by linear elements and a spring, respectively

[Commentary] This section shows a general method for calculating response values using nonlinear spectra in cases where only the inertia of the structure can be assumed to be the effect of an earthquake. Therefore, in cases where no effects of soil displacement due to an earthquake can be ignored, the verification using the response displacement method in which the soil displacement is subjected to the structure is additionally required. Since the yield strength of reinforcement is generally higher than that of the JIS requirement for lower limit, higher shear force acts on the structure than when the nominal strength is used in the calculation. Response values for the seismic performance verification should therefore be calculated both in cases where the JIS requirement for lower limit of tensile yield strength and the actual yield strength are used for the calculation.

In cases where actual yield strength is unknown, the lower limit strength of JIS requirement multiplied by a material modification factor of 1.2 may be used.

3.3.2 Modeling of structure

(1) Structures should be divided into units that vibrate independently for modeling purposes. If structures continuously in place have nearly the same period, failure mode and lateral load carrying capacity, they may be divided into independently vibrating units and modeled individually.

(2) The structure and soil should be considered as one unit, and be modeled using linear elements and a spring, respectively. If the plasticization of the foundation or soil has small effects, the foundation may be replaced with a support spring when developing a model.

(3) For members and the soil, nonlinear effects should be taken into consideration. For considering the effects of nonlinearity of members, the methods shown in "Design: General Requirements" Section 7.2.2.2 may be used. The effects of the nonlinearity of the soil should be taken into consideration using an appropriate method.

[Commentary] (1) Nonlinear spectra are generally developed focusing on the dynamic response of a single structure. Structures should be divided into independently vibrating units to meet the prerequisites. Bridge piers or viaducts that are located continuously may be divided into appropriate design independent vibrating units in the longitudinal and transverse directions if the structures have nearly the same period, failure mode and lateral load carrying capacity. In cases where dividing into the independent vibrating units is difficult, the verification should be made in accordance with “Design: General Requirements”.

(2) Superstructure and foundation behave as one unit during an earthquake. Superstructure and foundation may therefore be treated as one unit and may be modeled using linear elements and a spring, respectively. In cases where the soil or foundation has small effect on superstructure, the superstructure and foundation may be modeled separately. The effects of the foundation may be modeled using the support spring of the superstructure in view of the interaction.

3.3.3 Calculation of equivalent natural period

The equivalent natural period should be calculated using the secant stiffness joining the origin to the yield point of the entire structure on the load-displacement curve that is obtained in the static nonlinear analysis of the structure considering the nonlinearity of the member and soil.

[Commentary] The equivalent natural period should be calculated by the following equation using the yield stiffness joining the origin to the yield point of the entire structure on the load-displacement curve that is obtained in the static nonlinear analysis of the structure.

$$T_{eq} = 2.0\sqrt{\frac{W}{K}} \qquad \text{(C3.3.1)}$$

where, T_{eq} : Equivalent natural period of the structure (sec)

W : Equivalent weight of the structure (kN)

Bridge piers: $W = W_u + 0.3W_p$

W_u : Weight of the superstructure supported by the bridge pier (kN). The weight of the beam or an imposed weight may generally be used.

W_p : Weight of the portion of bridge pier above soil surface considered in the seismic design (kN)

Rigid-frame viaducts: $W = W_u + 0.4W_p$

W_u : Weight of the superstructure of the rigid-frame structure (kN). The weights of the upper layer beam, deck slab, and an imposed weight may generally be specified.

W_p : Weight of the portion of the viaduct above soil surface and below the bottom surface of the upper layer beam considered in the seismic design (e.g. underground beams, middle-layer beams and columns) (kN).

K : Secant stiffness at the yield point of the structure. (kN/m)

$$K = \frac{R}{\delta} \qquad \text{(C3.3.2)}$$

R : Lateral load when the yield point is reached for the entire structure (kN).

δ : Lateral displacement when the yield point is reached for the entire structure (m)

3.3.4 Calculation of design response values

(1) The required yield point seismic coefficient spectra that are developed for the plasticization condition of the structure, according to the type of design seismic ground motion, structural type, material properties and soil type should be used for calculating the design response value using nonlinear spectra.

(2) When calculating the design response value of the structure using nonlinear spectra, the response ductility factor should be calculated from the required yield point seismic coefficient spectrum using the equivalent natural period and yield seismic coefficient that can be obtained in static nonlinear analysis while regarding the inertia as an incremental load.

(3) The maximum response displacement of the structure should be calculated using the response ductility factor. In addition, the angle of rotation of each member and the response value for the cross sectional force at the maximum response displacement of the structure should be calculated.

[Commentary] Shown below is a general procedure for calculating the response ductility factor of the structure using the required yield point seismic coefficient spectra.

(i) Obtain the yield point seismic coefficient, K_{hy} in static nonlinear analysis of the entire structural system.
Defined as the yield point seismic coefficient for the structure, K_{hy} is the seismic coefficient when the structural member initially yields.

(ii) Obtain the equivalent natural period, T_{eq} in accordance with Eq. C3.3.3 Calculation of Equivalent Natural Period.
Natural period T_{eq} should be calculated by equation (Eq. C3.3.3) using the lateral displacement δ_y / K_{hy} at the yield point seismic coefficient of the structure.

$$T_{eq} = 2.0 \times (\delta_y / K_{hy})^{1/2} \qquad \text{(C3.3.3)}$$

(iii) Obtain the intersection of (i) and (ii) above using the designated seismic coefficient spectrum at the yield point and identify the response ductility factor.
The response ductility factor μ_r identified should be used to calculate the maximum response displacement δ_r of the structure by equation (C3.3.4).

$$\delta_r = \delta_y \times \mu_r \qquad \text{(C3.3.4)}$$

(iv) Calculate the angle of rotation and shear of each member at the maximum response displacement of the structure.

An example is shown in Fig. C3.3.1.

The designated seismic coefficient spectrum at the yield point can also be used to estimate the strength that should be provided to the structure (designated seismic coefficient at the yield point) in cases where equivalent natural period T_{eq} and design ductility factor δ r are known. The intersection of curves of equivalent natural period and ductility factor represented by the value along the horizontal axis indicates the designated seismic coefficient at the yield point.

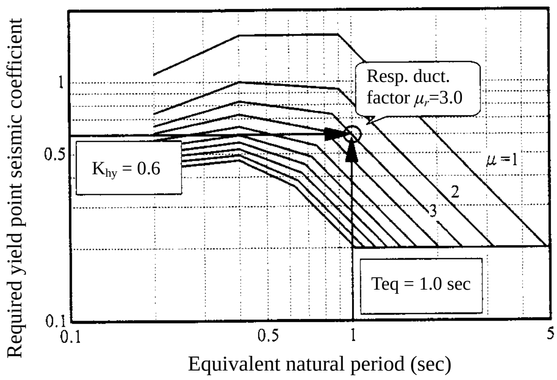


Fig. C3.3.1 Method for obtaining the response ductility factor, μ_r from equivalent natural period, T_{eq} and seismic coefficient at the yield point, K_{hy}

3.4 Seismic Performance Verification

(1) Seismic performance should be verified regarding the failure mode of the member using the angle of rotation and shear of each element as response values.

(2) The failure mode of the member should be checked as shown below.

$V_{mu} / V_{yd} \leqq 1.0$: **Flexural failure mode**

$V_{mu} / V_{yd} > 1.0$: **Shear failure mode**

where, V_{mu} **: Shear force when flexural strength develops in the member.** $V_{mu} = M_u / L_a$

M_u : **Flexural load carrying capacity of the member. Calculated considering all the longitudinal reinforcement arranged in the cross section. As the strength of steel, the characteristic value multiplied by the material modification factor ρ_m should be used.**

L_a : **Shear span.**

V_{yd} : **Design shear carrying capacity of the member.**

(3) The seismic performance of the structure should be verified that each element and member satisfy equation (3.4.1).

$$\gamma_i \cdot S_d \;/\; R_d \leqq 1.0 \qquad \textbf{(3.4.1)}$$

where, S_d : **Design response value.**

Angle of rotation and shear force of each element at the maximum response displacement obtained from the nonlinear spectra.

R_d : **Design limit value.**

Design shear carrying capacity and ultimate angle of rotation or ultimate displacement of the member.

Herein, the design shear carrying capacity of the member should be specified for the minimum cross-sectional area of shear reinforcement at out of the plastic hinge.

γ_i : **Structural factor which may be set at 1.0.**

[Commentary] (1) Members should be verified according to the failure mode of the members because the response of and the damage in the members vary according to the failure mode.

For the member to which the flexural failure mode is considered to be applicable, the angle of rotation of the member in the form of a plastic hinge should be verified. As for the member to which the shear failure mode is considered to be applicable, shear carrying capacity should be verified. When checking either of the parameters, limit values should be specified. Here, the member to which the shear failure mode is considered to be applicable rapidly loses its horizontal resistance when flexural yielding occurs. The verification should therefore be made to determine whether flexural yielding occurs or not.

(2) The failure mode of a RC member should be identified by comparing the shear carrying capacity of the member with the maximum shear occurring in the member when the flexural capacity is reached. Then, the maximum shear occurring in the member should be obtained from actual flexural capacity. For calculating the flexural capacity, the material strength should be used where all the longitudinal reinforcing bars are considered and the actual yield strength of the tensile steel is taken into consideration. In cases where SD295, SD345 or SD390 reinforcing bars are used as tensile reinforcement, a material modification factor ρ_m of 1.2 multiplied by the lower limit strength of the JIS requirement may be used as the nominal value of tensile yield strength.

In simple structures, V_{mu} can be obtained direct from the design flexural capacity by properly assuming the shear span. In cases where assuming the shear span is difficult such as in pile members or statically indeterminate structures, no shear failure occurring due to a shear at the point where the load carrying capacity of the structure reaches the maximum seismic coefficient may be determined as a failure mode using equation (Eq. C3.4.1). Then, if equation (Eq. C3.4.1) is satisfied, the verification may be made as a member to which flexural failure is applicable.

$$\gamma_i \cdot V_d \ / V_{yd} \ > \ 1.0 \qquad \text{(C3.4.1)}$$

where, V_{yd} : Design shear

V_d : Design shear carrying capacity.

γ_i : Structural factor which may be set at 1.0.

The design shear carrying capacity should be calculated using the minimum cross-sectional area of shear reinforcement in the member. In calculation, the effects of a shear span to depth ratio should be considered in accordance with "Design: General Requirements."

(3) When specifying the limit of the angle of rotation of the member in accordance with Section 7.2.2.2 of "Design: General Requirements," the limit may be specified using equation (Eq. C3.4.2). As the longitudinal reinforcement used for calculating the limit of the angle of rotation, the hoop located in the plastic hinge zone or closed stirrup should be used. In the plastic hinge zone, adequate reinforcement should be arranged so as to meet the requirements specified in this section and based on the Explanation in Section 2.4.4.

$$\theta_n = \theta_m + \eta\{1 - (M_n \ / \ M_m)\} \qquad \text{(C3.4.2)}$$

where, θ_n : Ultimate angle of rotation. Specified in compliance with Section 7.2.2.2 of "Design: General Requirements."

η : Factor to consider the softening gradient of the member which may generally be set at 1.0.

θ_m : Angle of rotation at the maximum strength. Specified in accordance with Section 7.2.2.2 of "Design: General Requirements."

M_n : Bending moment at the ultimate angle of rotation. Yield bending moment may be specified.

M_m : Bending moment at the maximum load carrying capacity.

PART 3 DURABILITY DESIGN

CHAPTER 1 GENERAL

1.1 Scope

This Specification presents a method for ordinary concrete structures built under ordinary conditions to meet the provisions in Chapter 8 of "Design: General Requirements."

[Commentary] Structures should meet the designated performance requirements throughout their design service life. Chapter 8 of "Design: General Requirements" describes the methods for verifying that no problems occur throughout the design service life due to the deterioration of materials in the structure subjected to chloride intrusion, carbonation, frost action or chemical intrusion. Structures are checked for expected deterioration phenomena. The structures built under ordinary environment with few effects of chloride intrusion may actually pass various durability checks described in Chapter 8 of "Design: General Requirements" easily if highly durable concrete with a water-cement ratio below a certain level is used, a certain concrete cover depth is provided and the crack width is held to a certain level. For such structures, therefore, design work may be simplified by selecting the concrete cover and water-cement ratio that pass the check for carbonation. Chapter 2 of this Specification presents the minimum concrete cover and water-cement ratio of concrete that should be provided by concrete structures constructed under a general environment for the beam, column and slab. A general environment here refers to an environment free from the fear of chloride intrusion, frost action or chemical intrusion. Selecting concrete cover and water-cement ratio in the specified range and achieving the limit crack width shown in Section 8.3.2 of Chapter 8 of "Design: General Requirements" enable the elimination of durability check described in Chapter 8 of "Design: General Requirements".

If a structure is away from the seacoast by at least 1.0 km, it is generally free from the effects of flying salt. Even if the structure is away from the seacoast by a larger margin, it may sometimes be subjected to the effects of flying salt depending on the surrounding topography and meteorological conditions including monsoon or oceanographic conditions. Decision should therefore be made for individual structures.

For structures vulnerable to chloride intrusion due to flying salt ,or special structures or structures requiring a high level of durability although not under the effects of flying salt, durability check should be made as stipulated in Chapter 8 of "Design: General Requirements".

When conducting durability check using the method described in Chapter 8 of "Design: General Requirements", if the range of combinations of specifications for passing the durability test is known easily, it will contribute to efficient design work. Chapter 3 of this Specification presents nomograms and mathematical tables that facilitate the identification of the range of parameters for concrete that passes durability tests for chloride intrusion, carbonation, frost action or chemical intrusion such as the material, mix proportions and concrete cover.

CHAPTER 2 CONCRETE COVER OF STRUCTURES UNDER ORDINARY ENVIRONMENTS

(1) If an ordinary structure constructed under a general environment meets the requirements for the water-cement ratio of concrete and the concrete cover shown in Table 2.1 and the crack width is below the limit shown in Section 8.3.2 of Chapter 8 of "Design: General Requirements", the structure may be assumed to pass the check for carbonation specified in Chapter 8 of "Design: General Requirements."

Table 2.1 Minimum concrete cover and maximum water-cement ratio of structures meeting standard durability requirements*

	Maximum value of W/C**	Minimum concrete cover c (mm)	Construction error Δc_e (mm)
Column	50	45	±15
Beam	50	40	±10
Slab	50	35	±5
Bridge Pier	55	55	±15

* A design service life of 100 years is assumed.
** Ordinary Portland cement is used.

(2) When the durability check specified in Chapter 8 of "Design: General Requirements" is eliminated by adopting the water-cement ratio and concrete cover shown in Table 2.1, the values of the parameters should be presented in the design drawings.

[Commentary] (1) In concrete structures constructed under a general environment with little chloride intrusion, the corrosion of reinforcing bars can be prevented by giving concrete cover so as to provide adequate durability against deterioration factors such as carbonation expected under an ordinary environment. The concrete cover is generally provided by the design engineer considering concrete quality, environmental conditions of the structure, construction error and importance of the structure with a view to passing durability check. Shown here are standard combinations of concrete quality and concrete cover for which durability check may be eliminated without causing any problem under a general environment. Table 2.1 lists the standard values for the maximum water-cement ratio, minimum concrete cover and construction error for concrete using ordinary Portland cement, for respective members. If the requirements listed in the table are satisfied and the crack width is below the limit under a general environment shown in Section 8.3.2 of Chapter 8 of "Design: General Requirements", the structure may be assumed to pass the check for carbonation described in Chapter 8 of "Design: General Requirements. Table 2.1 assumes no cases where either inspection or repair is difficult for the completed structure, where the structure is constructed under severe conditions or where precast members are used even under a general environment. In these cases, it should be verified that durability requirements are met based on the results of durability check described in Chapter 8 of "Design: General Requirements."

(2) As stipulated in Chapter 4 of "Design: General Requirements," the design drawings should carry all the characteristic values specified in design and be handed to construction engineers. The characteristic values for concrete concerning durability check include the coefficient of rate of carbonation, chloride ion diffusion coefficient and relative dynamic modulus of elasticity. If the durability check described in Chapter 8 of "Design: General Requirements" is eliminated in accordance with this Specification by adopting the concrete cover and water-cement ratio shown in Table 2.1, the concrete cover and water-cement ratio should be provided in design drawings instead of the characteristic values for durability check. The assumed construction error should also be provided.

CHAPTER 3　SIMPLE DESIGN METHOD CONCERNING DURABILITY

3.1　General

This chapter describes simple methods for identifying the range of parameters such as concrete materials, mix proportions and cover that pass the test in the durability check for chloride intrusion, carbonation and chemical intrusion shown in Chapter 8 of "Design: General Requirements."

[Commentary] This chapter presents nomograms and mathematical tables that facilitate the identification of the range of parameters such as concrete material, mix proportions and cover that satisfy various check equations shown in Chapter 8 of "Design: General Requirements". Values are obtained by back calculation of solutions satisfying the check equations shown in Chapter 8 of "Design: General Requirements" under limited conditions.

3.2　Chloride Attack

3.2.1　Check for steel corrosion due to chloride ion ingress

(1) In order to pass the test in the check for the corrosion of reinforcement steel due to chloride ion intrusion under an environment causing chloride ion intrusion, an appropriate combination of design concrete cover c_d and design diffusion coefficient for chloride ion D_d should be specified.

(2) In order to achieve the specified design diffusion coefficient D_d, an appropriate combination of the width of flexural crack and water-cement ratio of concrete should be specified.

[Commentary] (1) Figure C 3.2.1 is used to simply find a combination of design concrete cover c_d and design diffusion coefficient D_d for chloride ion that passes the test in durability check, under the given conditions without using any equations shown in Chapter 8 of "Design: General Requirements" when conducting durability check for the corrosion of reinforcement steel due to chloride ion intrusion described in Chapter 8 of "Design: General Requirements". Equation (8.3.5) in the "Design: General Requirements" ($\gamma_i C_d/C_{lim} = 1$) is effective along the boundary line between the areas with and without chloride intrusion.

Figure C 3.2.1 can be used as described below.

(i) Chloride ion concentration on concrete surface C_0 is specified according to the environmental conditions given.

(ii) Limit concentration for the occurrence of steel corrosion C_{lim} is specified.

(iii) Safety factors γ_{cl} and γ_i are specified.

(iv) $\gamma_{cl}\ C_0/(C_{lim}/\gamma_i)$ is calculated using the values (i) through (iii) above.

(v) The value obtained in (iv) is plotted on the vertical axis. A straight line is drawn from the point that cuts the line, parallel to the horizontal axis.

(vi) The value of the intersection of the straight line drawn in (v) above and a curve representing $\gamma_i C_d / C_{lim} = 1$ is read on the horizontal line as $c_d / \sqrt{D_d \cdot t}$.

(vii) Service life t for chloride ion intrusion is substituted in $c_d / \sqrt{D_d \cdot t}$ obtained in (vi) to obtain $c_d / \sqrt{D_d}$. The result is the limit value for the combination of design concrete cover c_d and design diffusion coefficient for chloride ion D_d that passes the examination in the check (combination of the minimum design concrete cover and the maximum design diffusion coefficient).

(viii) If c_d or D_d is properly specified, the range of the other parameter for passing the examination in the check may be obtained.

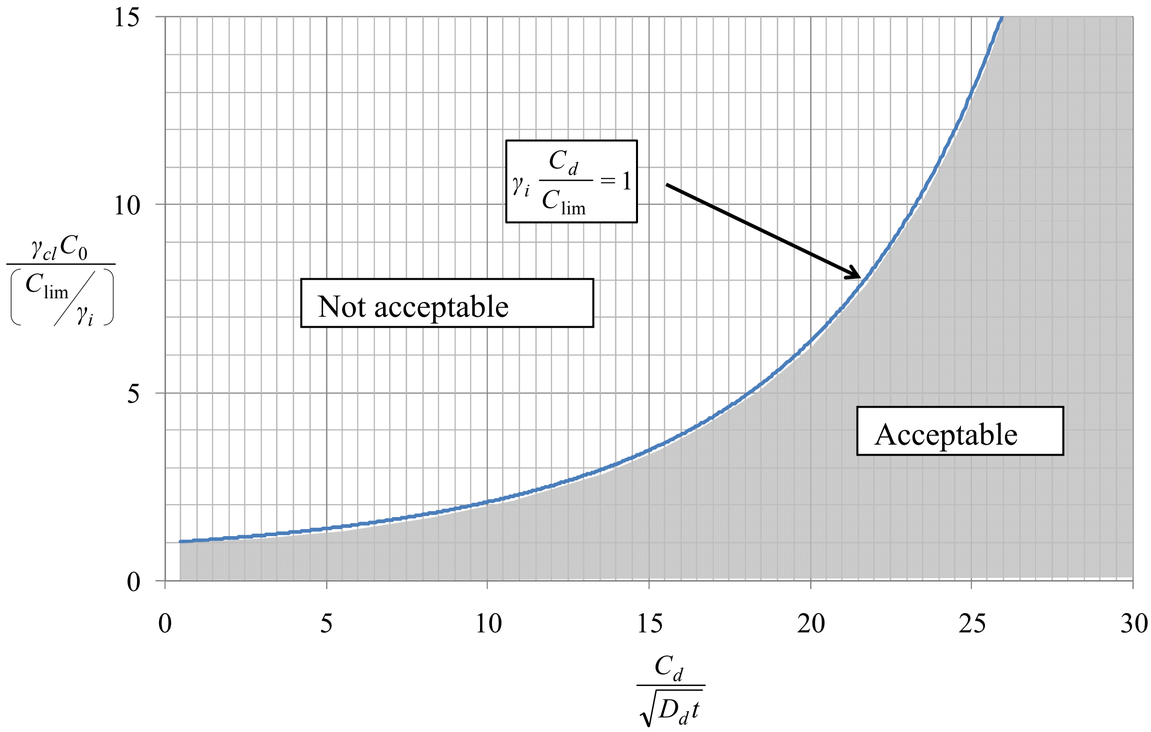


Fig. C 3.2.1 Combination of design concrete cover c_d and design diffusion coefficient for chloride ion D_d that passes the examination for chloride attack

Exp. Table 3.2.1 lists the limit values of parameters that pass the examination in the "check for steel corrosion due to chloride attack " described in Chapter 8 of "Design: General Requirements". The table shows the relationships between the parameters on the boundary line between the areas with and without chloride intrusion in Fig. C 3.2.1, or in the case where ($\gamma_i C_d / C_{lim} = 1$) is effective in equation (8.3.5) in the "Design: General Requirements". The objective of the table is also to simply find a combination of design concrete cover c_d and design diffusion coefficient for chloride ion D_d that passes the test in the check under given conditions without using any equations shown in Chapter 8 of "Design: General Requirements". The values in the table were obtained under the following conditions. The values should not be used as they are under different conditions.

Chloride ion concentration on concrete surface C_0: Value shown in Table C 8.2.2 of the "Design: General Requirements."

Limit concentration for the occurrence of corrosion of reinforcement steel C_{lim}: 1.2 kg/m^3.

Safety factor γ_{cl} considering the variation of design chloride ion concentration at the location of reinforcement C_d: 1.3.

Table C 3.2.1 Maximum design diffusion coefficient for passing the examination for chloride ingress D_d (approximate value)

splash zone (C_0=13kg/m^3) design concrete cover (mm)

life time	25	30	35	40	50	60	70	100	150	200
20 year	-	-	-	0.123	0.192	0.276	0.376	0.767	1.72	3.07
30 year	-	-	-	-	0.128	0.184	0.25	0.511	1.15	2.04
50year	-	-	-	-	-	0.11	0.15	0.307	0.69	1.23
100year	-	-	-	-	-	-	-	0.153	0.345	0.613

near shoreline (C_0=9kg/m^3) design concrete cover (mm)

life time	25	30	35	40	50	60	70	100	150	200
20 year	-	-	0.115	0.15	0.235	0.338	0.46	0.939	2.11	3.75
30 year	-	-	-	0.1	0.156	0.225	0.307	0.626	1.41	2.5
50year	-	-	-	-	-	0.135	0.184	0.375	0.845	1.5
100year	-	-	-	-	-	-	-	0.188	0.422	0.751

0.1km from coast (C_0=4.5kg/m^3) design concrete cover (mm)

life time	25	30	35	40	50	60	70	100	150	200
20 year	-	0.14	0.191	0.249	0.389	0.561	0.763	1.56	3.5	6.23
30 year	-	-	0.127	0.166	0.26	0.374	0.509	1.04	2.34	4.15
50year	-	-	-	-	0.156	0.224	0.305	0.623	1.4	2.49
100year	-	-	-	-	-	0.112	0.153	0.311	0.7	1.25

0.25km from coast (C_0=3kg/m^3) design concrete cover (mm)

life time	25	30	35	40	50	60	70	100	150	200
20 year	0.15	0.216	0.295	0.385	0.601	0.866	1.18	2.4	5.41	9.62
30 year	0.1	0.144	0.196	0.256	0.401	0.577	0.785	1.6	3.61	6.41
50year	-	-	0.118	0.154	0.24	0.346	0.471	0.962	2.16	3.85
100year	-	-	-	-	0.12	0.173	0.236	0.481	1.08	1.92

0.5km from coast (C_0=2kg/m^3) design concrete cover (mm)

life time	25	30	35	40	50	60	70	100	150	200
20 year	0.288	0.414	0.564	0.737	1.15	1.66	2.26	4.61	10.4	18.4
30 year	0.192	0.276	0.376	0.491	0.768	1.11	1.5	3.07	6.91	12.3
50year	0.115	0.166	0.226	0.295	0.461	0.663	0.903	1.84	4.14	7.37
100year	-	-	0.113	0.147	0.23	0.332	0.451	0.92	2.07	3.68

1km from coast (C_0=1.5kg/m^3) design concrete cover (mm)

life time	25	30	35	40	50	60	70	100	150	200
20 year	0.62	0.893	1.22	1.59	2.45	3.57	4.86	9.92	22.3	39.7
30 year	0.413	0.595	0.81	1.06	1.65	2.38	3.24	6.61	14.9	26.4
50year	0.248	0.357	0.486	0.635	0.992	1.43	1.94	3.97	8.93	15.9
100year	0.124	0.179	0.243	0.317	0.496	0.714	0.972	1.98	4.46	7.93

Extremely small D_d values (0.1 or smaller) have been eliminated.

(2) The characteristic value D_k of chloride ion diffusion coefficient represents inherent characteristics of concrete. It is dependent on the concrete materials and mix proportions, and can be obtained by a designated testing method. Additionally, Chapter 5 of "Design: General Requirements" provides equations for predicting the diffusion coefficient from the water-cement ratio of concrete in cases where ordinary Portland cement, blast furnace slag cement or silica fume is used.

Figure C3.2.2 shows the "relationship between water-cement ratio and design diffusion coefficient" calculated using the relation between the water-cement ratio and concrete chloride ion diffusion coefficient (C5.2.25) shown in Chapter 5 of "Design: General Requirements" and the equation for predicting diffusion coefficient considering flexural cracking (8.3.8) shown in Chapter 8 of "Design: General Requirements". As the calculation conditions, concrete cover c was 50 mm, allowable crack width w_a was 0.2 mm, concrete material coefficient γ_c was 1.0, and the constant representing the effect of cracking on the transfer of chloride ions in concrete D_0 was 200 cm^2/year. Cases were assumed where the crack width was 0.5, 0.75 or 1.0 time the allowable crack width. This figure shows combinations of the maximum flexural crack width and the maximum water-cement ratio for achieving the target design diffusion coefficient. In cases where the conditions are somewhat different from the above calculation conditions, the qualitative tendency between the two parameters is more or less the same, so the figure is applicable. The values in the figure, however, should not be used as they are.

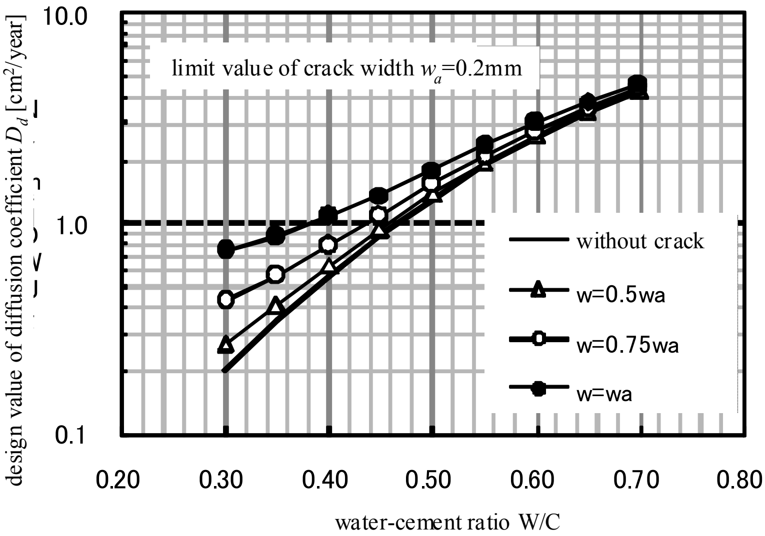


Fig. C3.2.2 Effects of water-cement ratio and crack width on diffusion coefficient

Exp. Fig. 3.2.2 can be used as described below.

(i) A crack width is specified for control purposes, and the curve to be used is determined. (As an example, w is set to be 0.15 mm, or 0.75 w_a).

(ii) The value of design diffusion coefficient D_d used for checking for steel corrosion is plotted on the vertical axis. Then, a straight line is drawn from the point cut by the vertical axis, parallel to the horizontal axis.

(iii) Water-cement (W/C) ratio is obtained from the intersection of the straight line drawn in (ii) above and the curve at w = 0.75 w_a.

(iv) The water-cement ratio obtained in (iii) above is the upper limit of allowable water-cement ratio.

Table C3.2.2 is a mathematical table for simply obtaining the maximum water-cement ratio and crack width required for achieving the target design diffusion coefficient D_d. The values in the table should not be used as they are under different conditions than those in this table.

Table C3.2.2 Design diffusion coefficient D_d calculated from crack width and water-cement ratio

W/C	without crack	crack width (mm)		
		0.1	0.15	0.2
0.30	0.20	0.271	0.429	0.751
0.35	0.35	0.413	0.567	0.873
0.40	0.57	0.633	0.782	1.08
0.45	0.89	0.95	1.1	1.38
0.50	1.33	1.39	1.53	1.81
0.55	1.90	1.96	2.1	2.37
0.60	2.60	2.65	2.79	3.05
0.65	3.39	3.45	3.58	3.84
0.70	4.24	4.29	4.42	4.68

Calculation conditions
Type of cement: Ordinary Portland cement
Concrete cover c: 50 mm
Allowable crack width w_a: 0.2 mm
Material coefficient for concrete γ_c: 1.0
Constant indicating the effect of cracking on the transfer of chloride ions in concrete D_0: 200 cm^2/year

3.2.2 Specification of coefficient of chloride ion diffusion in concrete

Coefficient of chloride ion diffusion in concrete may be obtained by any of the following methods.

(1) Use of the relationship between water-cement ratio and apparent diffusion coefficient

(2) Laboratory tests using electrophoresis or soaking method, or weathering tests

(3) Investigations of actual structures

[Commentary] The coefficient of chloride ion diffusion in concrete is affected by the material and mix proportions used, and also by the environmental conditions. The diffusion coefficient may vary even if the same material and mix proportions are adopted. The methods for obtaining diffusion coefficients subjected to various effects include the use of the relationship between the water-cement ratio and apparent diffusion coefficient from existing data, implementation of laboratory tests, and use of core samples collected from existing structures that are under the similar natural conditions and expected to be subjected to similar actions to those for the structure to be designed, or of exposed specimens.

(1) Several regression equations have been developed based on the surveys of distributions of chloride ions in existing concrete as equations for predicting the apparent diffusion coefficient from concrete material and mix proportions. Section 5.2.12 of "Design: General Requirements" provides equations for predicting the apparent diffusion coefficient from water-cement ratio in cases where ordinary Portland cement, blast furnace slag cement or silica fume is used.

(2) For obtaining chloride ion diffusion coefficient in laboratory tests, refer to JSCE 571: Method for Testing Effective Diffusion Coefficients of Chloride Ions in Concrete by Electrophoresis (draft) and JSCE 572: Method for Testing Apparent Diffusion Coefficients of Chloride Ions in Concrete by Soaking (draft). When using core samples collected from actual

structures, refer to JSCE 573: Method for Measuring the Distribution of Chloride Ions in Concrete in Actual Structures (draft).

Laboratory tests (refer to Table C3.2.3) are beneficial because they enable the direct acquisition of diffusion coefficients for the material and mix proportions related to material design. The degree of freedom increases in using concrete materials and mix proportions. Laboratory tests are therefore effective for using new types of materials in particular.

Table C3.2.3 Methods for obtaining diffusion coefficients in laboratory tests

Test method	Standard	Remarks
Electrophoresis	JSCE 571: Method for Testing Effective Diffusion Coefficients of Chloride Ions in Concrete by Electrophoresis (draft)	Conversion of effective diffusion coefficient to apparent diffusion coefficient is required.
Soaking method	JSCE 572: Method for Testing Apparent Diffusion Coefficients of Chloride Ions in Concrete by Soaking (draft)	

In the soaking method, concrete specimens are soaked in salt water with increased density for a designated period of time, the specimen is sliced to obtain chloride ion concentration in concrete at different depths and the diffusion coefficient is calculated from the distribution of chloride ion concentrations. One of the benefits of the method is the direct acquisition of apparent diffusion coefficients used for checking as stipulated in the specifications. The test, however, requires several months. The test period is expected to be more than one year for concrete with a low water-cement ratio of 40% or lower. In cases where chloride ions enter concrete only near the concrete surface while the soaking method is adopted because of a low water-cement ratio, the distribution of concentrations should be obtained for calculating the apparent diffusion coefficient, in accordance with JSCE G574: Planar Analysis Method for Elements in Concrete by the EPMA Method (draft).

In electrophoresis, chloride ions are forced to move by applying direct current constant voltage to concrete, and the rate of movement is used to obtain the diffusion coefficient. The test is finished within one month and effective for measuring the diffusion coefficient in concrete with a low water-cement ratio for which the soaking method requires a long time. Electrophoresis aims at measuring not the apparent diffusion coefficient shown in specifications but the effective diffusion coefficient that represents the ease of movement of chloride ions in concrete micro pore solution. The values obtained by electrophoresis cannot be used as they are for checking the specifications. The effective diffusion coefficient is converted to the apparent diffusion coefficient for checking purposes because two are theoretically correlated to each other.

When using the specimens for weathering tests, electrophoresis is effective because it can reflect the environmental conditions of the structure to be designed relatively properly. The weathering period increases at a low water-cement ratio as for the soaking method.

(3) A method is available for measuring chloride ion distribution in concrete in a structure separate from the methods of chloride ion diffusion coefficient testing in laboratory. In this method, samples are collected from structures and chloride ions in samples are analyzed to specifications, and the apparent diffusion coefficient is calculated based on the measured

distribution of chloride ion concentrations. The diffusion coefficient in concrete in an actual structure obtained can be reflected direct in a new structure.

3.2.3 Chloride ion concentration on concrete surface

The chloride ion concentration used for chloride intrusion checking may be specified according to the volume of flying salt in the study area.

[Commentary] When checking the chloride intrusion by the method described in Chapter 8 of "Design: General Requirements", the effects of flying salt on the volume of chloride ion intrusion is considered based on the chloride ion concentration on concrete surface C_0. Chloride ion concentration on concrete surface C_0 has been known to vary according to the climate of the area where the structure is constructed, topography surrounding the structure and element of the structure. Using the value of chloride ion concentration on concrete surface C_0 based on the existing records and measurements for similar structures is desirable because the concentration well represents the environmental conditions of the structure.

The volume of flying salt that affects the chloride ion concentration on concrete surface has been known to be determined not only by the distance from the seashore or elevation but also by the regional characteristics, e.g. whether the region is located on the coast of the Japan Sea or the Pacific Ocean. The past surveys have revealed that the volume of flying salt is higher along the coast of the Japan Sea in the Hokkaido, Tohoku and Hokuriku regions than in the coastal areas in other regions. In Okinawa Prefecture, the volume is also high. A conceivable reason may be the passage of winter monsoon or typhoons. Chloride ion concentration on concrete surface should therefore be specified according to the meteorological conditions of the region under study.

Section 8.2 of "Design: General Requirements" provides a mathematical table for specifying chloride ion concentrations on concrete surface according to the region where the structure is located, distance from the structure to the seashore and elevation (Table C8.2.2). The surface chloride ion concentrations in the table may be used if the past records or measurements for similar structures are not used.

The chloride ion intrusion into concrete is affected by the volume of flying salt that adheres to concrete surface. The volume of chloride ions on concrete surface may sometimes be higher on the sea side of a bridge that is parallel to the coastal line than on the land or mountain side. The salt that adheres to concrete surface may sometimes be removed by rain, wind or snow. The values shown in Table C8.2.2 in Chapter 8 of "Design: General Requirements" do not reflect such local conditions but are for the elements of a structure susceptible to chloride intrusion because of a large volume of flying salt.

3.3 Carbonation

For checking steel corrosion due to carbonation, an appropriate combination of design concrete cover c_d and water-cement ratio should be selected according to the environmental conditions.

[Commentary] In order to pass the test for carbonation checking, the design thickness of carbonated concrete (thickness of carbonated concrete at the end of design service life) should not reach the limit thickness for the occurrence of steel corrosion. To that end, concrete cover should be increased to increase the limit thickness for the occurrence of steel corrosion or concrete should be made resistant to carbonation by reducing the water-cement ratio of concrete.

Figure C3.3.1 shows a sample area where the design concrete cover c_d and water-cement ratio *W/C* of concrete used for environmental conditions check meet the requirements in carbonation check. The figure is drawn while standard safety factors are used (γ_{cb} = 1.15, γ_c = 1.0, γ_i = 1.0 and γ_p = 1.0), constant representing the environmental conditions β_e is 1.6 (likely to get dry), service life for carbonation t is 100 years, concrete cover minus thickness of carbonated concrete c_k is 10 mm and the diameter of reinforcement steel is 25 mm. If these parameters have different values, this figure is inapplicable without modifications.

If concrete with a low water-cement ratio is used, the requirements can generally be met in carbonation check even with a thin concrete cover. Concrete cover should, however, not be less than the diameter of reinforcement steel.

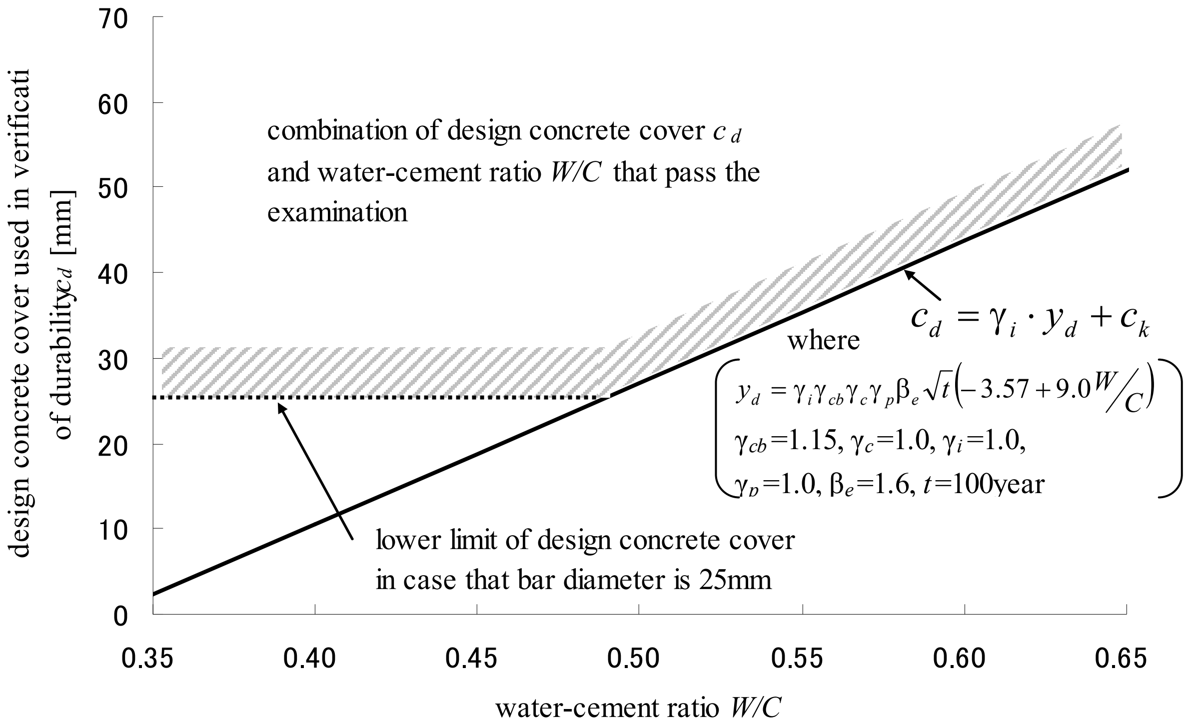


Fig. C3.3.1 A sample relation between the thickness of carbonated concrete and water-cement ratio

3.4 Damage from Freezing and Thawing

For checking damage from freezing and thawing, an appropriate combination of air content in concrete and water-cement ratio should be selected according to the environmental conditions.

[Commentary] To check damage from freezing and thawing, it should be verified in freezing and thawing tests that the design relative dynamic modulus of elasticity E_d exceeds $\gamma_i E_{min}$ (where γ_i is a structure factor and E_{min} is the minimum limit value of relative dynamic modulus of elasticity in freezing and thawing tests). In the design phase, therefore, a design value may be specified higher than the minimum limit value of relative dynamic modulus of elasticity that meets the performance requirements for concrete structures against damage by freezing and thawing, referring to Table 8.4.1 in Chapter 8 of "Design: General Requirements". If a high relative dynamic modulus of elasticity of 100% is specified as the design value, for example, the requirements for checking may be met but the conditions for mix proportions become severe during the production of concrete. For reference, plotted on Fig. C3.4.1 are relationships between water-cement ratio and relative dynamic modulus of elasticity in freezing and thawing tests available in existing researches. The polygonal line in the figure shows a relationship for the maximum water-cement ratio for achieving the designated relative dynamic modulus of elasticity shown in Section 5.2.8 of "Design: General Requirements". The air content of concrete varies from 4 to 7%.

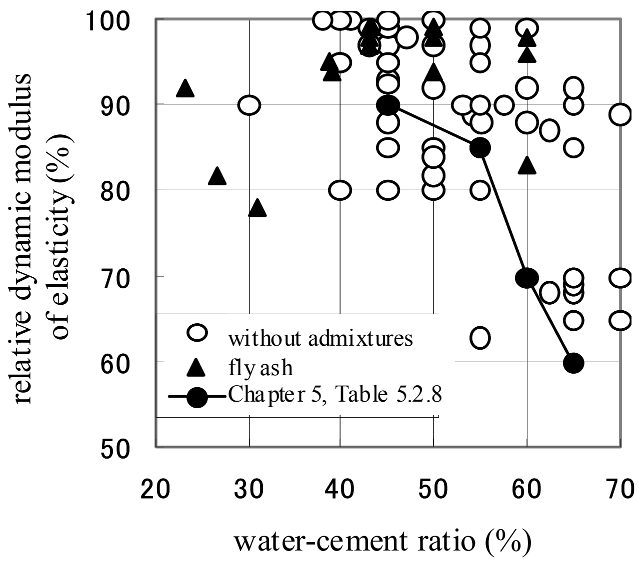


Fig. C3.4.1 Relationship between water-cement ratio of concrete and relative dynamic modulus of elasticity in freezing and thawing test

In cases where the characteristic value for the relative dynamic modulus of elasticity is 90% or higher in the freezing and thawing tests for an ordinary structure, or where water-cement ratio is 45% or lower and air content is 6% or higher where the structure is affected by chlorides in deicing agents or sea water, the check for damage by freezing and thawing may be eliminated.

In cases where the structure is required to maintain high level of durability, or maintain the initial soundness without losing its aesthetic value during its service life, the relative dynamic modulus of elasticity and mass of the specimen should not be reduced in freezing and thawing tests. The design relative dynamic modulus of elasticity during freezing and thawing testing should sometimes be set at 95% or higher.

3.5 Chemical Attack

The design depth of chemical attack should be set below the design concrete cover.

[Commentary] If the design concrete cover has been specified under certain design conditions, the check requirements can be met by specifying the design depth of chemical attack below the design value of concrete cover. Then, instructions should be given to manufacture concrete so as to meet the specified depth of chemical attack. For the concrete structures constructed against expected chemical attack, it should be verified that the concrete is sufficiently resistant to chemical attack based on the results or records of existing researches or the results of experiments.

When specifying the depth of chemical attack during the service life first, check requirements may be met by specifying the design concrete cover larger that the depth of chemical attack. Then, however, the maximum value among the design values of concrete cover specified under different conditions should be specified as the design concrete cover.

PART 4　THERMAL STRESS ANALYSIS

CHAPTER 1　GENERAL

1.1　Scope

This Specification presents examples of thermal and stress analyses required for verifying whether the cracking due to cement hydration affects the performance of the concrete structure or not as stipulated in Chapter 12 of "Design: General Requirements". Examples of analysis models and standard values representing the thermal and mechanical properties of materials required for analysis are also presented.

[Commentary] For verifying whether cracking occurs or not owing to cement hydration and checking crack width, the heating due to cement hydration, resultant concrete temperature variations and changes in volume of concrete due to temperature variations and autogenous shrinkage should be identified accurately. The stress of concrete caused by volumetric change should be calculated accurately. With the remarkable improvement of arithmetic processing capacity of computers in recent years, analytical processing now requires less time than before and various analysis methods and models are available. The analysis methods and models have both advantages and disadvantages. The most appropriate analysis method and model should be selected considering various conditions. This Specification shows standard analysis methods for verifying whether the initial cracks due to cement hydration affect the performance of concrete structures or not. Chapters 2 and 3 present standard methods for thermal analysis and stress analysis of concrete, respectively. Chapter 4 describes the mechanical and thermal properties of concrete and the thermal properties of soils and rocks that are required for the above analyses.

CHAPTER 2　THERMAL ANALYSIS

2.1 Analytical Methods

The thermal analysis of concrete shall be conducted using adequate methods in accordance with factors such as the type and configuration of the structure.

[Commentary] The methods of the thermal analysis of concrete include numerical methods such as finite element method, finite difference method, simplified numerical method such as Schmidt or Carlson method, and simplified calculation methods with the use of figures and tables, whichever can be used. Thermal analyses of concrete can also be classified into linear analyses, which assume that heat generation characteristics and thermal properties are not dependent on temperature, and nonlinear analyses, which allow for temperature dependence. However, since each method has limitation in its application, it is necessary to select the most adequate method considering the required accuracy of the analysis and various conditions of the structures subjected to the analysis. In thermal analysis, it is necessary to determine a valid analytical model and boundary conditions in accordance with environmental and other conditions of the objective structure. Calculation of temperature changes and thermal stresses may be conducted until the temperature change becomes practically stationary.

2.2　Boundary and Initial Condition

The boundary and initial condition used in the thermal analysis of concrete shall be determined adequately considering such factors as the configuration of the structure, the conditions of heat dissipation and the initial temperature of the concrete.

[Commentary] The determination of boundary conditions is as fundamental as that of the analytical model and has a significant influence on the results of the analysis. The heat transfer boundary is a boundary where heat is transferred between the concrete and atmosphere and the fixed temperature boundary is a boundary where the temperature is kept constant. In general, radiation can be ignored in the thermal analysis of concrete.

The heat transfer boundary shall be characterized by the coefficient of heat transfer. The coefficient of heat transfer shall be determined considering such factors as the use or nonuse of concrete forms, their type and thickness, the time from placement until stripping, curing method and wind speed.

Dealing directly with convection and diffusion of air layers near the boundary is one way to accurately reflect the characteristics of the heat transfer boundary. Characteristics of the heat transfer boundary, however, can be easily reflected in analyses by using a coefficient of heat transfer reflecting the average effect of those characteristics. The coefficient of heat transfer has a significant influence on concrete temperature at and near the surface of members, and also on the temperature rise in the interior when the thickness of the member is relatively thin. The influence of wind velocity on the exposed surface of concrete is generally expressed as 12-14 W/m^2 ºC in terms of the coefficient of heat transfer per 2-3m/s of wind velocity. The coefficient of heat transfer increases as wind velocity increases at a rate of 2.3 to 4.6 W/m^2 ºC per m/s of wind velocity. When determining the coefficient of heat transfer considering the influence of such factors as the forms and curing method, the following equation may be used:

$$\eta = \frac{1}{\frac{1}{\beta} + \sum \frac{d_{Fi}}{\lambda_{Fi}}} \tag{C2.2.1}$$

where,

η : modified coefficient of heat transfer, (W/m^2°C)

β : coefficient of heat transfer at surface exposed to atmosphere, (W/m^2°C)

(generally taken as 12-14W/m^2°C)

d_{Fi} : thickness of curing material, (m)

λ_{Fi} : thermal conductivity of curing material, (W/m^2°C)

The reference data of the coefficient of heat transfer are shown in Table C2.2.1.

Table C2.2.1 Reference value of coefficient of heat transfer η

No.	Curing method	η (W/m^2°C)
1	Metal frame Water sprinkling（sprinkling water depth: less than 10mm）	14
2	Water sprinkling（Sprinkling water depth10mm~50mm） Including ....curing	8
3	Water sprinkling (Sprinkling water depth: 50mm~100mm)	8
4	Plywood	8
5	Sheet	6
6	Curing mat Water sprinkling＋Curing mat, Water sprinkling + Sheet	5
7	Styrene foam（Thickness: 50mm）+ Sheet	2

2.3 Rate of Heat Generation of Concrete

The rate of heat generation used in the thermal analysis of concrete shall be modeled, considering the age of concrete and concrete temperature depending on portion in the structure. If, however, the minimum member size is not smaller than 0.5 m, it may be assumed in the thermal analysis that the adiabatic temperature rise of concrete is regarded as common characteristics of concrete throughout the structure.

[Commentary] The rate of heat generation in concrete is strongly influenced by temperature. Since temperatures in a structure are usually not uniform, different parts of the structure usually generate heat at different rates. In a thermal analysis, therefore, it is desirable that thermal properties be determined from mix parameters such as the type of cement, chemical and nonchemical admixtures, cement content and water-cement ratio, and calculated rates of heat generation reflecting temperature dependence be used. At present, however, few thermal analyses are conducted in that way.

If the minimum member size is not smaller than 0.5 m, the rate of heat generation in most parts of a concrete member except its exposed surface is almost same as that of adiabatic heat generation. In such cases, it has been analytically confirmed that accuracy of thermal analysis does not suffer even if the adiabatic temperature rise characteristics of the concrete for the placing temperature are taken as its heat generation characteristics, the rate of heat generation in the concrete is determined accordingly, and it is assumed to be uniform throughout the structure. At present, however, it is common practice to use this approach even when the minimum member size is smaller than 0.5 m.

The mixing method used may influence the rate of heat generation several hours after the manufacture of concrete, but usually the influence on the calculation of the thermal cracking index is not substantial. In cases where hydration is retarded by the type and dosage of chemical admixtures, close attention needs to be paid to early-stage characteristics.

CHAPTER 3　STRESS ANALYSIS

3.1　Analytical Method

Stresses due to volume changes of concrete shall, in general, be calculated, by determining volume changes of restraints such as concrete, reinforcing steel and rock, soil or foundation, so that the boundary and compatibility conditions and the equilibrium condition are satisfied.

[Commentary]　Since mass concrete structures are usually more rigid than ordinary concrete structures, simplification of the compatibility conditions including the boundary conditions may result in lower accuracy of analysis.　It is desirable, therefore, to use an analysis method in which both the compatibility and equilibrium conditions are satisfied and volume changes and unsteady mechanical properties of concrete can be considered (e.g., finite element method).　Results of finite element analysis may vary depending on element resolution, analytical domain and boundary conditions, and the properties of the restraining body and the restrained body. If the finite element method is used, therefore, it is important to determine these parameters carefully, paying attention to past analysis data, so that the required level of accuracy is achieved. In the analysis considered here, $\sigma t(t)$ is defined as the maximum value of principal tensile stress in the concrete member at an age of t.

There are various approximation methods that can be used as a means of calculating stresses in mass concrete structures in which volume changes due to temperature changes are predominant over those due to autogenous shrinkage. One of those methods is the CP (CL) method described below.

In general, the restraining actions causing thermal stresses consist of the internal and external restraining actions. The external restraining action is classified into the one against axial deformation and the one against flexural deformation.

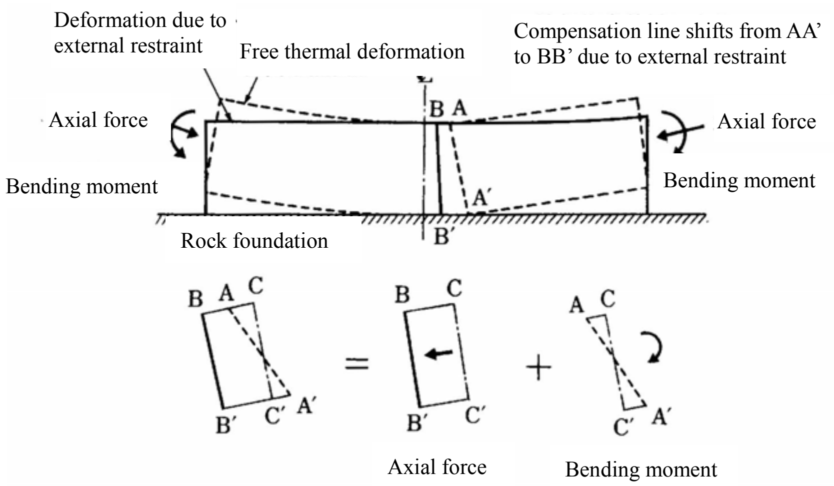


Fig. C3.1.1　External restraint condition

When free thermal deformation is caused in a concrete block, the deformation is separated into deformation in an axial direction, i.e., expansion and contraction, and flexural deformation in a vertical direction as shown in Fig. C3.1.1. Hence, two kinds of external restraining effects are considered concerning the restraint of these deformations. Therefore, induced stress is regarded to consist of the stress components due to the internal restraint, that due to the external restraint against axial deformation and that due to the external restraint against flexural deformation. Each stress component is calculated as follows:

When temperature distributions at ages of $t_{i\text{-}1}$ and t_i are obtained in mass concrete shown in Fig. C3.1.2, the distribution of changes in thermal strain can be obtained by multiplying temperature difference by the coefficients of thermal expansion a. The stress due to internal restraint occurs where the concrete block subjected to temperature change is free from any restraint of rock or other objects, but it is not caused by external axial forces or flexural moments. The stress therefore can be obtained by drawing the compensation line that represents an axial force of 0 and a flexural moment of 0. Increment of stress due to the internal restraint can be determined by Eq.(C3.1.1) from the difference between the compensation line and temperature strain distribution curve (shaded part of Fig. C3.1.2).

$$\Delta\sigma_{It} = E(t_i)(\alpha\Delta T - \varepsilon_{comp}) \tag{C.3.1.1}$$

where, ε_{comp} is a strain value at a point of the compensation line.

Curvature increment $\Delta\phi_i$ of free thermal deformation is obtained from the inclination of the compensation line, and so the increment of free axial strain ε_i from the strain at the center of gravity on the compensation line.

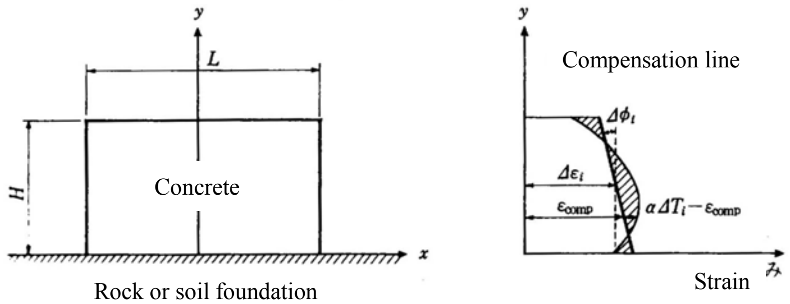


Fig. C3.1.2 Compensation line application

Restraining axial force and flexural moment occurring from age $t_{i\text{-}1}$ to age t_i are obtained by Eq. (C.3.1.2) and Eq. (C.3.1.3), respectively.

$$\Delta N_{Ri} = R_N E(t_i) A \Delta\varepsilon_i \tag{C.3.1.2}$$

$$\Delta M_{Ri} = R_M E(t_i) I \Delta\phi_i \tag{C.3.1.3}$$

Increment of stress due to the external restraint can be determined by the following equation:

$$\Delta\sigma_{Ri} = \frac{\Delta N_{Ri}}{A} + \frac{\Delta M_{Ri}}{I} y \tag{C.3.1.4}$$

where,

A: cross sectional area of mass concrete block,
I: moment of inertia of mass concrete block,
Y: distance from the center of gravity of mass concrete block

R_N, R_M: factors to represent the effects of restraining bodies such as a rock bed which restrains free thermal deformation of mass concrete block, with R_N for the degree of axial deformation restraint and R_M for that of flexural deformation restraint.

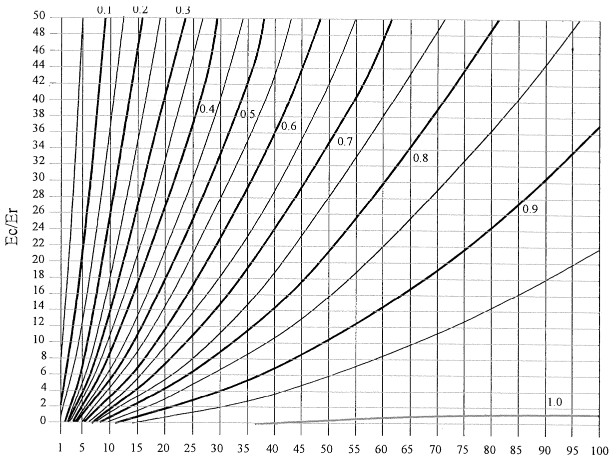
Thermal stress at each age is provided by Eq. (C3.1.5) as the summation of these stress increments.

$$\sigma = \sum_i (\Delta\sigma_{It} + \Delta_{\sigma Ri}) \tag{C.3.1.5}$$

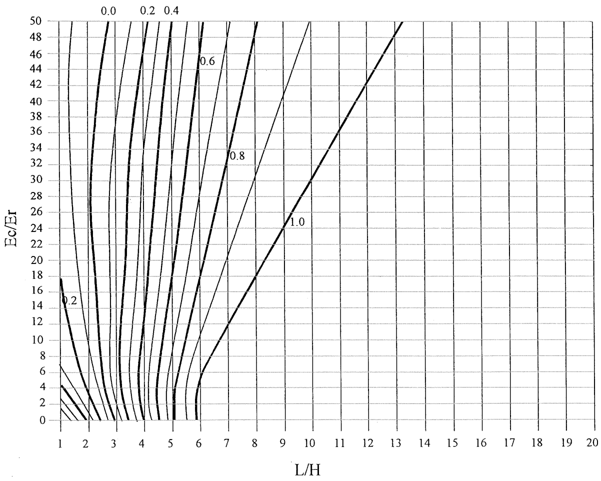
The values of R_N and R_M particularly depend on such factors as the difference of stiffness between the restraining body and the mass concrete block, and the ratio of the bottom length to the height (*L/H*) of the mass concrete block. An example of R_N and R_M are shown in Fig. C3.1.3 (for slabs) and Fig. C3.1.4 (for walls), which were obtained from the results of numerical calculation by a 3-dimensional finite element method. These coefficients of external restraint can be used as described in Table C3.1.1 (Usage of Coefficient of External Restraint).

Table C3.1.1 Application of external restraint factor

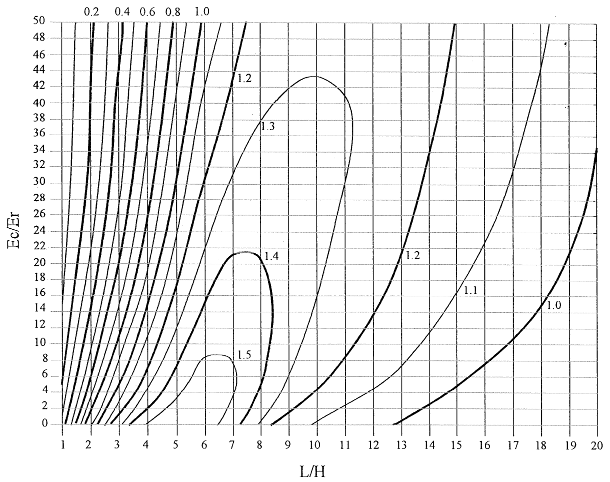
<table>
<tr><th rowspan="3">Type of structure</th><th rowspan="3">Restraining body</th><th rowspan="3" colspan="2">Lift division</th><th colspan="3">External restraint factor used for calculation</th></tr>
<tr><th rowspan="2">Axial restraint factor</th><th colspan="2">Bending restraint factor</th></tr>
<tr><th>Before reversing</th><th>After reversing</th></tr>
<tr><td rowspan="3">Slab structures</td><td rowspan="3">Soil foundation, rock foundation</td><td colspan="2">No</td><td>R_N
(Fig. C3.1.3(a))</td><td>R_{M1}
(Fig.C3.1.3(b))</td><td>R_{M2}
(Fig. C3.1.3 (c))</td></tr>
<tr><td rowspan="2">Yes</td><td>1 lift</td><td>R_N
(Fig. C3.1.3(a))</td><td>R_{M1}
(Fig.C3.1.3(b))</td><td>R_{M2}
(Fig. C3.1.3 (c))</td></tr>
<tr><td>At least 2 lifts</td><td>$0.8R_N$
(Fig. C3.1.3(a))</td><td>$0.8R_{M1}$
(Fig.C3.1.3(b))</td><td>$0.8R_{M2}$
(Fig. C3.1.3 (c))</td></tr>
<tr><td rowspan="3">Wall structure</td><td rowspan="3">Soil foundation, rock foundation</td><td colspan="2">Footing</td><td>R_N
(Fig. C3.1.3(a))</td><td>R_{M1}
(Fig.C3.1.3(b))</td><td>R_{M2}
(Fig. C3.1.3 (c))</td></tr>
<tr><td colspan="2">1 Lift</td><td>R_N
(Fig. C3.1.4(a))</td><td>R_{M1}
(Fig.C3.1.4(b))</td><td>R_{M2}
(Fig. C3.1.4 (c))</td></tr>
<tr><td colspan="2">At least 2 lifts</td><td>R_N
(Fig. C3.1.4(d))</td><td>R_{M1}
(Fig.C3.1.4(e))</td><td>R_{M2}
(Fig. C3.1.4 (f))</td></tr>
</table>



(a) Axial restraint factor R_N

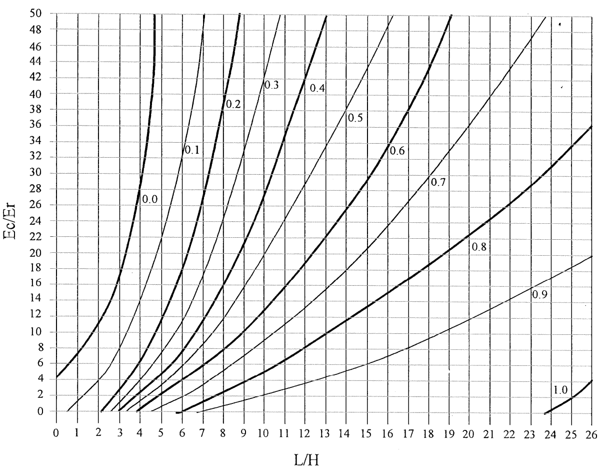


(b) Bending restraint factor R_{M1}
($\Delta\Phi$ before reversing)

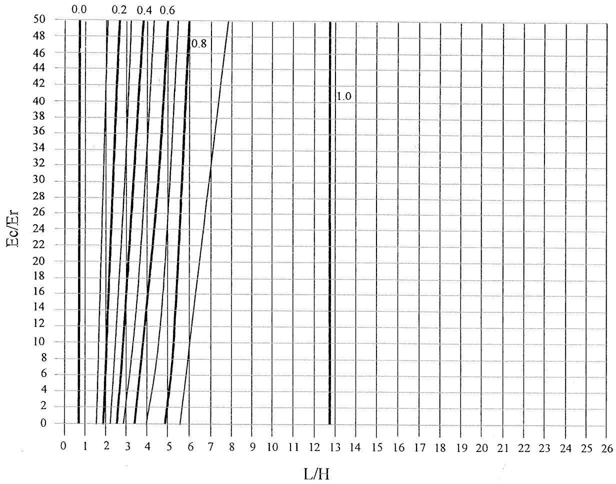


(c) Bending restraint factor R_{M2}
($\Delta\Phi$ after reversing)

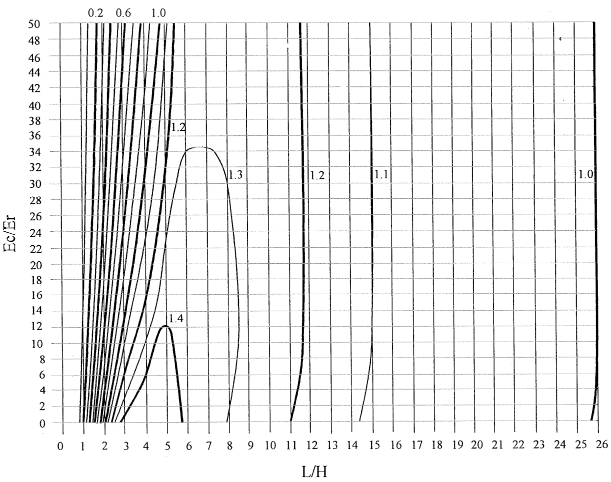
Fig. C3.1.3 External restraint factor (Slab structures)



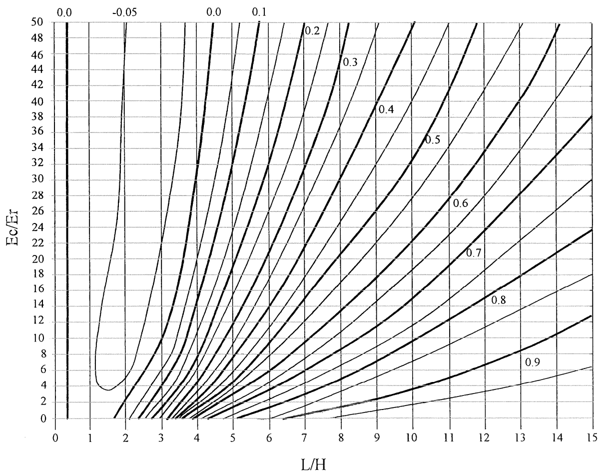
(a)Axial restraint factor R_N
(First lift)



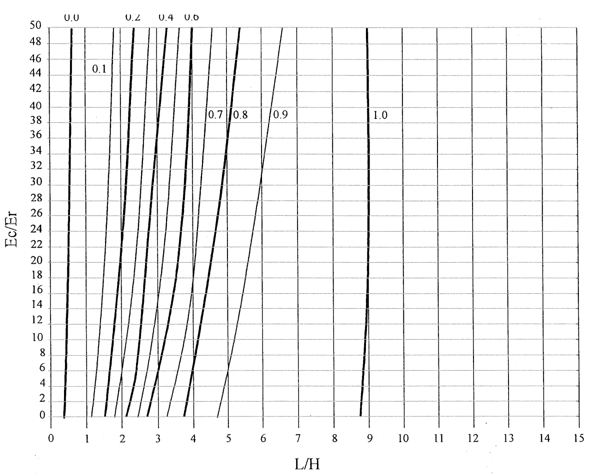
(b) Bending restraint factor R_{M1}
(First lift: ΔΦ before reversing)



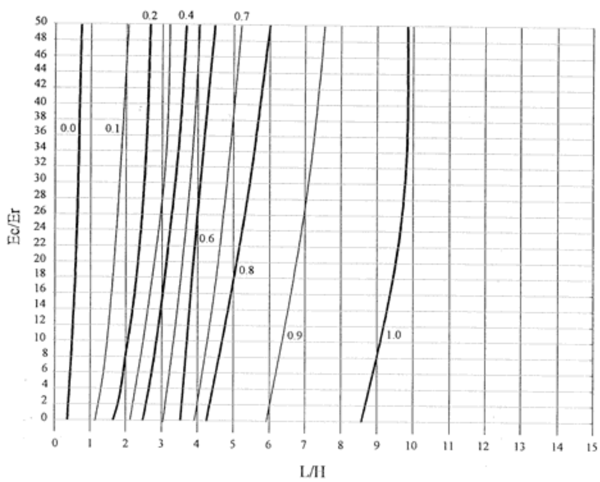
(c) Bending restraint factor R_{M2}
(First lift: ΔΦ after reversing)



(d) Axial restraint factor R_N
(Second lift)



(e) Bending restraint factor R_{M1}
(Second lift: ΔΦ before reversing)



(f) Bending restraint factor R_{M2}
(Second lift: ΔΦ after reversing)

Fig. C3.1.4 External restraint factor (Wall structures)

3.2 Consideration of Autogenous Shrinkage

In cases where autogenous shrinkage cannot be ignored, stresses shall be calculated considering both volume changes due to heat of hydration and autogenous shrinkage. Autogenous shrinkage shall be determined by means of a test or formula with proven applicability and accuracy.

[Commentary] Volume changes at the construction stage are usually influenced mostly by temperature changes. There are cases, however, where autogenous shrinkage due to hydration cannot be ignored depending on the type of cement or admixture or the mix proportion of concrete. In such cases, the accuracy of stress analysis can be improved by considering the effect of volume changes due to both temperature change and autogenous shrinkage. This section describes a method for considering the effects of autogenous shrinkage and temperature change.

In such cases, stresses may be calculated, on the assumption that thermal expansion and autogenous shrinkage are not dependent on stress, from effective strains calculated from Eq. (C3.2.1), taking into account the unsteady mechanical properties of concrete.

$$\varepsilon_{ij,ef} = \varepsilon_{ij} - \varepsilon_{ij,t} - \varepsilon_{ij,ag} \qquad \text{(C3.2.1)}$$

where,

ε_{ij}: total strain

$\varepsilon_{ij,\,ef}$: effective strain

$\varepsilon_{ij,\,t}$: thermal strain $=\alpha \Delta T \delta_{ij}$

α: coefficient of thermal expansion

δ: Kronecker delta

$\varepsilon_{ij,\,ag}$: autogenous shrinkage of concrete; may be taken as a function of time.

Autogenous shrinkage is influenced by many factors including the water-cementitious material ratio, type of cement, type and replacement rate of admixture, and temperature. If a concrete mix containing a large quantity of fine blast-furnace slag is used, autogenous shrinkage may become un-ignorably large relative to volume changes due to temperature changes even when the cement content is low as in the case of mass concrete. Since autogenous shrinkage can also be influenced by the use or nonuse of spray curing, it is important to take the effect of the construction method into consideration.

At present, it is still difficult to predict strains due to autogenous shrinkage, taking all of these material- and construction-related factors into consideration. A choice is permitted, therefore, between a laboratory test and an estimation formula depending on the design and construction requirements. If an estimation formula is used, it is recommended to use a formula that allows for cement composition, concrete mix proportions and concrete temperature.

In the case of mass concrete, drying shrinkage occurs only near the concrete surface exposed to the air, so Eq. (C3.2.1) does not allow for drying shrinkage. When the minimum member size is smaller than about 400 mm, temperature changes, autogenous shrinkage and drying shrinkage, which occurs later than the first two, may become too great to ignore. In such cases, it is desirable to consider the effect of drying shrinkage as well as autogenous shrinkage.

3.3 External Restraining Body

The part of the external restraining body to be considered shall fully cover the part where stresses due to volume changes of the restraining body cannot be ignored. Such extent shall, in general, be verified in advance by calculation. In case that the external restraining body is hardened concrete and rock, restraining effect may be considered assuming that there is no slip between the restraining body and concrete.

[Commentary] The part of the external restraining body to be considered is difficult to define because it is somewhat dependent on the calculation method. Usually, the part to be considered is two to five times as large as the restrained body in the horizontal and vertical directions, and the size requirements are dependent on the stiffness ratio between the concrete and the restraining body. In principle, these conditions require a three-dimensional stress analysis of the entire system including the restraining body, but in reality it is often cumbersome and not easy to do. The recommended practice, therefore, is to use the CL method or the CP method as a means of analyzing this kind of restraining effect easily and accurately.

Even when the external restraining body in contact with the structure is highly rigid, deformation of the restraining body and displacement along the interface may result. This boundary characteristic has not yet been accurately modeled, but because it usually reduces autogenous stresses, a general rule is to assume perfect restraint (i.e., no displacement).

3.4 Effect of Restraining by Reinforcement

In cases where the restraining effect of reinforcing steel cannot be ignored, stresses shall be calculated assuming that deformation of reinforcing steel and concrete at the same location are identical.

[Commentary] Usually, concrete and reinforcing steel have similar coefficients of thermal expansion. Therefore, in cases where volume change due to temperature is the only concern, the restraining effect of the reinforcing steel on the concrete can be ignored. However, both autogenous shrinkage and drying shrinkage are restrained by reinforcing steel, if a large amount of steel is used, additional tensile stresses that occur when concrete deformation of these types is restrained by steel are allowed for.

CHAPTER 4 MATERIAL PROPERTIES

4.1 Mechanical Properties

4.1.1 Tensile strength of concrete

The tensile strength of concrete to be used for the calculation of the cracking index shall, in principle, be determined by splitting tensile strength test using specimens.

[Commentary] Because of such factors as the degree of drying, loading rate and differences in size, tensile strengths of concrete in structures, differing from that obtained through tension tests using small and wet test specimens, are greatly influenced by the manufacturing inconsistency and construction performance. Manufacturing inconsistency was taken into account when the characteristic compressive strength of concrete was determined. Other influences are considered indirectly by introducing a factor of safety (γ_{cr}) associated with the probability of occurrence of cracks. Therefore, the splitting tensile strength of test specimens in accordance with JIS A 1113 may be taken as the tensile strength to be used for the calculation of the cracking index.

In general, development of the tensile strength with respect to concrete age can be estimated by the compressive strength. Examples are shown in Eq. (C4.1.1) and Eq. (C4.1.2). On the right-hand side of Eq. (C4.1.1), f'_{ck} is indicated as characteristic compressive strength, but for such purposes as post-completion reviews, experimentally determined or other compressive strength values, if available, may be used instead.

The reason why the relation between tensile strength and compressive strength indicated here is not identical to the relation indicated in the Standard Specification for "Design: General Requirement" is that accuracy, particularly for young concrete, is pursued here. Although these equations have been proved to be reliable for ordinary strength concrete, formulae for high-strength concrete with characteristic compressive strengths of 50 N/mm^2 or more need to be determined separately by referring to test data and other information.

$$f'_c(t) = \frac{t}{a+bt} d(i) f'_{ck} \qquad \text{(C4.1.1)}$$

$$f_{tk}(t) = c\sqrt{f'_c(t)} \qquad \text{(C4.1.2)}$$

where,

$f'_c(t)$: compressive strength of concrete at an age of t days (N/mm^2)

$f_{tk}(t)$: tensile strength of concrete at an age of t days (N/mm^2)

f'_{ck} : design strength of concrete (N/mm^2)

t : age, in day

i: reference age for characteristic compressive strength (in days; i = 28 or 91)

a, b: values in Table C4.1.1 are used in standard practice, though they vary with type of cement. Where blast furnace slag cement B is used, moderate the values equivalent to those for heat portland cement may be used. No constants can be given for fly ash cement B because only insufficient data have been accumulated. Values should be specified based on the past records.

c: taken as 0.44 in standard practice, though it varies with such factors as degree of drying of concrete.

d: increasing ratio of compressive strength of 91days against that of 28days. Standard values of d(28) is shown in Table C4.1.1. *d*(91)=1 regardless of the type of cement.

Table C4.1.1 Values of *a,b,d* in Eq.(C4.1.1)

Type of Cement	*a*	*b*	*d*(28)
Ordinary Portland Cement	4.5	0.95	1.11
Moderate Heat Portland Cement	6.2	0.93	1.15
High Early Strength Portland Cement	2.9	0.97	1.07

4.1.2 Young's modulus of concrete

The effective Young's modulus of concrete to be used in calculating thermal stresses shall be given with proper consideration for the effect of the age and drying.

[Commentary] Since the magnitude of thermal stress is determined by the temperature gradient in the structure, the ratio of stiffness between the restraining body and the restrained body and irreversible stiffness changes of concrete, it is necessary to evaluate the mechanical properties and volume changes of hardening concrete of the restrained body by sequential stress analysis. As a method for equally dealing with stiffness changes due to hydration and stress relaxation due to creep, the method of calculating thermal stresses by using effective Young's modulus, which is defined by extending the average Young's modulus for the cross section of the restrained concrete to allow for stiffness reduction due to creep, relaxation, etc., was proposed. Young's modulus of concrete of the restrained body gradually increases with age, which is also influenced by the mix proportion. On the other hand, the stress relaxation of the concrete due to its creep or relaxation phenomenon is observed much remarkable at early ages.

According to the results of previous studies, when load is applied at an age of 2 days, the creep coefficient becomes 1.5-2.0 times greater than that when the load is applies at an age of 28 days. Where the temperature of concrete is kept high, the creep coefficient becomes larger, too. Because of these, it is common practice to simply estimate Young's modulus from the compressive strength determined according to maturity and reduce Young's modulus in consideration of creep or relaxation at the corresponding age. In this case, compressive and tensile creep characteristics are identical.

When a simplified method to estimate the effective Young's modulus is needed, Eq. (C4.1.3) can be used. However, since creep characteristics of concrete with characteristic compressive strengths exceeding 50 N/mm^2 are not well clarified yet, they should be determined through creep test.

$$E_e(t) = \Phi(t) x 4.7 x 10^3 \sqrt{f'_c(t)} \qquad \text{(C4.1.3)}$$

where, $E_e(t)$: effective Young's modulus at the age of t days, in N/mm^2,

ϕ (t): compensating factor taking account of creep during concrete temperature increasing

for up to 3 days: ϕ =0.73

for after 5 days: ϕ =1.0

from 3 days to 5 days: linear interpolation can be used.

$f'_c(t)$: estimated compressive strength of concrete at an age of t days using Eq. (C4.1.2), in N/mm^2.

In cases where the effect of stress changes due to temperature changes, autogenous shrinkage or drying shrinkage cannot be ignored, it is desirable to determine the concrete stress $\sigma(t_i)$ by calculating creep strains corresponding to changing stresses. A simpler method is to calculate concrete stress from Eq. (C4.1.4), using the effective Young's modulus $E_e(t_i, t_j)$ corresponding to the effective strain increment $\Delta\varepsilon_{ef}(t_j)$ at age t_j at the time of loading. The effective Young's modulus $E_e(t_i, t_j)$ may be taken as Eq. (C4.1.5), in which different creep coefficients are used according to the age of concrete at the time of loading.

$$\sigma(t_j) = \Sigma E_e(t_i, t_j) \times \Delta\varepsilon_{ef}(t_j) \qquad \text{(C4.1.4)}$$

$$E_e(t_i, t_j) = \frac{E_c(t_j)}{1 + \{E_c(t_j)/E_c\}\phi(t_i, t_j)} \qquad \text{(C4.1.5)}$$

where, $\sigma(t_i)$: concrete stress at age t_i

$\Delta\varepsilon_{ef}(t_j)$: effective strain increment at age t_j

$E_e(t_i, t_j)$: effective Young's modulus at t_i of concrete loaded at age of t_j

E_c : Young's modulus of concrete at age of 28 days (standard curing period)

$\phi(t_i, t_j)$: creep coefficient calculated from E_c at t_i of concrete loaded at age of t_j

4.2 Thermal Properties

4.2.1 Thermal Properties of Concrete

The adiabatic temperature rise of concrete shall be adequately determined considering such factors as the materials and mix proportion of concrete used and concrete temperature at placing. The thermal properties to be used in thermal analysis, i.e., thermal conductivity, thermal diffusivity and specific heat, shall be adequately determined considering the mix proportion of concrete used.

[Commentary] The design value of adiabatic temperature rise of concrete is expressed by Eq. (C4.2.1) unless either chemical admixtures or admixtures that are highly effective for retarding hydration are used.

$$Q(t) = Q_\infty\left(1 - e^{-rt}\right) \qquad \text{(C4.2.1)}$$

where,

Q_∞ : ultimate adiabatic temperature rise to be determined by testing,

r : constant for rate of temperature rise to be determined by testing,

t : age (days),

$Q(t)$: adiabatic temperature rise at an age of t (ºC).

If the type of cement, cement content and the temperature at the time of placement are given, Q_∞ and r may be estimated by the regression equations shown in Table C4.2.1 as examples. The committee for revising the "Cracking Control Guidelines for Mass Concrete" of the Japan Concrete Institute proposes Eq. (C4.2.2) to express the characteristics of adiabatic temperature rise more faithfully.

$$Q(t) = Q_\infty\left(1 - e^{-r(t-t_0)^s}\right) \qquad \text{(C4.2.2)}$$

where, t_0 and s are the base and the speed of temperature rise, which are introduced to specify the equation expressing the proposed adiabatic temperature rise as a regression equation based on the existing data, and vary according to the temperature at the time of placement. The value of s is 1 for cement other than low-heat Portland cement. The equation gives Q_∞, r, t_0 and s (in the case of low-heat Portland cement only) according to the temperature during placement for a cement content of 250 kg/m^3 to 450 kg/m^3 or 450 kg/m^3 to 700 kg/m^3. These should be referred to as required.

Table C4.2.1 Standard values of Q_∞ and r in Eq. (C4.2.1)

Type of cement	Temperature during placement	$Q_{(t)} = Q_\infty\left(1-e^{-rt}\right)$			
		$Q_\infty = aC + b$ 1)		$r = gC + h$ 1)	
		a	b	g	h
Ordinary Portland cement	10	0.12	11.0	0.0015	0.135
	20	0.11	13.0	0.0038	-0.036
	30	0.11	12.0	0.0040	0.337
Moderate-heat Portland cement	10	0.11	6.0	0.0003	0.303
	20	0.10	9.0	0.0015	0.279
	30	0.11	9.0	0.0021	0.299
High early strength Portland cement	10	0.13	15.0	0.0016	0.478
	20	0.13	12.0	0.0025	0.650
	30	0.13	10.0	0.0014	1.720
Low-heat Portland cement	10	0.11	4.2	0.0006	0.105
	20	0.10	8.0	0.0012	0.071
	30	0.10	9.4	0.0019	0.055
Blast furnace slag cement type 2)	10	0.13	13.2	0.0013	0.034
	20	0.13	11.9	0.0018	0.148
	30	0.13	10.9	0.0023	0.396
Fly ash cement type 3)	10	0.15	3.7	0.0011	0.107
	20	0.14	4.5	0.0019	0.213
	30	0.14	4.5	0.0030	0.487

1) C: Cement content (kg/m^3)
2) In cases with 40% of blast furnace slag (Blaine fineness: 4200 cm^2/g). For other percentages, refer to existing data or conduct testing.
3) The percentage of fly ash is 18%.

The thermal properties of cement paste vary depending on the progress of hydration and the water content. The thermal properties of concrete, however, may be assumed to be constant because the thermal properties of aggregate, which accounts for a major part of concrete volume, are constant. Thermal constants of concrete generally depend on the mix proportion of concrete, especially on such factors as the property and unit content of aggregate, and the moisture content of concrete. Therefore, the constants are recommended to be determined considering the effects of these factors.

For concrete to be used in ordinary concrete structures, the thermal conductivity, specific heat and thermal diffusivity may be taken as 2.6-2.8 W/mºC, 1.05-1.26 kJ/kgºC and $0.83\text{-}1.1\times10^{-6}$ m^2/s, respectively. Once the thermal conductivity and density are determined, the other constants can be estimated by the following equations.

$$h^2_c=3.34\times10^{-7}\lambda_c \tag{C4.2.3}$$

$$C_c=3.03\times10^{3}/\rho \tag{C4.2.4}$$

4.2.2 Thermal properties of soil and rock

In cases where thermal analysis requires the thermal properties of soil, rock or other materials, such properties shall be determined conservatively by, for example, referring to available data.

[Commentary] In general, mass density, thermal conductivity and specific heat of rock may be taken as 2600-2700 kg/m^3, 1.7-5.2 W/m℃ and 0.71-0.88 kJ/kgºC, respectively.

PART 5　GENERAL STRUCTURAL DETAILS

CHAPTER 1　GENERAL

1.1　Scope

This Specification presents details of reinforcement arrangement that meets the requirements in Chapter 13 of "Design: General Requirements."

[Commentary]　The details of arrangement of reinforcement in structures are important prerequisites for the performance verification.　The data validity on the reinforcement arrangement is generally related only to part of the longitudinal reinforcement and shear reinforcement in the study cross section.　At present, no performance can be verified directly that is related to numerous reinforcing bars details.

This Specification shows a standard method of reinforcement arrangement that meets the requirements in Chapter 13 of “Design: General Requirements.”　Complying with this Specification therefore is expected to meet the requirements stipulated in the related sections of Chapter 13 of “Design: General Requirements.”　The details of reinforcement arrangement in members and structures are described in Chapters 7 through 13 of this Specification in cases where the design methods based on the linear analysis shown in Part 1 of “Design: Standards Methods” are applied.

CHAPTER 2 CONCRETE COVER

(1) The minimum depth of concrete cover shall be the larger of the diameter of reinforcement "and" the concrete cover that meets the requirements in Sections 8.3 and 10.7 of "Design: General Requirements" plus a margin of construction error. This concrete cover shall be clearly presented in design drawings. For the regions requiring no fire resistance, it is not necessary to meet the requirements in Section 10.7 of "Design: General Requirements."

(2) The margin of construction error should be specified considering the construction conditions for members or regions and the construction management system for reinforcing bar works.

(3) The concrete cover that meets the requirements in Section 8.3 of "Design: General Requirements" may be calculated in accordance with "Design: Standards Part 3."

(4) For footings or important members of structures placed directly against earth, it is recommended that the concrete cover should not be less than 75 mm.

(5) For reinforced concrete placed in water, without using antiwashout underwater concrete, it is recommended that the concrete cover should not be less than 100 mm.

(6) In cases that concrete is likely to be subject to abrasion by running water, etc. it is recommended that the concrete cover should be appropriately increased.

[Commentary] (1) For determining the minimum concrete cover, the larger of the diameter of reinforcement steel and the concrete cover that meets the durability requirement in which no fire resistance is required, plus a margin of construction error shall be taken into consideration, as shown in Fig. C2.1. The concrete cover here means the distance from the outermost steel to concrete surface. If joints are used for the reinforcement, the distance from the outermost edge of the joint zone should be defined as the concrete cover. In the case of deformed reinforcement, the concrete cover may be calculated regarding a nominal diameter of the deformed reinforcement as the diameter of the reinforcement. In order to secure bond strength, the distance to concrete surface should not be shorter than the sum of the diameter of the reinforcement and a margin of construction error for all the reinforcement.

It is important that the actual concrete cover after construction is not smaller than the larger of the diameter of the reinforcement and the concrete cover that meets the durability requirement. The margin of construction error should be properly specified considering the actual construction conditions. The design drawings should clearly present the concrete cover and the margin of construction error for each region.

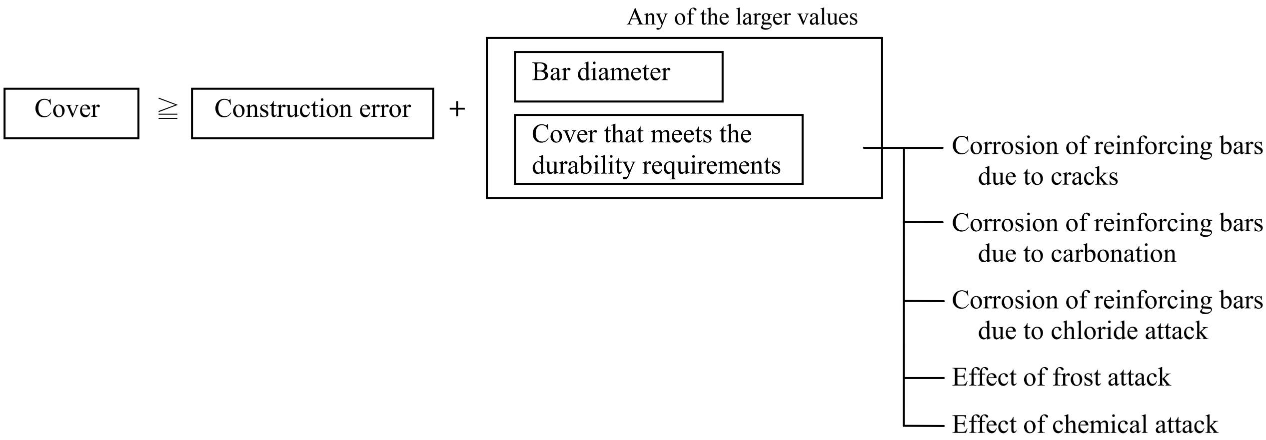


Fig. C2.1 Calculation of concrete cover (in cases where no fire resistance is required)

When a group of parallel deformed bars act as a unit and are bundled in contact, the unit may be treated as a single bar having an equivalent diameter derived from the equivalent total area, for the purpose of determination of cover requirements (Fig. C2.2).

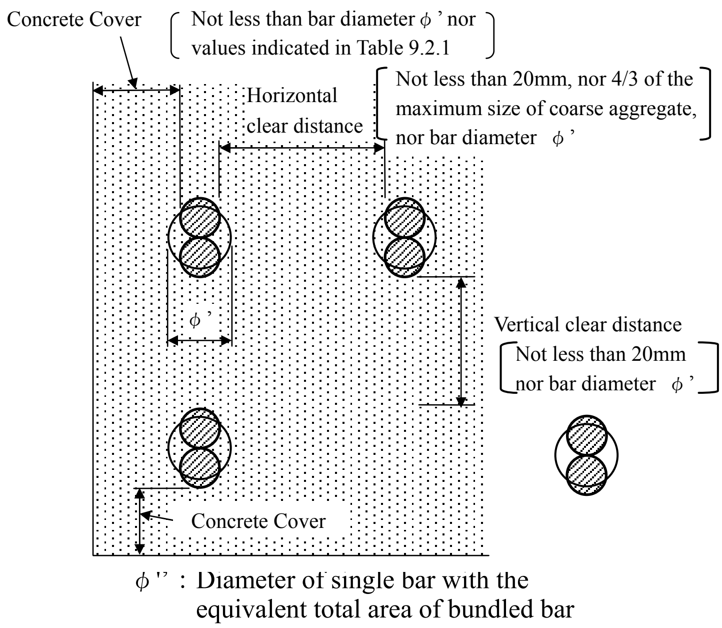


Fig. C2.2 Concrete cover and clear distance for bundled bars

(2) A margin of construction error should be taken into consideration when specifying the concrete cover because the construction error occurs at the time of reinforcement assembly.

The margin of construction error should be specified for each region because the method of assembly varies from region to region. The margin of construction error is also determined by the management system or management values for reinforcing bar works during construction. These points should be taken into consideration when specifying a margin of construction error. The margins of construction error based on the results of surveys of margins of construction error and the construction management systems are shown in "Design: Standards Part 3."

(5) Because of the difficulty in ensuring sufficient compaction, and the possibility of concrete not being able to reach all the narrow spaces between the formwork and the bars, and the fact that examination of the condition of concrete placed in water is difficult, the concrete cover in cases

when concrete is placed under water without using antiwashout underwater concrete shall not be less than 100mm to ensure safety of the structure. In the case of cast-in-situ concrete piles considering the ease of placing, irregularity of the inside face of drilled earth, and installation of cages, it is recommended that concrete cover be around 150 mm.

(6) Where concrete is subjected to abrasion, for example upper side of a slab without effective protection, it is recommended that the concrete cover be increased by at least 10 mm compared to the standard value determined in accordance with the provisions of this section, in order to ensure that design strength is not decreased even after abrasion.

CHAPTER 3 CLEAR DISTANCE

3.1 Clear Distance

(1) Lateral clear distance between longitudinal reinforcements in beams shall neither be less than 20 mm, nor less than 4/3 times the maximum size of coarse aggregate, nor less than the diameter of the bar. The lateral clear distance shall also be such that an internal vibrator can be inserted for compacting the concrete.

Vertical clear distance for longitudinal reinforcement arranged in two or more layers should neither be less than 20 mm nor less than the diameter of the bar (See Fig. 3.1).

(2) Clear distance for longitudinal reinforcement in columns shall neither be less than 40 mm, nor less than 4/3 times the maximum size of coarse aggregate, nor less than one and a half times the diameter of the bar.

(3) In cases when the diameter of the deformed bars used is less than 32 mm and the arrangement of bars is complicated to the extent that sufficient compaction of concrete may not be possible, two longitudinal bars in members such as beams, slabs, etc. may be bundled vertically and two or three vertical longitudinal bars in members such as columns, walls, etc. may be bundled (See Fig. 3.2).

(4) The spacing of reinforcement should be verified whether the requirements for concrete construction performance are met or not.

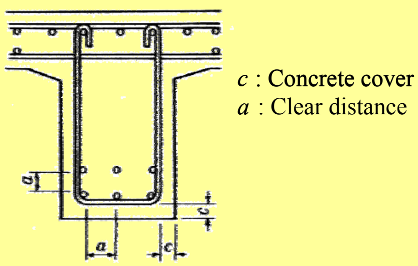


Fig. 3.1 Clear distance and concrete cover

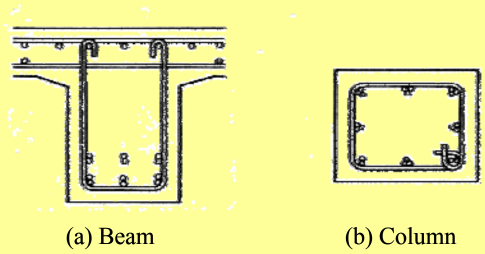


Fig. 3.2 Bundled bars

[Commentary] <u>(1)</u> In order to ensure that concrete surrounds the tensile reinforcement appropriately a minimum lateral clear distance for longitudinal reinforcement in beams is prescribed here to be neither less than 20 mm, nor less than 4/3 times the maximum size of coarse aggregate. Considering the site-conditions, the design clear distance should be greater than the values specified here by an appropriate margin. Experience has shown that sufficient bond develops in cases when the lateral clear distance is about equal to the diameter of the bar, provided the tensile reinforcement is well covered by the concrete. The lateral clear distance shall also be such that an internal vibrator can be inserted for compacting the concrete. Clear distance for splices in reinforcement may be provided in Section 13.7 (4) in "Design: General Requirements."

<u>(2)</u> Clear distance for longitudinal reinforcements in columns has been specified in a manner similar to that for beams, though the values are somewhat greater than those in the case of beams since it may be more difficult to place concrete in columns.

(3) Placing and compacting concrete using an internal vibrator may become difficult in cases when the reinforcement ratio of primary horizontal reinforcement in beams, slabs, etc. and vertical reinforcement in columns, walls, etc. is high. In such cases, two or three bars may be bundled for ease of placement and compaction of concrete, to prevent segregation in concrete, and to produce dense concrete.

Since it may be difficult to place and compact concrete if the bundled bars come apart or move, it is important to tie the bars securely and use suitable spacers to keep the bundled bars in position during construction. Since bond with concrete decreases in the case of bundled round bars, only deformed bars can be bundled.

When bars are bundled, there may be cases when mortar does not adequately surround the reinforcement unless the concrete is sufficiently compacted. It is, therefore, necessary to carry out the construction carefully, and it is recommended that the clear distance between reinforcement be widened to an extent for inserting internal vibrators.

(4) At the time of design, the verification should be made whether the requirements for concrete construction performance are met or not using the slump assumed based on the concrete mix proportions. For the relationship between the spacing of reinforcement and the concrete slump, refer to Chapter 4 of "Construction: Standard."

3.2 Clear Distance of Prestressing Tendon

(1) Clear distance shall be provided between tendons or between sheaths, or between the tendon or sheath and the reinforcement to ensure that concrete adequately surrounds the tendons and sheaths and that concrete is fully compacted.

(2) Clear distance between sheaths for post-tensioning tendons should satisfy the following requirements:

(i) Horizontal and vertical clear distances between sheaths should not be less than 4/3 times the maximum size of the coarse aggregate.

(ii) In areas where an internal vibrator is to be inserted, the horizontal clear distance of sheaths or groups of sheaths shall be the space necessary to be able to insert the internal vibrator.

(iii) Small size sheaths may be arranged in contact with each other in cases it is unavoidable. Even in such cases, in vertical direction, not more than two sheaths shall be allowed to be in contact.

(iv) When the sheaths in contact with each other are curved, the vertical clear distance between the sheaths or groups of sheaths in the bend portion should not be less than the diameter of a sheath.

(v) The horizontal clear distance between the longitudinal reinforcement and sheath should neither be less than 20 mm, nor less than 4/3 times the maximum size of aggregate, nor less than the diameter of the bar.

(vi) The vertical clear distance between the longitudinal reinforcement and sheath should neither be less than 20 mm nor less than the diameter of the bar.

(3) Clear distance between sheaths for pre-tensioning tendons should satisfy the following requirements:

(i) The horizontal and vertical clear distances between prestressing tendons at the end of a member should not be less than 3 times the diameter of the tendon. Furthermore, the horizontal clear distance should not be less than 4/3 times the maximum size of the coarse aggregate.

(ii) When the prestressing tendons are bundled in a region other than the member end, the numbers of the layers and the tendons in a group shall not exceed 2 layers and 4 tendons, respectively, and the clear distance between such groups shall not be less than 4/3 times the maximum size of the coarse aggregate.

[Commentary] (2) (i) and (ii) The horizontal clear distance should be provided to insert an internal vibrator to ensure that concrete adequately surrounds the tendon or sheath and is fully compacted. For compacting concrete, an internal vibrator with a diameter of approximately 50 mm is generally used. Then, the horizontal clear distance of sheaths should not be less than 60 mm.

(iii) and (iv) Allocating numerous sheaths in contact with one another should be avoided inasmuch as possible. In cases where sheaths need to be arranged in contact with one to provide space for inserting an internal vibrator because of a limited member thickness, sheath groups may be used in which two layers of small sheaths are placed only after special considerations. Herein the small sheath indicates the one of which the diameter is less than around 70 mm. In addition, the special considerations indicate the ones with regard to the cross-sectional dimensions for calculating the stress, the interval of the bend up locations, the concrete quality and the placing procedures, etc.

When sheaths are curved, the concrete between the sheaths needs to resist bearing stress of tendons acting on the sheath wall surface. If the clear distance is small in cases where no grout is injected into the sheath at the bend up location in the direction of the bearing stress, as illustrated in Fig. C3.1(a), the concrete or sheath may be damaged. In practice, as mentioned in the clear distance between the sheaths at the bend up location should not be less than the diameter of a sheath. (see Fig. C3.1(b))

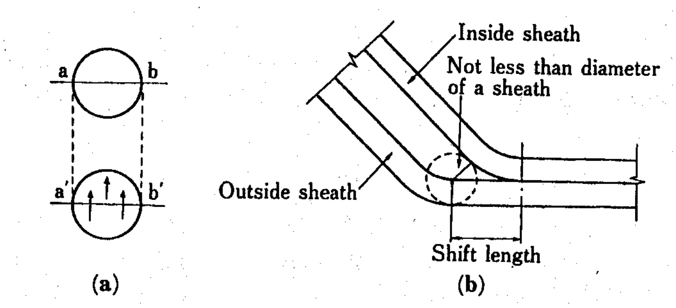


Fig. C3.1 Arrangement of curved sheaths

(3) In the pre-tensioned concrete structure, a large bond stress acts along the length of the prestressing tendons especially at the end of the concrete member. Therefore, adequate clear distance should be provided to ensure sufficient compaction of concrete, which facilitates development of adequate bond.

CHAPTER 4 BEND CONFIGURATIONS OF REINFORCEMENT

(1) Inside radius of bend for bent bars shall be not less than 5 times the diameter of the bar (see Fig. 4.1). When reinforcing bars within a distance of 2Φ+20 mm from the surface of concrete member are used as bent bars, the inside radius of bend of such reinforcement shall not be less than 7.5 times the diameter of the bar.

(2) Inside radius of bend for reinforcement along the outer side of a corner in a rigid frame shall not be less than 10 times the diameter of the bar diameter (see Fig. 4.2).

(3) Reinforcement along the inner side of a haunch or a corner in a rigid frame shall not be tensile reinforcement bent from a slab or a beam, but separate straight reinforcement provided along the inner side of a haunch (see Fig. 4.2).

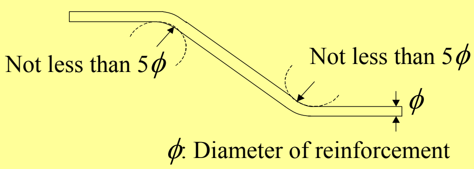


Fig. 4.1 Inside radius of bend for bent bar

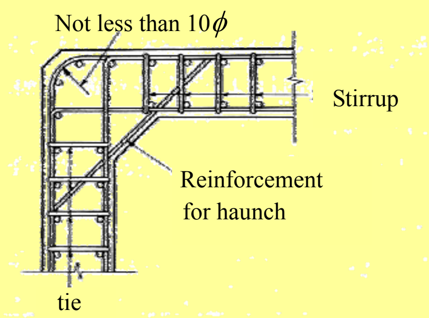


Fig. 4.2 Reinforcement along inner side of haunch corner in rigid frame, etc.

[Commentary] <u>(1)</u> Inner radius of the bend for bent bars should be determined such that it does not generate large bearing stresses in concrete. Where bend bars are within 2 ϕ +20 mm from the surface of a concrete member, the bearing strength of concrete around the bend points is decreased. Therefore, inner radius of the bend for bent bars should be greater to reduce the bearing stresses around the bend points. When high strength deformed bars are used, the inner radius of the bend for bent bar should be as large as possible.

<u>(3)</u> As shown in Fig. C4.1, in haunches or rigid frames it is possible for concrete to peel-off along the inner side, as a bent tension reinforcement tends to straighten under the action of tensile loads. Separate additional reinforcement should be provided in such cases, as per the provisions of this section. In cases when bending tensile reinforcement along the inner side of a haunch cannot be avoided, stirrups or ties should be provided at the bent.

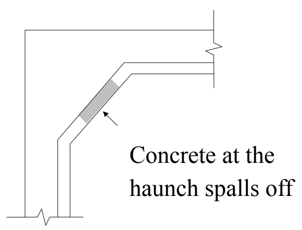


Fig. C4.1 Unfitting bar arrangement at haunch

CHAPTER 5 DEVELOPMENT OF REINFORCEMENT

5.1 General

If one of the following methods is used for providing anchorage at the end of reinforcing bars, the development length for reinforcing bars should be specified as described in this chapter.

(i) To embed reinforcing bars into concrete and develop through bond stress between the reinforcement and concrete.

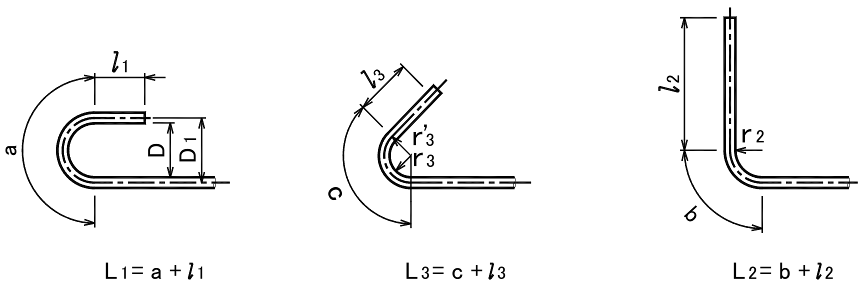
(ii) To embed reinforcing bars into concrete and provide an anchorage using standard hooks.

[Commentary] Standard hooks should be applied in accordance with Table C5.1. The standard hooks should be installed in the tension reinforcing bars of slabs and beams, longitudinal reinforcing bars of piles and longitudinal reinforcing bars arranged at the corners of columns. If adequate development length can be secured for the longitudinal reinforcing bars of piles, no hook should be installed.

The shapes of standard hooks are shown in Fig. C5.1. The dimensions of standard hooks for the longitudinal reinforcing bars are listed in Table C5.2 (a) and (b). The dimensions of standard hooks for stirrups and hoops are shown in Table C5.3 (a) and (b).

Table C5.1 Application of hooks

		Semicircular hook	Acute-angled hook	Right-angled hook	With no hooks
Ordinary round bar		○	×	×	×
Deformed bar	Longitudinal reinforcing bar	○	○	○	○
	Stirrup	○	○	○	×
	Hoop	○	○	×	×



Semicircular hook   Acute-angled hook   Right-angled hook

Fig. C5.1 Shapes of standard hooks[1]

Table C5.2 (a) Dimensions of standard hooks for longitudinal bar (SD295, SD345) (mm)

Bar diameter	Semicircular hook						Right-angled hook				
	D	D_1	a	l_1	L_1	$2L_1$	r_2	b	l_2	L_2	$2L_2$
D10	50	60	94	66	160	320	25	47	123	170	340
D13	70	83	130	70	200	400	35	66	164	230	460
D16	80	96	151	79	230	460	40	75	195	270	540
D19	100	119	187	83	270	540	50	94	236	330	660
D22	110	132	207	93	300	600	55	104	266	370	740
D25	130	155	243	107	350	700	65	123	307	430	860
D29	150	179	281	119	400	800	75	141	349	490	980
D32	160	192	302	128	430	860	80	151	389	540	1080

Table C5.2 (b) Dimensions of standard hooks for longitudinal bar (SD390) (mm)

Bar diameter	Semicircular hook						Right-angled hook				
	D	D_1	a	l_1	L_1	$2L_1$	r_2	b	l_2	L_2	$2L_2$
D10	60	70	110	60	170	340	30	55	125	180	360
D13	80	93	146	64	210	420	40	73	156	230	460
D16	100	116	182	68	250	500	50	91	199	290	580
D19	120	139	218	82	300	600	60	109	231	340	680
D22	140	162	254	96	350	700	70	127	273	400	800
D25	150	175	275	105	380	760	75	137	303	440	880
D29	180	209	328	122	450	900	90	164	356	520	1040
D32	200	232	364	136	500	1000	100	182	388	570	1140

Table C5.3 (a) Dimensions of standard hooks for stirrups and hoops (SD295, SD345) (mm)

Bar diameter	Semicircular hook						Acute-angled hook					
	D	D_1	a	l_1	L_1	$2L_1$	r_3	r'_3	c	l_3	L_3	$2L_3$
D10	40	50	79	61	140	280	20	25	59	61	120	240
D13	60	73	115	65	180	360	30	37	87	83	170	340
D16	70	86	135	65	200	400	35	43	101	99	200	400
D19	80	99	156	84	240	480	40	50	118	122	240	480
D22	90	112	176	94	270	540	45	56	132	138	270	540
D25	100	125	196	104	300	600	50	63	148	152	300	600
D29	120	149	234	116	350	700	60	75	177	183	360	720
D32	130	162	254	136	390	780	65	81	191	199	390	780

Table C5.3 (b) Dimensions of standard hooks for stirrups and hoops (SD390) (mm)

Bar diameter	Semicircular hook						Acute-angled hook					
	D	D_1	a	l_1	L_1	$2L_1$	r_3	r'_3	c	l_3	L_3	$2L_3$
D10	50	60	94	66	160	320	25	30	71	69	140	280
D13	70	83	130	70	200	400	35	42	99	81	180	360
D16	80	96	151	79	230	460	40	48	113	97	210	420
D19	100	119	187	83	270	540	50	60	141	119	260	520
D22	110	132	207	93	300	600	55	66	156	134	290	580
D25	130	155	243	107	350	700	65	78	184	156	340	680
D29	150	179	281	119	400	800	75	90	212	178	390	780
D32	160	192	302	128	430	860	80	96	226	194	420	840

Development lengths of reinforcing bars of various shapes are shown in Fig. C5.2. The development lengths in cases of mechanical anchorage are shown in Fig. C5.3. Examples for members with a shown in Section 13.6.3 (1) of "Design: General Requirements" are shown in Table C5.4 (a) and (b). Basic development lengths at a = 1 are shown in Table C5.4 (c). The basic development length of each member l_{do} can be obtained by multiplying the value of α shown in Table C5.4 (a) and (b) by the basic development length shown in Table C5.4 (c).

Mechanical anchorage should be applied in accordance with the "2007 Guidelines for Anchorage and Joints for Reinforcing Bars" prepared by the Japan Society of Civil Engineers. In cases where the anchorage is provided at regions that cannot be handled as massive concrete or only thin concrete covering is available in the anchorage zone in particular, whether the hook should be replaced with mechanical anchorage or should not be determined by conducting tests in which the dimensions and stress conditions in the anchorage zone are simulated.

If the descriptions in Section 13.6.3 (1) (III) of "Design: General Requirements" are applicable (see Fig. C5.4), the basic development length for reinforcement l_d should be set 1.3 times the development length obtained as stipulated in the section. Using concrete that induces no bleeding, however, the development length should not be corrected for the depth of concrete placement.

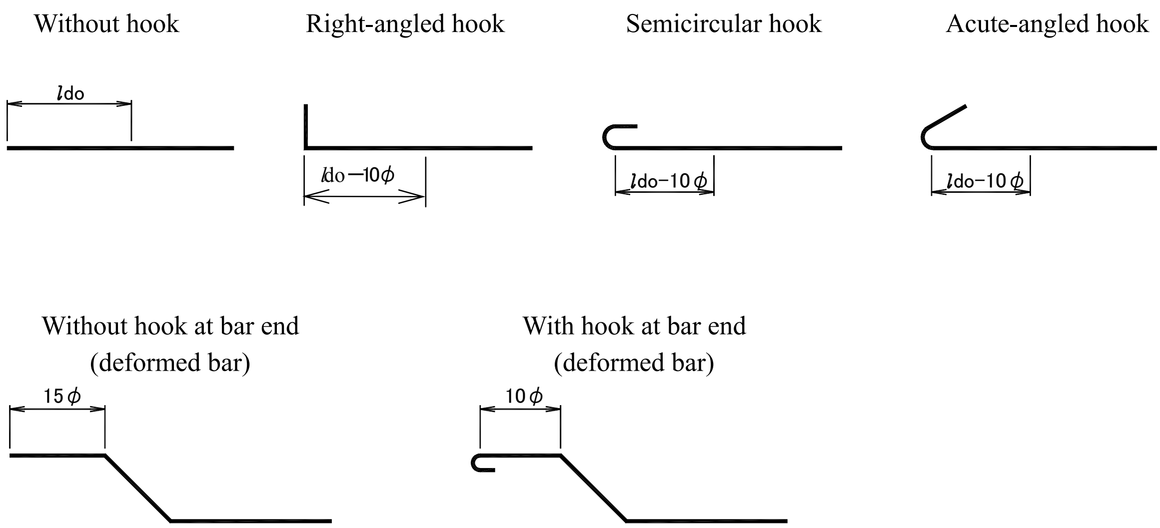


Fig. C5.2 Anchorage lengths for reinforcement of various shapes

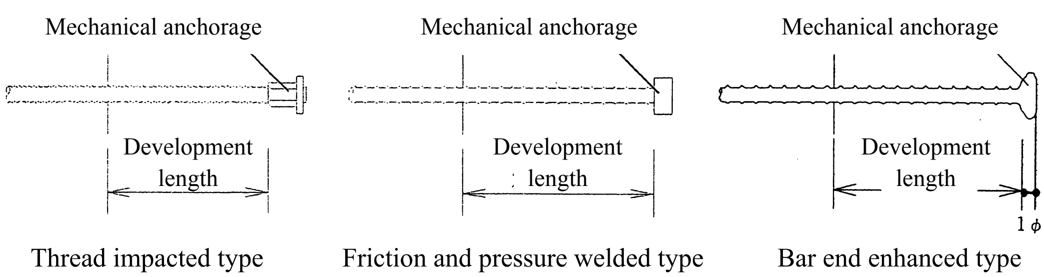


Fig. C5.3 Anchorage lengths in cases of mechanical anchorage

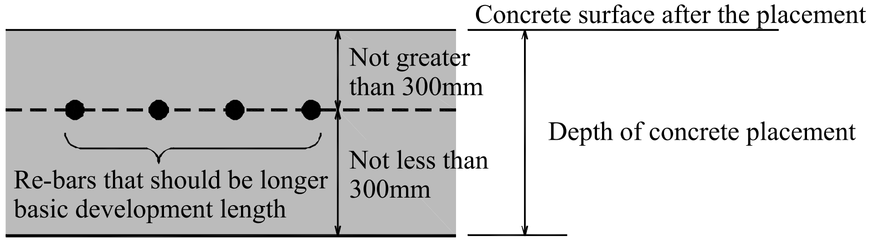


Fig. C5.4 Reinforcing bars with prolonged basic development length

Table C5.4 (a) Examples of α for beams, columns and bridge piers

		Longitudinal bar diameter					
		D16	D19	D22	D25	D29	D32
Concrete cover	20	0.8	0.9	0.9	0.9	1.0	1.0
	25	0.7	0.8	0.9	0.9	0.9	0.9
	30	0.7	0.8	0.8	0.9	0.9	0.9
	35	0.6	0.7	0.8	0.8	0.9	0.9
	40	0.6	0.7	0.7	0.8	0.8	0.9
	45	0.6	0.6	0.7	0.7	0.8	0.8
	50	0.6	0.6	0.6	0.7	0.7	0.8
	60	0.6	0.6	0.6	0.6	0.7	0.7
	70	0.6	0.6	0.6	0.6	0.6	0.7

*1 Transverse reinforcement should be D13. The spacing and clear distance of the longitudinal reinforcement should be 250 mm or less, and double of the concrete cover or larger, respectively.

Table C5.4 (b) Examples of α for slabs

		Longitudinal bar diameter					
		D16	D19	D22	D25	D29	D32
Concrete cover	20	0.9	0.9	1.0	1.0	1.0	1.0
	25	0.8	0.9	0.9	1.0	1.0	1.0
	30	0.8	0.8	0.9	0.9	0.9	1.0
	35	0.7	0.8	0.8	0.9	0.9	0.9
	40	0.6	0.7	0.8	0.8	0.9	0.9
	45	0.6	0.7	0.7	0.8	0.8	0.9
	50	0.6	0.6	0.7	0.8	0.8	0.8
	60	0.6	0.6	0.6	0.7	0.7	0.8
	70	0.6	0.6	0.6	0.6	0.7	0.7

*1 No transverse reinforcement is assumed to be used because the longitudinal reinforcement is located closest to the surface.

*2 Clear distance of longitudinal reinforcement should be double of the concrete cover or larger.

Table C5.4 (c) Basic anchorage length l_{d0} (mm) ($\alpha = 1.0$)

	Design strength of concrete f'_{ck} (N/mm^2)							
	21		24		27		30	
	SD345	SD390	SD345	SD390	SD345	SD390	SD345	SD390
D10	526	595	481	544	445	503	415	469
D13	684	773	626	707	578	654	539	610
D16	842	952	770	871	712	805	664	750
D19	1000	1130	914	1034	845	956	788	891
D22	1157	1308	1059	1197	979	1107	912	1031
D25	1315	1487	1203	1360	1112	1257	1037	1172
D29	1526	1725	1396	1578	1290	1459	1203	1360
D32	1684	1903	1540	1741	1424	1610	1327	1500

*1 $\gamma_c = 1.3$, $\gamma_s = 1.0$

Examples of the development lengths obtained based on the diameter of reinforcement to be developed, A_t/s and c_b are listed in Table C5.5. The type of reinforcing bar to be developed is SD345, and the design strength of concrete f'_{ck} is set to be 21 N/mm^2 or greater. In cases where SD390 is anchored, the development length in the table should be multiplied by 1.13. In cases where the design strength of concrete f'_{ck} is 24, 27 or 30 N/mm^2, the development length in the table should be divided by 1.1, 1.2 or 1.3, respectively.

Table C5.5 Calculated development length for reinforcing bars (ϕ *times*)

Bar diameter	A_t/s	c_b							
		10	20	30	40	50	60	70	80～
D10	2.0	32	32	32	32	32	32	32	32
	1.5	32	32	32	32	32	32	32	32
	1.0	32	32	32	32	32	32	32	32
	0.5	42	32	32	32	32	32	32	32
	0.0	47	37	32	32	32	32	32	32
D13	2.0	32	32	32	32	32	32	32	32
	1.5	32	32	32	32	32	32	32	32
	1.0	42	32	32	32	32	32	32	32
	0.5	47	37	32	32	32	32	32	32
	0.0	53	42	37	32	32	32	32	32
D16	2.0	32	32	32	32	32	32	32	32
	1.5	37	32	32	32	32	32	32	32
	1.0	42	37	32	32	32	32	32	32
	0.5	47	42	37	32	32	32	32	32
	0.0	53	47	42	32	32	32	32	32
D19	2.0	37	32	32	32	32	32	32	32
	1.5	42	37	32	32	32	32	32	32
	1.0	47	42	37	32	32	32	32	32
	0.5	53	47	42	32	32	32	32	32
	0.0	53	47	42	37	32	32	32	32
D22	2.0	42	37	32	32	32	32	32	32
	1.5	47	42	37	32	32	32	32	32
	1.0	47	42	37	32	32	32	32	32

	0.5	53	47	42	37	32	32	32	32
	0.0	53	53	47	42	37	32	32	32
D25	2.0	42	37	37	32	32	32	32	32
	1.5	47	42	37	32	32	32	32	32
	1.0	47	47	42	37	32	32	32	32
	0.5	53	47	42	42	37	32	32	32
	0.0	53	53	47	42	37	37	32	32
D29	2.5	42	42	37	32	32	32	32	32
	2.0	47	42	37	37	32	32	32	32
	1.5	47	47	42	37	32	32	32	32
	1.0	53	47	42	42	37	32	32	32
	0.5	53	53	47	42	42	37	32	32
	0.0	53	53	47	47	42	37	37	32
D32	2.5	47	42	37	37	32	32	32	32
	2.0	47	42	42	37	32	32	32	32
	1.5	47	47	42	42	37	32	32	32
	1.0	53	47	47	42	37	37	32	32
	0.5	53	53	47	47	42	37	37	32
	0.0	53	53	53	47	42	42	37	32

*1 The value c_b is of smaller among the concrete cover and 1/2 of the clear distance of the bars.
*2 $\gamma_c = 1.3$, $\gamma_s = 1.0$, *3 The value of c_b may be subjected to linear interpolation.

5.2 Development of Longitudinal Reinforcement

(1) At least 1/3 of the positive moment reinforcement in slabs or beams shall be extended beyond the support without bending and embedded.

(2) At least 1/3 of the reinforcement provided for negative moment in slabs or beams shall be extended beyond the point of inflection, and be either embedded in a compression zone, or continued to the neighboring reinforcement provided to resist negative moments.

(3) A bent bar should be extended as main reinforcement, or the end of a bent bar should be embedded in the compression zone with minimum concrete cover.

(4) Without leading to abrupt change in the amount of reinforcement, and taking l_s as the effective depth of cross-section, critical sections to check development length of the longitudinal tension reinforcement in flexural members shall be set as given below.

(i) The development length should not be shorter than the reduced anchorage length l_o measured from the point l_s away from the cross section where the bending moment is at the extreme.

(ii) When bending the reinforcing bar in the cross section where part of the reinforcing bar is not required from the calculation, the bar should be bent at the point l_s away in the direction toward the smaller bending moment from the cross section where part of the bar is not required to resist the calculated bending moment.

(iii) Development length for a bent bar anchored in the compression zone of concrete should not be less than 15ϕ for the cases when hooks are not provided, and 10ϕ for cases when the hooks are provided, where ϕ is the diameter of the bar.

(iv) In principle, the tensile reinforcement should be anchored in portions of concrete

not subjected to tensile stresses. However, it may be anchored in portions of concrete subjected to tensile stresses, provided the provisions (a) or (b) given below are satisfied. In such cases, the tensile reinforcement shall be extended by $(l_d + l_s)$ beyond the section where the bar is no longer required to resist calculated flexure, where l_d and l_s may be taken as the basic development length and the effective depth of the cross section in general, respectively.

(a) The design shear capacity between the point where the bar is terminated and the section where it is no longer required to resist calculated flexure shall not be less than 1.5 times the design shear force.

(b) The design moment capacity of continuing reinforcement at a point where adjacent reinforcement terminates shall not be less than 2 times the design moment and the design shear capacity between the point of bar cutoff and the section where the bar is no longer required to resist calculated flexure shall not be less 4/3 times the design shear force.

(v) Where positive moment reinforcement in a slab or beam is developed beyond the support at the end, the development length of the reinforcement shall not be less than the development length l_o for stress in reinforcement at a section which is at the distance of l_s, which may be taken as the effective depth, from the center of the support and shall be extended to the end of the member.

(vi) At the fixed end of a cantilever, development length should not be smaller than the reduced development length that should be from a point inside the anchorage zone by the smaller among 1/2 of the effective depth of the cross section and 10 times the bar diameter in cases where the end of the tensile reinforcing bar is confined horizontally in the anchorage zone. In cases where the end of the tensile reinforcement is not confined horizontally in the anchorage zone, development length should be from a point inside the anchorage zone by the effective depth of the cross section.

(vii) At the bottom end of a column, development length should not be smaller than the basic development length l_d that should be from a point inside the footing by the smaller among 1/2 of the effective depth of the column cross section and 10 times the bar diameter.

(viii) If the thickness or depth of the cross section in the member to be developed is smaller than that of the existing member, the reinforcing bar should be prolonged to the end of the member to be developed. Methods of the development of the reinforcement and methods of the arrangement of reinforcing bars at the connections of the member should be examined so that the anchorage zone may transmit the stress acting on the existing member to the member to be developed.

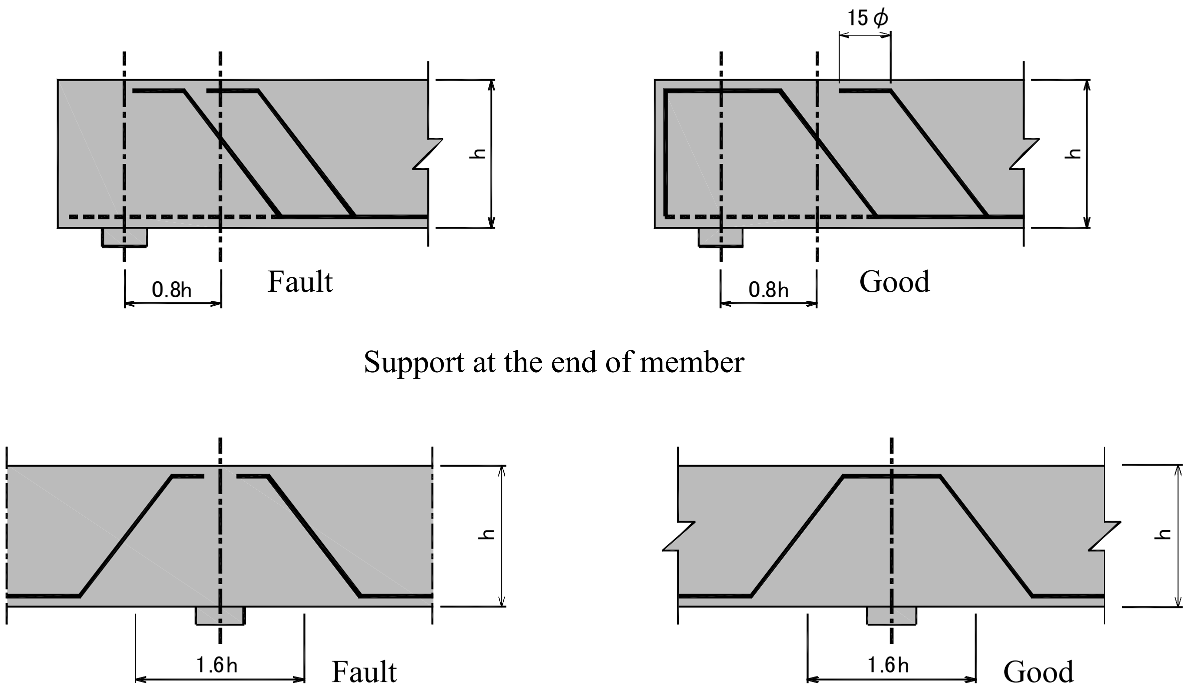
(ix) If the thickness or depth of the cross section in the member to be developed is much larger than that of the existing member, development length may be calculated by an appropriate method considering the characteristics of bond between the reinforcement and concrete, rather than complying with Section 13.6.3 of "Design: General Requirements."

(x) The l_s in members with variable depth of the cross section for Section 5.2 (4) (i) may be taken to be equal to the effective depth, d at the section of maximum moment, for

the Section 5.2 (4) (ii) it may be taken to be the effective depth, d at the section where the reinforcement is no longer required to resist calculated flexure.

[Commentary] (3) In cases such as continuous and fixed beams, the extension of a bent bar is used not only for anchoring the bent bar but also as reinforcement to resist positive or negative moments. When the extension of a bent bar is not used as positive or negative moment reinforcement, the end of bent bar should be embedded in the compression zone of concrete. However, when embedding in the tension zone is unavoidable, provisions of Section 5.2 (4) (iv) shall be satisfied.

Development length should not be less than 15ϕ in cases with no standard hooks that is embedded in the compression zone of concrete. Development length should not be less than 10ϕ in cases where standard hooks are provided. Since no stress or tensile stress, however, acts near the top fiber at the support of a beam, no anchorage should be provided within 0.8 h of the support at the end of members, or within 1.6 h around the support of the central portion of members (see Fig. C5.5).



Support at the central portion of member

Fig. C5.5 Examples of arrangement of bent reinforcing bars

(4) A sudden change in the longitudinal reinforcement content across a cross-section, such as in the portion where the bars may be cutoff, could lead to a sharp drop in the load carrying capacity, and render the member susceptible to failure at the section, even if the adequate development length is provided for. Thus, only a gradual reduction is recommended in cases when the longitudinal reinforcement content needs to be changed.

Figure C5.6 gives examples of critical sections for extension of reinforcement. Bars "A", "B", and "C" in Fig. C5.4 (a) respectively are illustrative the examples for the following cases:

Bar "A" indicates a tension reinforcement anchored in concrete subjected to tensile stresses. In such cases in accordance with the provisions of Section 5.2 (4) (i), since the maximum stress in Bar "A" on account of flexure occurs at the cross-section ‘1’, the tension reinforcement shall extend at least to point "a", which is at a distance l_s from the cross-section ‘1’. However, this requirement is bound to be satisfied as Section 5.2 (4) (iv) requires that the reinforcement be extended up to the point marked "c", which is at a distance of more than ($l_d + l_s$) from the cross-section ‘2’, where the Bar "A" is no longer required to resist calculated flexure.

Bar "B" indicates a bent bar. According to Section 5.2 (4) (ii) a point "d" should be at a distance of at least l_s from the cross-section 3, where the Bar "B" is no longer required to resist calculated flexure. In addition, according to Section 5.2 (4) (ii) the bent point "d" should be at the distance of l_o from the point "b" which is the point of maximum stress in the Bar "B". However, these two requirements are usually satisfied in many cases, and therefore further examination may be unnecessary. Bent deformed reinforcing bars should be anchored as shown in Fig. C5.7.

Bar "C" is an example of tension reinforcement anchored beyond the support. In such cases, the development length shall be neither less than $l_{o③}$, between point "f" (the terminal point to the Bar "C") and the point "d" (the point of maximum stress in the Bar "C"), nor, be less than $l_{o④}$ in accordance with Section 5.2 (4) (v) between the point "f" and point "e".

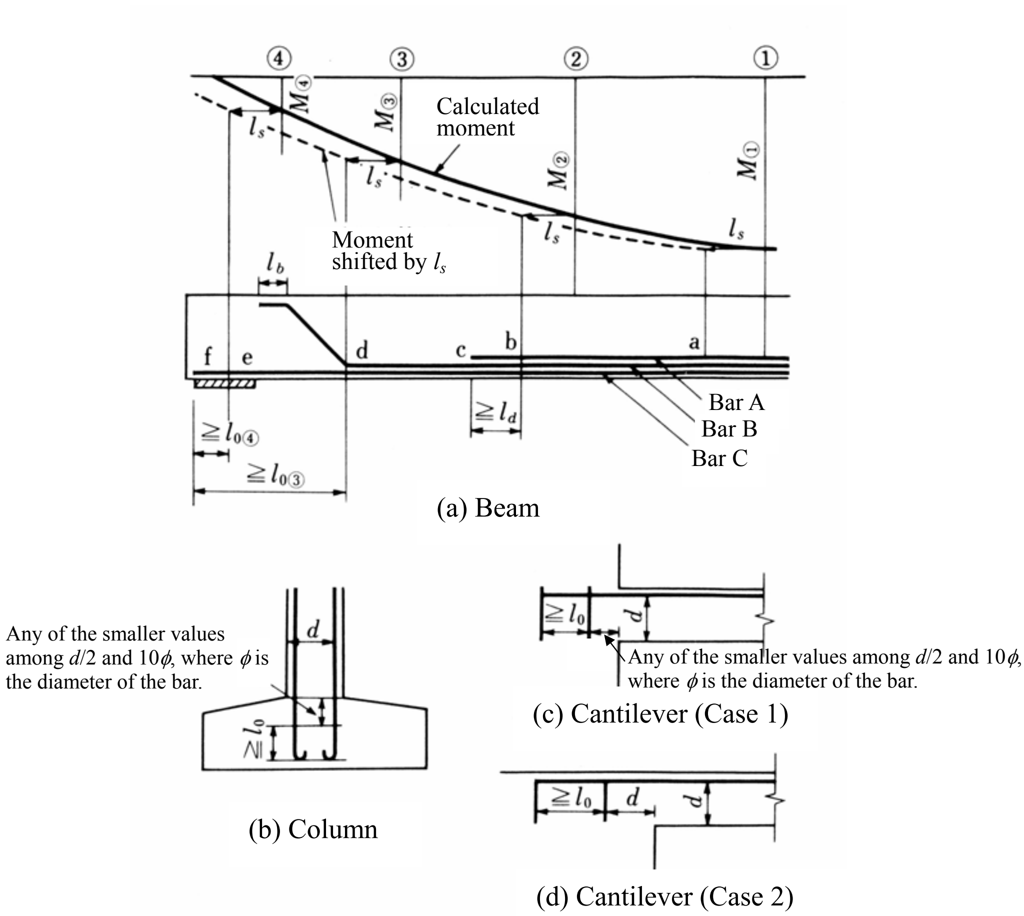


Fig. C5.6 Examples of critical sections for development of reinforcement

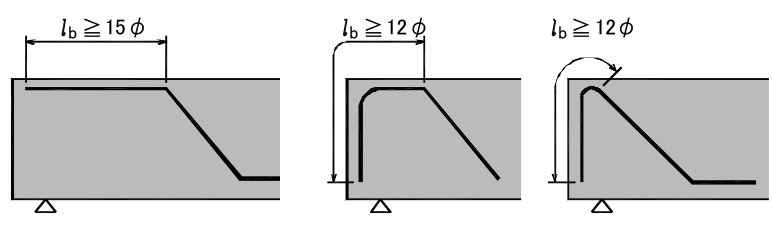


Fig. C5.7 Examples of arrangement of bent reinforcing bars[1)]

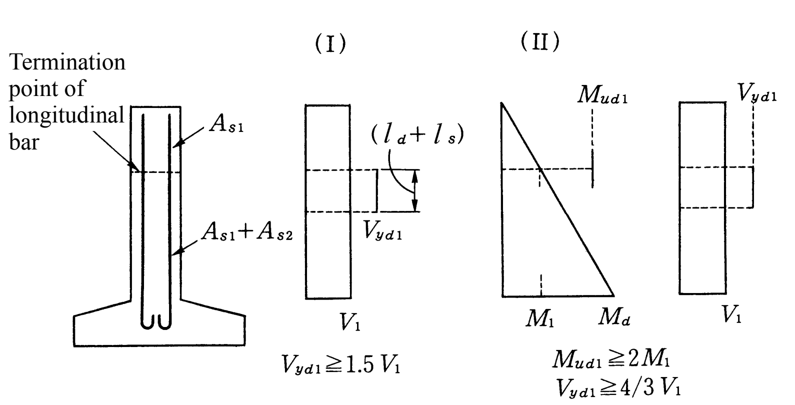
Where, '1' : Cross section corresponding to the peak bending moment
'2' : Cross section where Bar "A" is no longer required as per calculations.
'3' : Cross section where Bar "B" is no longer required as per calculations.
$l_{o(n)}$: Reduced development length according to the stress in the reinforcement in section "n".
l_b : Development length of bent bar

The l_s in member with variable depth for Section 5.2 (4) (i) may be taken to be equal to the effective depth, d at the section of maximum moment, for the Section 5.2 (4) (ii) it may be taken to be the effective depth, d at the section where the reinforcement is no longer required to resist calculated flexure.

(iv) In cases when the tension reinforcement is anchored in concrete subjected to tensile stresses, the reinforcement should extend beyond the section where it is no longer required to resist calculated flexure in accordance with the provision in Section 5.2 (4) (ii), by an amount equal to l_s, and embedded there. In order to prevent harmful cracks caused at the end of the bar, provisions (a) and (b) in this section shall be satisfied in the region between the end of the bar and the cross section where the bar is no longer required to resist calculated flexure. Provisions (i) and (ii) in this section have been laid down on the basis of experimental results.

In cases when a member with reinforcing bars anchored at an intermediate point is subjected to cyclic loads, the flexure capacity becomes discontinuous at the location where the reinforcement is anchored. Flexural cracks that often occur at that location may grow into diagonal cracks. If such a member undergoes cyclic loads with displacements in excess of levels that cause the reinforcement to yield, the load carrying capacity may be significantly reduced. In cases when the displacement exceeds twice yield displacement, the decrease in the capacity may be very significant. Therefore, it shall be ensured that the performance of the intermediate portion containing the anchored reinforcement is not lower than that of the portion where is corresponding to maximum force.

Note that maximum cross-sectional force induced during an earthquake depends on actual strengths of materials and actual cross-sectional area of re-bars provided in a section. Therefore, this point should be carefully taken into account in the assumptions used in computation of design flexural moment and design shear force. For example, it is assumed that column members in bridge piers or column and beam members in frame structures reach plastic region, when structures are designed to satisfy the Seismic Performance 2 for level 2 earthquake ground motion. Maximum flexural moment caused in columns or beams depend on actual arrangements and actual strengths of longitudinal re-bars. Furthermore, the flexural moment increases with increase in stress of the re-bars if the re-bar strain reaches strain hardening zone. Hence, the flexural moment and shear force should be computed using the characteristic strength of materials and member factor. They are taken into consideration since shear force in the response analysis relates to the flexural moment. The condition of development of longitudinal re-bar described here is shown in Fig.5.8.



V_1 : Design shear force that could be occurred.
V_{yd1} : Design shear capacity at the termination point of re-bar
M_d : Design bending moment that may be occurred. In general, this may be set as the design flexural capacity of the member at the bottom end, M_{ud}
M_1 : Bending moment at the termination point of re-bar
M_{ud1} : Design flexural capacity at the termination point of re-bar

Fig. C5.8 Requirement for the development of longitudinal re-bar

(v) In cases then the positive moment reinforcement in a slab or beam is anchored beyond the support, the development length shall satisfy not only the present provision but also the provisions of Section 14.4 of "Design: General Requirements," regarding the structural details for arrangement of reinforcement in the support.

(vi) and (vii) For the development of longitudinal reinforcing bars for a RC member, development length should be secured by determining the start point of anchorage considering the effects of major loads acting on the member and the stress condition in the anchorage zone.

Figure C5.6 (b) illustrates the development length of reinforcement at the base of the column. In general, the reinforcement should be extended close to the bottom of the footing.

Figure C5.6 (c) illustrates the development length of reinforcement at the fixed end of a cantilever, where the reinforcement is confined upward and downward.

Figure C5.6 (d) illustrates the development length of reinforcement at the fixed end of a cantilever where the extended reinforcement is not confined upward. In such cases, it is recommended that the reinforcement be extended to the other end of the connecting member without terminating.

The development length of longitudinal reinforcing bars in a cantilever is specified considering the concrete confinement and stress conditions surrounded the existing longitudinal reinforcement. The development length of longitudinal reinforcing bars in a column, on the other hand, should be specified considering the stress condition of the existing member and the length of insufficient development due to bond deterioration during an earthquake as a major load. In this stipulation, the length of longitudinal reinforcement that suffers bond deterioration during the plastic deformation of a column is specified based on the existing test results. The stipulation is based on the assumption that no excessive cracks occur in the existing member.

Where the thickness or the depth of the base of the column or the member jointed with the cantilever is less than that of the column or the cantilever in Figs. C6.5 (b) to (d), the reinforcement may be extended to the end of the base of the column or the member jointed with the cantilever.

(ix) When the longitudinal reinforcing bars in the column are developed to the massive concrete footing, in which no damaging cracks are likely to occur, development length may be calculated separately considering the characteristics of bond between the reinforcement and concrete. Herein, the development length may be calculated considering the relationship among bond stress, sliding and strain[2]. In cases where earthquakes or other loads repeatedly cause high strain after yielding of longitudinal reinforcing bar that is developed in the concrete, the regions where the development is not effective in the member should be taken into consideration.

5.3 Development of Transverse Reinforcement

(1) Stirrups shall be provided so as to enclose positive or negative moment reinforcement in a manner that the ends are embedded in the compression zone.

(2) At the end of ties or hoops, a semicircular or an acute-angled hook, enclosing longitudinal reinforcement, shall be provided.

(3) Spiral reinforcement shall be anchored at each of its ends by $1^1/_2$ turns of the spiral. Within the plastic hinge section, however, the ends of spiral reinforcement should be overlapped so as to wrap the concrete twice or more times.

[Commentary] (1) When a diagonal crack is formed, the two portions of a beam tend to separate from each other. Stirrups are provided to prevent this separation of the two parts and to serve as vertical tension member in a Howe truss. Thus, the development of a stirrup should be ensured using a hook enclosing a compression bar. When compression reinforcement is used, stirrups should enclose such reinforcement. This ensures a proper anchorage for the stirrups and prevents the buckling of compression reinforcement (Fig. 5.9).

In cases where one set of stirrups is insufficient to encircle all the longitudinal reinforcement because of a large member width, multiple sets of stirrups should be used to encircle the reinforcement (Fig. 5.10).

(2) Tie bars, hoops and intermediate ties should be provided to prevent primary reinforcing bars from buckling, to ensure toughness and adequate distribution of stresses, and to serve as shear reinforcement. When lap splices are used in these reinforcements, sometimes the tie or hoop reinforcements fail to discharge its function in cases of cracking due to bending and spalling of cover concrete. Therefore, tie bar, hoop and intermediate tie should be embedded with a hook enclosing longitudinal reinforcement (Fig. 5.11).

(3) Within the plastic hinge section, the anchorage of the end of the spiral reinforcement may be inadequate, so the spiral reinforcement should wrap the concrete twice or more times, and the ends should be mechanically connected to each other or should be developed to internal concrete using acute-angled hooks at the end of the reinforcement.

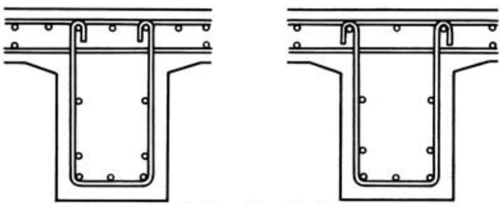


Fig. C5.9 Shape of stirrup end

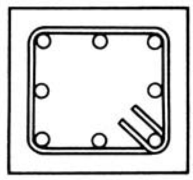


Fig. C5.11 Shape of tie end

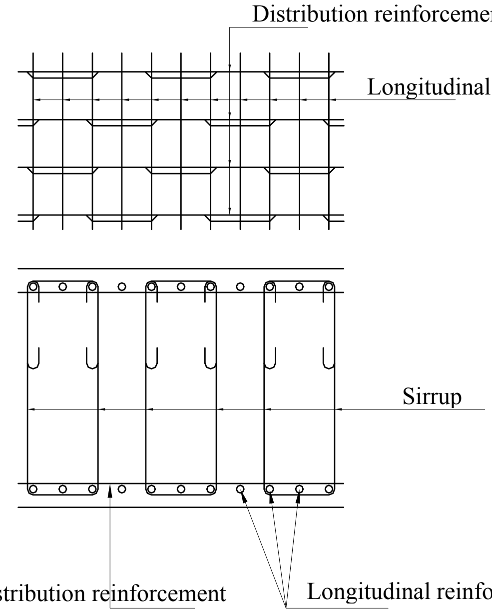


Fig. C5.10 Example of reinforcement arrangement when multiple sets of stirrups are used

CHAPTER 6 SPLICES IN REINFORCEMENT

6.1 General

Splices should be installed for longitudinal reinforcement in accordance with Section 6.2, and for transverse reinforcement in accordance with Section 6.3.

[Commentary] In the case when no special discussion is conducted, splices may be provided as this Chapter.

6.2 Splices for Longitudinal Reinforcement

(1) In cases when the provided reinforcement is not less than 2 times the amount required by design calculations, and, the ratio of spliced reinforcement to total reinforcement is not greater than 1/2, length of lap splice shall not be less than the basic development length, l_d.

(2) When both of the two conditions in (1) are not satisfied, length of lap splices shall not be less than 1.3 l_d and transverse reinforcement shall be provided for the lap splices.

(3) When neither of the two conditions in (1) are satisfied, length of lap splices shall not be less than 1.7 l_d and transverse reinforcement shall be provided for the lap splices.

(4) Length of lap splices shall not be less than 20 times the diameter of the bar.

(5) Spacing between ties, intermediate ties and hoops at lap splices shall be not more than 100 mm as shown in Fig. 6.1.

(6) Length of lap splices for concrete in water shall not be less than 40 times the diameter of the bar.

(7) Lap splices shall not be provided in plastic hinge zones subjected to repeated stresses.

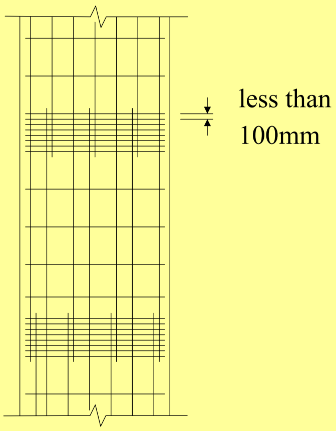


Fig. 6.1 Spacing between ties, intermediate ties and hoops at lap splices

[Commentary] (1) Since lap splices are similar to development of anchorages in terms of stress transfer, the length of lap splice has been specified in relation to the basic development length for anchorages.

Lap splices are easy to provide but their strengths could decrease considerably, resulting from inadequate compaction of concrete, segregation of concrete, or poor quality of concrete in the neighborhood of splices. Also, without the presence of a sufficient amount of transverse reinforcement, concrete surrounding lap splices may split along the reinforcing bars, causing brittle failure, if subjected to high tensile stresses or cycles of high stress. Therefore, splices should be located in areas subject to relatively low stresses and need to be sufficiently reinforced by transverse reinforcement.

(2) Such cases are commonly encountered, and it is recommended that to ensure the safety, the required development length should be not less than 1.3 times the basic development length, and the splices be reinforced with transverse reinforcement.

(3) This clause is provided according to (2) above.

(6) Requirements for a longer development length for concrete in water has been given considering that the bond strength may decrease due to the presence of water-clay slurry or bentonite slurry. When any adverse effect by the slurry is not foreseen, and normal bond strength is expected, the requirement may be waived.

(7) Lap splices should not be used in members subject to low-cycle fatigue, such as columns subjected to repeated stresses due to seismic loads. In cases providing lap splices become unavoidable, they shall conform to the requirements stated herein. In cases where lap splices need to be used in the plastic hinge section, the length of splice should not be less than 1.7 times the base anchorage length l_d, hooks should be installed and the splice zone should be reinforced by spiral re-bars or other splices reinforcement metals.

6.3 Splices for Transverse Reinforcement

(1) Splices for stirrups

No lap splices should be used for stirrups.

(2) Splices for ties

(i) If ties are spliced, they shall be connected such that splices can transmit full strength of the bare bars.

(ii) If lap splices are used, they should be staggered such that the spliced portions are not connected.

[Commentary] (1) Lap splices based on the bond between the reinforcement and concrete should not be recommended for use in stirrups, as cracks along stirrups may occur in some cases. Because stirrups are arranged near concrete surface, in the case that lap splices are used for stirrups, cracking or spalling of concrete cover will cause the bond between the reinforcement and concrete to be lost and adversely affects stress transfer between them. In addition, the progress of cracking may induce large stress locally or deteriorate the bond, however, predicting the locations to be affected is difficult, and the weak points in stirrups may induce unexpected contingencies. No lap splices should therefore be used for stirrups.

In cases of members with a large cross-section where lap splices may be unavoidable, sufficient lap length as specified herein, should be provided. When a hook is provided at the end of stirrup, a bar of D13 or larger shall be provided perpendicular to the stirrup as shown in Fig.C.6.1. In cases of hooks at the end of the stirrup, they preferably point inwards in the member. Lap splices should, however, be used for stirrups only carefully.

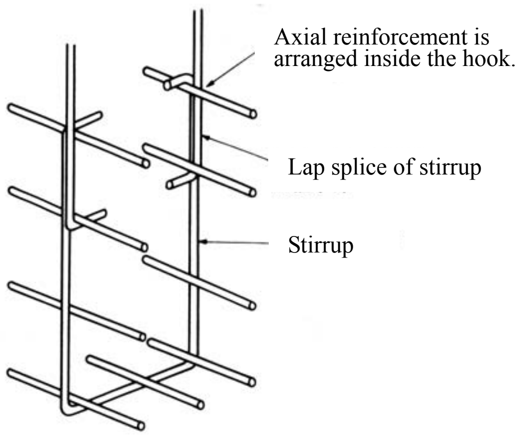


Fig. C.6.1 Arrangement of splice for stirrup

(2) In seismic performance verification, yielding in longitudinal re-bars is assumed. It is, therefore, specified that closed stirrups, ties or spiral re-bars enclosing longitudinal re-bar should be used as transverse re-bars. However, re-bar which is often used in wall-type structures to determine re-bar spaces should not be considered as lateral re-bar because of its insufficient development. In case of spiral re-bars, it is necessary to verify its safety sufficiently in regard to rupture or development at the ends. In some members, seismic performance may not be the dominant required performance. In this case, it may not be necessary to refer this specification.

(i) Ties, which enclose all longitudinal re-bars in the area where spalling of cover concrete occurs with large deformation, should transmit full strength, even if the spalling occurs. The required condition should also be satisfied when splices are provided for ties. Considering this requirement, flare welding or mechanical coupler are recommended, see Fig. C.6.2. For the matters related to flare welding and mechanical splices, refer to the "Recommendation for Design, Fabrication and Evaluation of Anchorages and Joints in Reinforcing Bars [2007]".

In cases where the splices zone is in the internal concrete, lap splices may be used while regarding the ends of the hoop as standard hooks (refer to Fig. C.6.3).

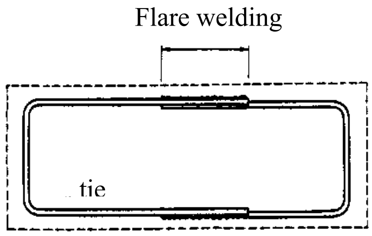


Fig. C.6.2 Splices of ties by flare welding

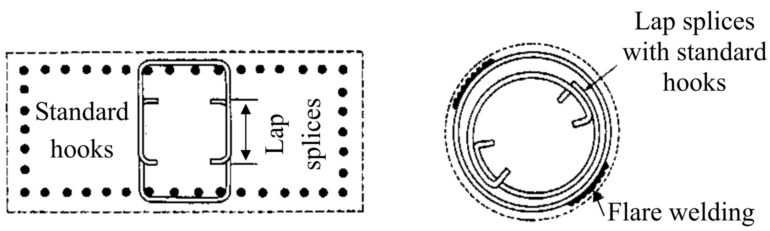


Fig. C.6.3 Lap splices of ties

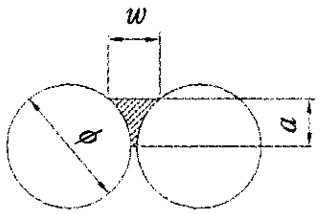
The cross section of flare welding, length of welding, and the shape of welding splices are described below.

(a) The dimensions of a flare welding joint are shown in Fig. C.6.4. The bead width w should not be less than a half of the bar diameter. The minimum width should be 6 mm.

(b) The length of welding should not be less than ten times the bar diameter. When changing the length of welding, welding size, welding materials, etc., the length of welding should be determined after verifying the performance of the splices.

(c) The shape of flare welding is shown in Fig. C.6.5.

(d) The size of flare welding joint, length of welding and the shape of the splices should be presented in design drawings.



$w \geqq \phi/2$, $a \geqq 0.39\phi - e$

where,

w: Bead width (mm)

a: Design throat thickness for strength calculation (mm)

e: Allowance for length. The following values may be generally used.

e=2mm ($\phi \leqq$10mm),
e=3mm (10mm< $\phi \leqq$22mm)
e=4mm (ϕ>22mm)

ϕ: Diameter (nominal diameter) of reinforcement (mm)

Fig. C.6.4 Bead width and throat thickness for flare welding splice

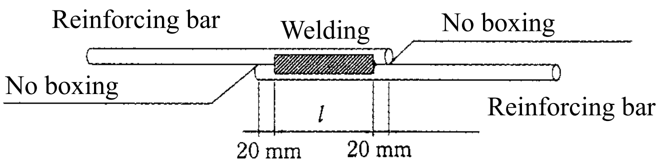


Fig. C.6.5 Shape of flare welding splice

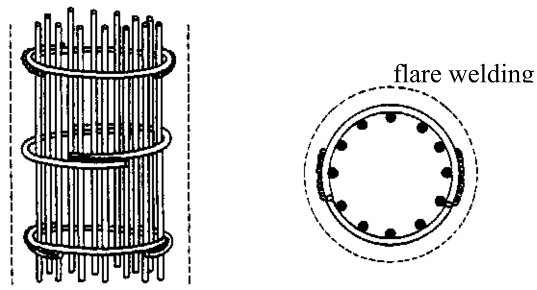


Fig. C.6.6 Spliced portion of ties

(ii) The reliability of spliced portion is less than the bare bar, and tends to be a weak point from construction viewpoint. They should, therefore, be staggered such that the spliced portions are not concentrated in member direction. For example, in the case of ties provided in a member with rectangular shape connected by flare welding at a point as shown in Fig. C.6.2, if the first splice portion of a tie is near the center of side with width “a”, the next tie which provided with intervals of “s” in the longitudinal direction should be near the center of side with width “b”, and the next portion width “a” again. When two re-bars are spliced at two side points facing each other, the same method should be applied (see Figs. C.6.2 and C.6.3). The arrangement of circular or elliptical ties in members with circular or elliptical section should also be done with the same method (see Fig. C.6.6).

In cases where the joints of hoops need to be concentrated in one and the same cross section because of various conditions (splices of medium hoops in the rectangular cross section in Fig.

C.6.3 are examples), " Recommendation for Design, Fabrication and Evaluation of Anchorages and Joints in Reinforcing Bars [2007]" should be honored considering the reliability attributable to construction according to the type of splice for hoops as in the case of longitudinal reinforcement splices.

CHAPTER 7 RE-BAR ARRANGEMENT IN BEAM

7.1 General

(1) Spacing of stirrups, where compression reinforcement is provided, shall not exceed 15 times the diameter of the compression reinforcement or 48 times the diameter of the stirrups.

(2) In beams with large depth, additional reinforcement shall be provided in the horizontal direction in the web.

(3) Additional reinforcement shall be provided to prevent cracking in the web near the support.

[Commentary] (1) To prevent buckling of compression reinforcement, spacing of stirrups has been specified. Also, it is specified not to exceed 48 times the diameter of the stirrups since wide spacing of stirrups with small diameter decreases the effectiveness of the stirrups.

(2) In beams with large depth, flexural cracking in the web cannot be prevented only by primary reinforcement provided at the side of extreme fiber in tension zone. Therefore, horizontal additional reinforcement is required to restrict the width of cracking. Horizontal additional reinforcement in the web is effective in preventing cracks which may occur vertically in the web, due to the conditions of construction, temperature change, shrinkage and so forth. It is recommended to provide reinforcement with an area of more than 0.2 % of area in the web, with spacing not greater than 300 mm.

(3) Since a near vertical cracking can occur close to the support of beam due to the effect of concentrated support reaction and so forth, additional reinforcement is required to be arranged sufficiently both in horizontal and in vertical directions.

7.2 Deep Beam

(1) Primary tension reinforcement in simply supported beams shall be provided continuously between supports and shall be anchored at the supports such that it can resist tensile forces greater than the maximum tensile force. It is recommended that, in general, primary tension reinforcement should be embedded beyond the support for a length greater than the basic development length.

(2) Positive and negative moment reinforcement in continuous beams shall be provided throughout the length of beams. Reinforcement provided in the bottom layer may be lap-spliced at intermediate supports, but shall be anchored at the end supports in a manner similar to simply supported beams. Additional horizontal reinforcement shall be provided with 1/2 the spacing required by the Clause (3) of this section in regions within a distance of 0.4 times the span length on both sides from the intermediate supports.

(3) On both sides of a deep beam, additional reinforcement having an area not less

than 0.08 % of the concrete cross sections shall be provided in both the horizontal and vertical directions with the spacing exceeding neither 2 times the width of beam, nor 300 mm.

(4) Detailing at supports shall be carried out in accordance with Section 7.3

[Commentary] (1) Because primary tension reinforcement is considered to be tension tie members in tied arches, all amount of the tension reinforcement required for maximum moment should extend beyond the support and should never be anchored in the spans.

(2) Provision concerning horizontal additional reinforcement to be arranged at the intermediate supports of the continuous deep beams has been made so as to resist the splitting tensile stresses caused by support reactions and to prevent cracking in the web under serviceability limit state.

7.3 Corbel

(1) In cases when the primary tension reinforcement is arranged in two or more layers, the reinforcing bars shall be placed within a distance of $d/4$ from the upper surface of corbels (See Fig. 7.1), where d is the effective depth of corbel at the column face.

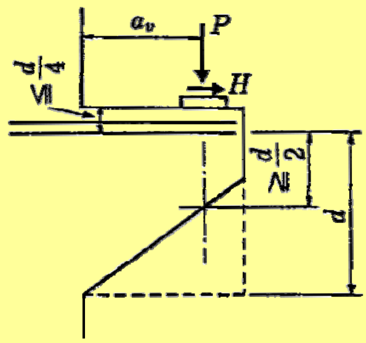


Fig. 7.1 Effective depth at the loading point and primary tension reinforcement in corbel

(2) Primary tension reinforcement shall be bent horizontally and anchored in supporting members, with the inside radius of bend not being less than 10ϕ. The distance of the inside loop of primary tension reinforcement shall be the maximum of the following: 13ϕ from the loaded point, 2ϕ from the edge of the loaded plate and cover thickness in concrete.

(3) In both sides of a corbel, additional reinforcement having area not less than 40 % of primary tension reinforcement shall be provided at a spacing not exceeding 300 mm.

[Commentary] (2) Since inadequate end anchorage of primary tension reinforcement causes a failure as shown in Fig. C 7.1, the end of bars must be looped and anchored extending beyond the loaded point.

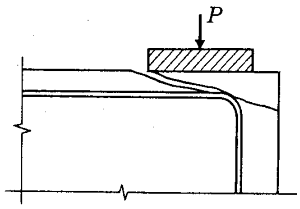


Fig. C.7.1 Example of failures due to insufficient anchorage of tension reinforcement

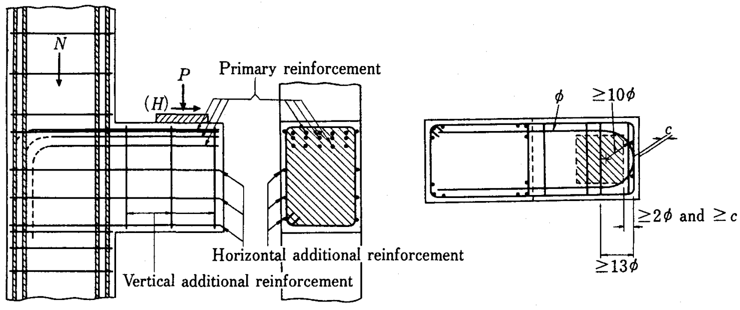


Fig. C.7.2 Example of reinforcement arrangement in corbel

(3) Additional reinforcement in side faces of corbels should also be arranged in the zone where primary tension reinforcement no longer exists so as to prevent the progress of diagonal cracks produced in the web and the splitting failure at the loaded point. It is recommended for additional reinforcement to be arranged in such a way to enclose the longitudinal reinforcement in columns to which corbels are attached.

An example of arrangement of reinforcement in corbels is shown in Fig. C.7.2.

CHAPTER 8 RE-BAR ARRANGEMENT IN COLUMN

8.1 Tied reinforced Column

(1) At least four reinforcing bars with a diameter not less than 13mm shall be provided. The cross-sectional area of the longitudinal reinforcement shall be neither less than 0.8 % nor greater than 6 % of the required cross-sectional area of concrete.

(2) Diameter of lateral ties and hoop reinforcement shall not be less than 6mm, and the spacing of lateral ties shall not be greater than the minimum lateral dimension of the column, 12 times the diameter of longitudinal bar and 48 times the diameter of the lateral tie (see Fig. 8.1).

Sufficient lateral ties shall be provided especially at the connections with beams or other members.

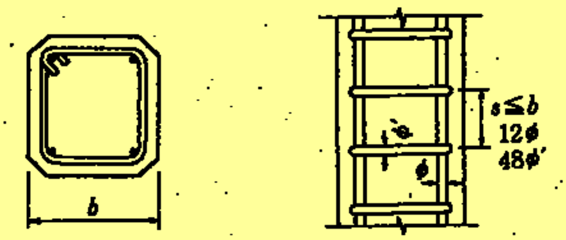


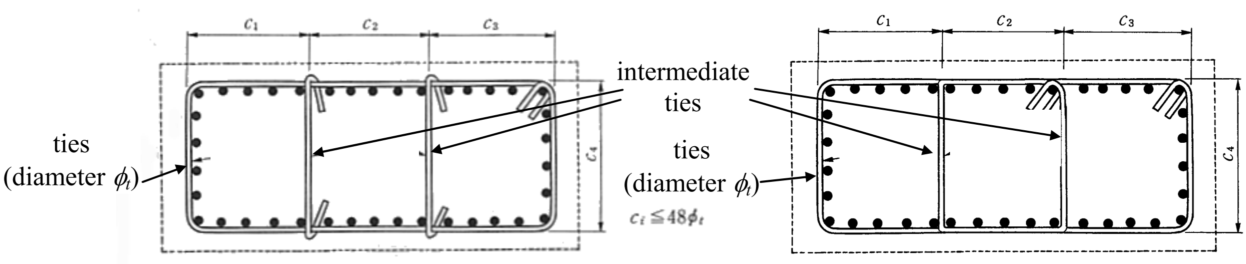
Fig. 8.1 Tied reinforced columns

[Commentary] (1) Diameter of longitudinal reinforcing bars has been prescribed to be not less than 13 mm in the provisions because the reinforcing bars, of which diameters are less than 13 mm are often unable to straighten out during placing owing to insufficient rigidity of the bars. And, the minimum number of bars has been prescribed to be not less than 4 because no adequate reinforcement of column can actually be achieved if less than 4 bars are arranged.

(2) Purposes of lateral ties are to prevent the buckling of longitudinal reinforcements, to fully utilize the compressive strength of concrete by confining the lateral strain of concrete due to axial compression forces, and to bear a part of shear forces.

Spacing of lateral ties has been prescribed to be not greater than 48 times the diameter of lateral tie because the purpose of lateral ties cannot be achieved if lateral ties which are too slender are arranged at wide intervals.

One side of a hoop c_i should not be shorter than 48 times the tie diameter and 1 m to prevent the restraining effect from being deteriorated greatly (Fig. C.8.1). Then, multiple ties should be arranged in combination.



(a) (b)

Fig. C.8.1 Arrangement of ties and intermediate ties in the large cross section

When intermediate ties are used, the same materials, diameter and interval in the longitudinal direction of the member may be applied as those for ties. In cases where it is necessary to arrange intermediate ties as shown in Fig. C.8.1 (b), they should be placed directly on the tie and adequate anchorage should be provided at the end. Care should be exercised not to prevent concrete casting work because of the arrangement of intermediate ties.

8.2 Spiral Reinforced Columns

(1) By providing at least six reinforcing bars with a diameter not less than 13 mm, the cross-sectional area of the longitudinal bars shall be neither less than 1 % nor greater than 6 % of the effective cross-sectional area of columns, and, shall also not be less than 1/3 the converted cross-sectional area of spirals.

The converted cross-sectional area of spirals A_{spe} shall be computed in accordance with following equation

$$A_{spe} = \frac{\pi d_{sp} A_{sp}}{s} \qquad \textbf{(8.1)}$$

where, d_{sp} **: diameter of the effective cross section of spiral reinforced column**

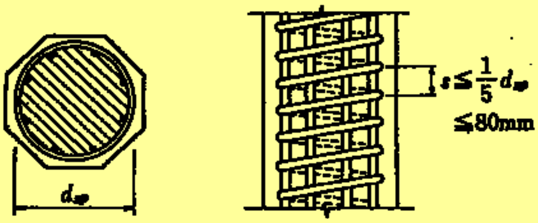
A_{sp} **: cross sectional area of spiral reinforcement**

s **: spacing of spiral reinforcement**

(2) Diameter of spirals shall not be less than 6 mm, and the spacing shall not exceed 1/5 of the diameter of effective cross section of columns and 80 mm (See Fig.8.2).

Converted cross-sectional area of spirals shall not be greater than 3 % of the effective cross-sectional area of columns.

Sufficient spirals shall be provided especially at the connections with beams or other members.



A_e : **effective cross-sectional area of spiral reinforced columns**

Fig. 8.2 Spiral reinforced columns

[Commentary] Spiral reinforced column is a reinforced concrete column having longitudinal reinforcement entwined by spiral bars. Column with longitudinal reinforcement bars hooped by circular ties instead of spirals may be treated as spiral reinforced column, if reinforcement is completely anchored.

(1) Diameter of longitudinal reinforcing bars has been prescribed to be not less than 13 mm because of the same reason as the case of tied columns set forth in Section 8.1(1). For keeping proper shape and spacing of spirals, it is necessary to arrange proper numbers of longitudinal reinforcing bars along the inner circumference of spirals and to tie the bars firmly to spirals. For this purpose, six longitudinal re-bars are necessary at least and it is desirable to arrange not less than 8 bars. The provisions for limitation of cross-sectional area of longitudinal reinforcement are installed for the necessity that provides longitudinal reinforcement of considerably large cross-sectional area in order to realize the effect of spirals sufficiently. It is necessary in practice to provide them not less than 1/3 the converted cross-sectional area of spirals.

(2) Diameter of spirals has been prescribed to be not less than 6 mm, because the spirals must have a high rigidity and, in practice, too small spacing of spirals must be avoided. The reason why the minimum spacing of spirals has been prescribed is to realize the full effect of spirals, which has been determined from the results of experiments and considerations for actual work.

8.3 Splices of Reinforcement in Column

(1) In principle longitudinal reinforcement shall be spliced using gas pressure welded splices, mechanical connections, pressure joint splices or enclosed welding splices. When using lap splices, the number of splices at any cross section of the column shall not be greater than 1/2 the number of longitudinal bars.

(2) When using lap splices in spirals, the length of each lap splice shall not be less than one and a half wind of the spiral.

[Commentary] (1) When using lap splices in longitudinal reinforcement of column, lap splices particularly tend to become the structural weak points. Because of this, splice positions must be shifted from each other throughout the column. Therefore, above requirements as well as the provisions of Section 13.7 must be satisfied.

CHAPTER 9 RE-BAR ARRANGEMENT IN A SLAB

9.1 General

(1) At a section, where the moment is maximum, the spacing of positive and negative moment reinforcement in slab shall exceed neither two times the thickness of the slab nor 300 mm. For other sections, the above spacing shall exceed neither three times the thickness of the slab nor 400 mm.

(2) In cases when negative moments may occur near the supports of a simply supported slab, appropriate reinforcement shall be provided.

(3) When creating an opening in the slab, additional reinforcement should be arranged so as to provide enough capacity resisting on cracking due to concentration of stresses, and other reasons, and prevent from excessive cracking.

[Commentary] (1) This Section limits the spacing of primary reinforcement, otherwise behavior of slab as a reinforced concrete may not be ensured.

(2) When a simply supported slab is loaded at the edge, the additional reinforcement shall be provided for the negative moment which may occur near the loaded area.

(3) Since openings in slab are inevitable in many cases, harmful cracks may occur due to the stress concentration around the openings. As the degree of stress concentration varies case by case, it is recommended to take appropriate measures in accordance with numerical analysis, experiments or observation of cracking in existing structures. Typical example of additional reinforcements is shown in Fig.C.9.1. These reinforcements around corners shall be developed with adequate anchorage.

The amount of primary and distribution reinforcements interrupted by an opening shall be added along the sides of the opening.

For slab with large opening, it is recommended to provide measures derived from the numerical analysis or the test results.

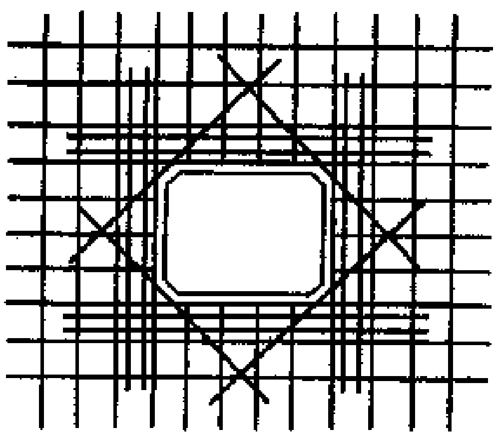


Fig.C.9.1 Additional reinforcements around opening

9.2 One-Way Slab

(1) When modeling a one-way slab as a linear member for analysis, adequate distribution reinforcement should be arranged at transverse direction to the span.

(2) For simply supported one-way slabs subject to uniformly distributed load, the amount of distribution reinforcement per unit length shall not be less than 1/6 the primary tension reinforcement per unit width of slab.

(3) For simply supported one-way slabs subject to a concentrated load, the amount of distribution reinforcement per unit length shall be at least α times the area of primary tension reinforcement per unit width. Here α shall be as given in (i) and (ii).

(i) Loading in the vicinity of the center of the slab

for distribution reinforcement at the bottom of the slab

$$\alpha = (1 - 0.25 \cdot l/b)(1 - 0.8 \cdot v/b) \qquad \textbf{(9.1)}$$

In cases when $l/b > 2.5$, α shall be computed taking $l/b = 2.5$.

(ii) Loading near the edge of the slab

for distribution reinforcement at the top of the slab

$$\alpha = (1 - 2 \cdot v/b)/8 \qquad \textbf{(9.2)}$$

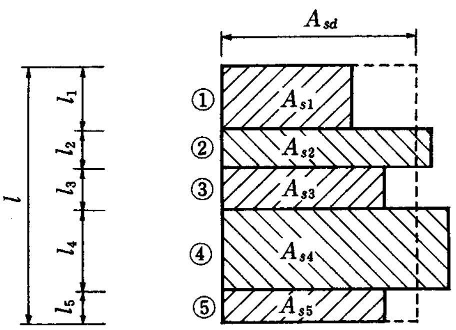
where, l **: span of slab**

b **: width of slab**

v **: distribution width of load**

(4) In cases when slabs are subject to both distributed and concentrated loads, reinforcement provided shall be the sum of area of reinforcement required for each type of load.

[Commentary] (1) In cases multiple concentrated loads act on a slab in a manner that their effective widths overlap, the effect of all the loads shall be adequately accounted for in the design of each region. However, in an application where the requirement of the reinforcement varies for different regions, the following simplification may be made. In cases when the variation in the requirement for the different regions is not significant, the reinforcement per unit width, A_{sd}, may be distributed as shown in Fig.C.9.2 in accordance with the theory of plasticity, except in the case of skewed slabs for which a separate treatment may be required.



, $A_{sd} \times l \geqq l_1 \times A_{s1} + l_2 \times A_{s2} + l_3 \times A_{s3} + l_4 \times A_{s4} + l_5 \times A_{s5}$

$A_{s1}, A_{s2}, \cdots A_{s5}$: amount of reinforcement per unit width for each region
$l_1, l_2, \cdots l_5$: effective width for each load

Fig.C.9.2 Example of reinforcement provided in slabs subject to multiple concentrated loads

Moments in one-way slab occur not only along the direction of the span but also in the transverse direction.

(2) In cases when the span to width ratio in a slab subjected to uniformly distributed loads is not less than 0.5, the coefficient given in Table C.9.1 may be used instead of 1/6 for distribution to primary reinforcement ratio.

Table C.9.1 Coefficient for distribution reinforcement

l/b	0.5	0.7	1.0	2.0
β	1/6	0.16	0.13	0.07

(3) In the cases when a one-way slab is subject to a concentrated load, it deforms in a concave manner in the vicinity of the load, which requires adequate reinforcement in the direction perpendicular to the primary reinforcement. In cases when a one-way slab is subject to concentrated load near the edge, negative moment may occur in a portion away from the load. When such a moment occurs in a slab, it is recommended that distribution reinforcement in the transverse direction at the top of slab should be provided.

9.3 Two-Way Slab

(1) In cases where a two-way slab with a short and long span ratio (l_x/l_y) of 0.4 or lower is subjected to a uniformly distributed load, it is assumed that the load is carried only by the short span. In cases where re-bars are arranged in the direction of the short span as bar members for which the total width of the slab is assumed to be effective, re-bars should be arranged as described below in the direction of the long span.

i) For slabs specified in 1), at least one-fourth the primary reinforcement per unit width of short span shall be provided as distribution reinforcement per unit length of short span.

ii) For continuous or fixed-end slabs, at least half the primary reinforcement per unit width of short span, shall be provided at the top at right angles to the direction of the short span. This reinforcement shall extend more than 1/3 the length of the short span from the front edge of the support.

(2) In end sections on the long side of a fixed or continuous slab, three sides are fixed and one side is left free. For the moment at the fixed end on the short side, additional reinforcement should be arranged.

(3) In cases when a two-way slab is not built monolithically with the walls or beams, or is discontinuous beyond the slab support, additional reinforcement at the corners of the top and bottom of slab shall be provided.

i) This additional reinforcement shall be provided over a distance equal to 1/5 of the longer span from the corner in both directions.

ii) Additional reinforcement shall be placed parallel to the diagonal from the corner at the top of slab and perpendicular to the diagonal at the bottom of slab. The reinforcement specified in this Section may be placed in two bands parallel to the sides of the slab.

iii) The area of additional reinforcement per unit width in each direction shall be equal to the area of positive moment reinforcement per unit width provided at the center of slab for the short span (See Fig. 9.1).

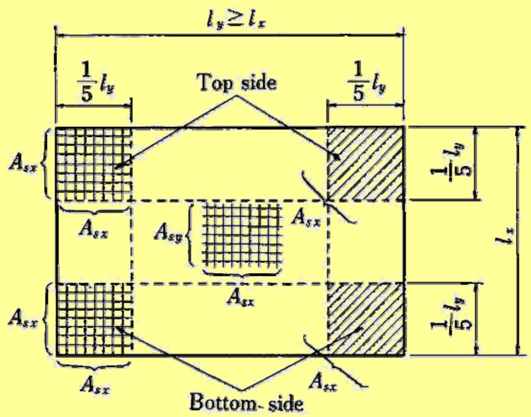


Fig. 9.1 Additional reinforcement at corners of two-way slab

[Commentary] (1) When two-way slabs with a ratio of shorter span to longer span l_x/l_y, not greater than 0.4 are subjected to uniformly distributed loads, the magnitude of moment in x direction is nearly equal to that calculated as a one-way slab in the short span direction. In this case, the moment may be computed as a beam with unit width of slab spanning in short span direction. It is necessary to provide distribution reinforcement for the moment in y direction accordingly.

(3) When two-way slab is not monolithically built with walls or beams, or discontinuous beyond the slab support, additional reinforcement shall be provided at corners of the top and bottom of slab against warping at corners and torsional moment.

9.4 Cantilever Slab

(1) The area of distribution reinforcement per unit length of slab shall not be less than 1/6 the area of the primary tension reinforcement per unit width of slab. Reinforcing bars with a diameter not less than 6 mm shall be provided in the transverse direction at the bottom of the slab with spacing not exceeding three times the slab thickness. For a cantilever slab subject to a large concentrated load, distribution reinforcement shall be increased in accordance with the provisions for a one-way slab.

(2) Primary tension reinforcement in a cantilever slab should be fully anchored.

[Commentary] (2) Hooks shall be provided for primary reinforcement in a cantilever slab. Where the thickness of slab is not enough to anchor a reinforcing bar shall be bent down at the free end and extended along the bottom of slab (Fig. C.9.3).

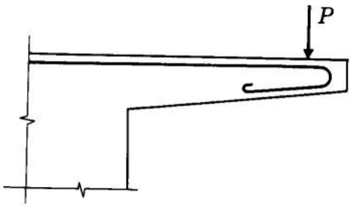


Fig. C.9.3 Development of primary reinforcement in cantilever slab subject to large concentrated load at free end

9.5 Skewed Slab

The primary, distribution and additional reinforcements shall be provided in skewed slabs as specified in (1) to (3) below.

(1) Arrangement of primary reinforcement (See Fig 9.2)

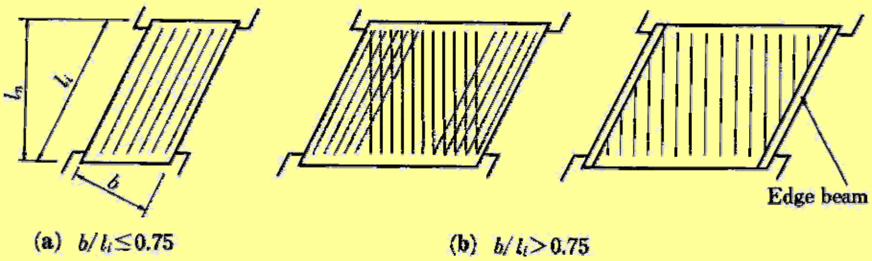


Fig. 9.2 Arrangement of primary reinforcement in skewed slab

(a) For $b/l_i \leq 0.75$, the positive moment reinforcement shall be arranged in the skewed direction.

(b) For $b/l_i > 0.75$, the positive moment reinforcement shall be distributed over

the center of the slab parallel to the direction of the normal span.

In case of $b/l_i > 0.75$ and the positive moment reinforcement arranged in the skewed span direction at two free sides, it is safe enough to provide the positive moment reinforcement required to resist the moment assuming the skewed span length.

(2) Arrangement of distribution reinforcement

(i) For $b/l_i \leq 0.75$, the amount of distribution reinforcement shall not be less than 1/4 of the positive moment reinforcement and shall be placed in the direction normal to the positive moment reinforcement or parallel to the edge of support.

(ii) For $b/l_i > 0.75$, the amount of distribution reinforcement shall be not less than 1/3 of the positive moment reinforcement and shall be placed parallel to the edge of support.

(3) Arrangement of additional reinforcement

At the top of obtuse corners of skewed slabs, additional reinforcement parallel to the skewed span and the support shall be provided over a distance equal to 1/5 of the skewed span length from the edge in each direction, as shown in Fig. 9.3. The additional reinforcement provided in each direction shall be the product of the positive moment reinforcement and the coefficient α as specified in Table 9.1.

For $b/l_i \leqq 0.75$, reinforcement provided at the top of obtuse corners may be considered as a part of the distribution reinforcement.

Table 9.1 Coefficient for additional reinforcement at obtuse corner

Skewed angle	Coefficient (α)
$90° > \theta \geq 75°$	0.6
$75° > \theta \geq 60°$	0.8
$60° > \theta \geq 45°$	1.0

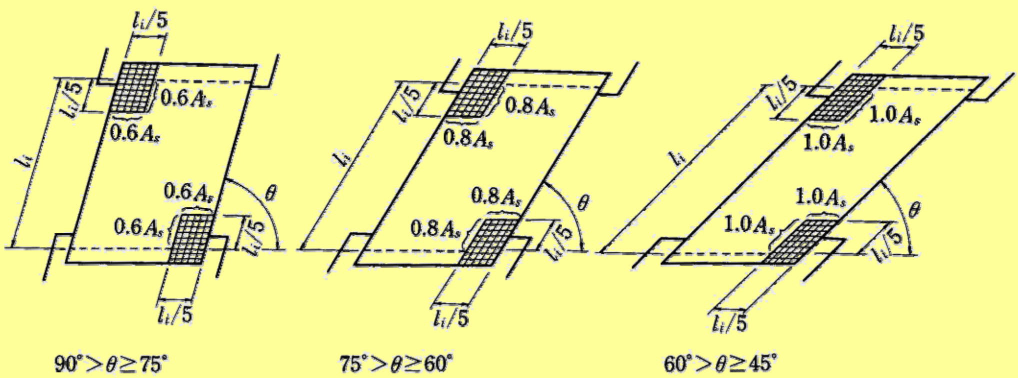


Fig. 9.3 Additional reinforcement at corners of skewed slabs

[Commentary] At obtuse corners of the support of simply supported skewed slab, torsional moments cause the tensile stress in the top of slab in the normal direction to a bisector of the obtuse corner. The tensile stress varies according to the diagonal angle, ratio between the diagonal span and slab width and loading condition. Accurately obtaining the tensile stress is difficult. Additional reinforcement should therefore be arranged based on the results of approximate studies of tensile stress.

For the additional reinforcement against torsional moment in obtusely angled areas, the range of arrangement can be properly specified by FEM analysis using shell elements. The arrangement of secondary reinforcement can also be determined by conventional methods as described below.

9.6 Circular Slab

In principle, appropriate reinforcement should be provided in the radial and circumferential directions to resist applied moment. Alternatively, in cases where such arrangement is not appropriate as at the center of the slab or for small slabs, they may be provided in two orthogonal directions.

[Commentary] This Section specifies for a circular slab that the reinforcing bar shall be provided in the radial and hoop directions of principal moment as it is the most effective.

When the above arrangement of reinforcement is unsuitable to apply to the members, such as the center region of slab or small slab, reinforcement may be provided in two orthogonal directions like a mesh.

9.7 Flat Slab

(1) Reinforcement against the moment transferred to the column shall be provided on each side of the column over a distance equal to the width of column plus 1.5 times the thickness of slab. (see Fig 9.4)

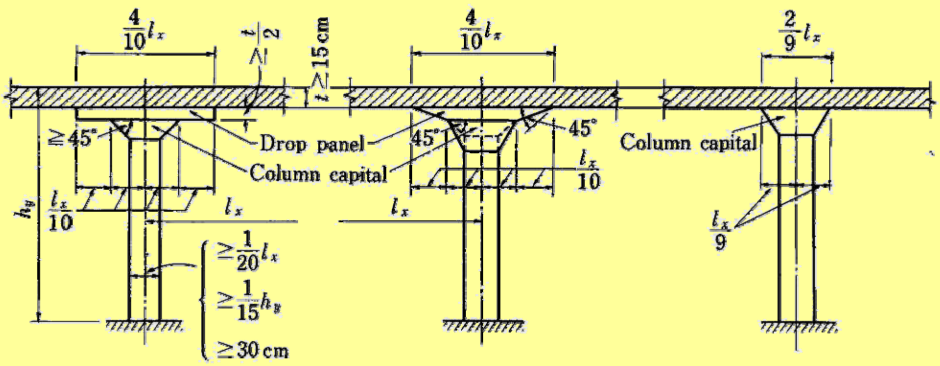


Fig. 9.4 Flat slab system

(2) In cases when moment is transferred to a column, reinforcement shall be provided at the bottom face of slab near the column head. In cases when the required reinforcement cannot be calculated using adequate tools of analysis, the cross-sectional area of reinforcement therein shall not be less than 1/2 that of reinforcement provided at the top face of slab. The reinforcement shall be provided over an area as given in (1) above.

CHAPTER 10 RE-BAR ARRANGEMENT IN SHELL AND WALL

(1) Spacing of longitudinal reinforcement shall not be greater than 2 times the thickness of shell or 300mm, whichever is smaller. Area of longitudinal bars shall not be less than 0.25 %, nor greater than 5 % of the cross-sectional area of concrete. In addition, the cross sectional area of transverse reinforcement provided perpendicular to the reinforcement along the principal axis shall not be less than 1/4 of the latter.

(2) In the portion subject to large tensile forces, longitudinal reinforcement should preferably be arranged in the same direction as that of the principal stresses due to the action of permanent loads.

(3) Longitudinal reinforcement in shells at the location of the connection with the supporting or the end member shall be anchored into the supporting or the end member.

(4) Splices in reinforcement shall not be located at the location of the joint between the shell and the supporting or end members.

(5) Reinforcing bars should be arranged in a wall as described in (a) through (e) below.

(a) The total sectional area of vertical reinforcement in a wall subjected to vertical loading should be between 0.4% and 4% of the total sectional area of concrete.

(b) The total sectional area of vertical and horizontal shear reinforcement in an earthquake-resistant wall should not be smaller than 0.15% of the total sectional area of vertical concrete in all directions.

(c) The diameter of the reinforcement in shear walls shall not be less than 13 mm, and the spacing of bars shall not be greater than 2 times the wall thickness or 300mm, whichever is smaller.

(d) The vertical reinforcement located at both sides of the wall shall be tied to each other using tie bars.

(e) The diameter of horizontal reinforcement should not be smaller than 13 mm and one-fourth of the diameter of vertical reinforcement. Horizontal reinforcement should be arranged at intervals of 300 mm.

[Commentary] (1) The minimum spacing of longitudinal reinforcement is based on the experience of existing shell structures built in the past in order to prevent hazardous cracks due to drying shrinkage of concrete and temperature gradient. The longitudinal reinforcement means the bars arranged in parallel with the curved surface of shell. Usually, these bars are arranged in two layers orthogonally.

(2) This is the basic concept of the direction of the longitudinal reinforcement since the flow of stress is very complicated in shell structures.

Longitudinal reinforcement, in general, shall be arranged in the same direction as that of principal stress as much as possible, which depends upon the combination of loads. In practice the direction of reinforcement may be arranged as that of principal stress due to permanent loads.

(5) Lateral ties or tie bars enclosing vertical reinforcements shall be arranged uniformly, such as in staggers, throughout the wall surface. The bars near the end surface of wall shall preferably be arranged more closely to each other, in accordance with the provisions of columns.

CHAPTER 11 RE-BAR ARRANGEMENT IN FOOTING

(1) Distribution reinforcement in footings shall not be less than 1/6 of primary reinforcement.

(2) Compression reinforcement shall not be less than 1/6 of tension reinforcement.

(3) Additional reinforcement should be arranged in the web of the footing as in the beam.

[Commentary] (1) and (2) Primary tensile reinforcement for a footing is anchored in the compressive area of concrete. The reinforcement should therefore be bent up from the bottom to the top of the footing, and down from the top to the bottom (refer to Fig.C.11.1).

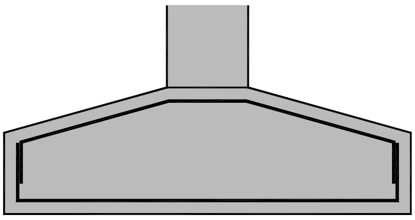


Fig. C.11.1 Anchorage of longitudinal reinforcement

(3) Additional reinforcement should be arranged in the web of the footing against the tensile stress due to vertical forces such as soil and pile reactions as for beams (refer to Fig. C.11.2).

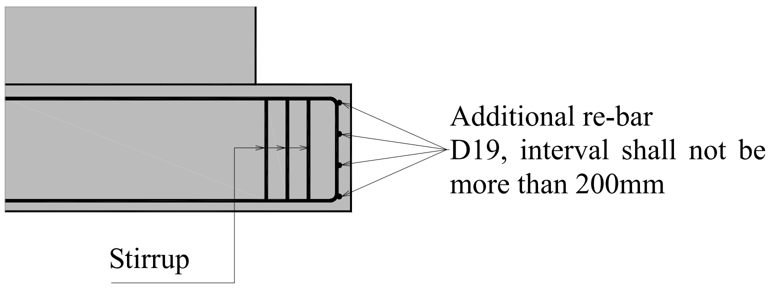


Fig. C.11.2 Secondary reinforcement in the web of footing

CHAPTER 12 RE-BAR ARRANGEMENT IN RIGID FRAME

(1) Primary reinforcement in a beam or column should be fully anchored to the joint of these members.

(2) The shear reinforcement required for checking should be arranged in a beam or column throughout the span of the member. Tie bars or stirrups should be arranged densely near the joint between the beam and the column.

(3) Closely spaced lateral ties or stirrups shall be provided in the neighborhood of the joints between columns and beams.

(4) The joint of the member should be fully reinforced with reinforcing bars or other materials so that the designated strength and deformability may fully develop in the member.

[Commentary] (1) The longitudinal reinforcement in a beam or column should be firmly anchored to the joint of the member. The method of anchorage of longitudinal reinforcement of each member may be determined based on the descriptions below on the assumption that the longitudinal is fully reinforced as described in (4) of this section[1]. The joint of the member is generally determined geometrically according to the dimensions of the member. No adequate anchorage may sometimes be provided in the joint. Then, providing anchorage using anchor plates or securing the anchorage zone using slabs should be considered.

(a) Cantilever slab

The longitudinal reinforcement in a cantilever slab is generally prolonged for use as negative reinforcement in an intermediate slab, or bent for use as positive reinforcement in an intermediate slab. When the reinforcement is anchored without being prolonged, it should be prolonged by $d + l_d$ or more from the study cross section, bent downward from the intermediate slab at an angle of 15 degrees, and prolonged by 10ϕ (refer to Fig. C.12.1).

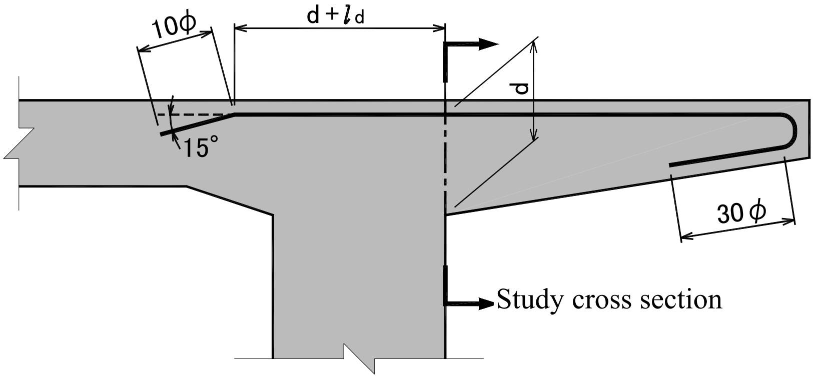


Fig. C.12.1 Arrangement of longitudinal reinforcement in the upper part of a cantilever slab

The reinforcement in the lower part of a cantilever slab should double as the haunch reinforcement in the slab. Then, the reinforcement should be prolonged to the negative reinforcement in the upper part of the intermediate slab and anchored using hooks (refer to Fig. C.12.2).

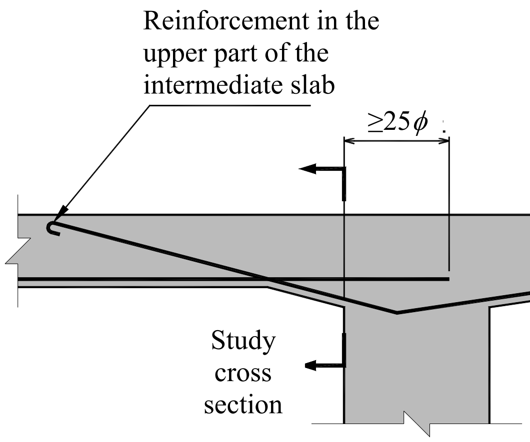


Fig. C.12.2 Arrangement of primary reinforcement in the lower part of the cantilever slab[1)]

(b) Beam

Examples of anchorage of longitudinal reinforcement in an upper layer vertical and horizontal beams are shown in Figs. C.12.3 and C.12.4, respectively. An example of anchorage of longitudinal reinforcement in a middle layer beam is shown in Fig. C.12.5.

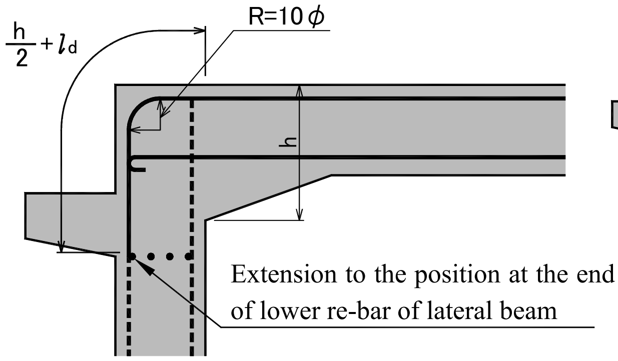


Fig. C.12.3 Anchorage of longitudinal reinforcement in an upper layer vertical beam[1)]

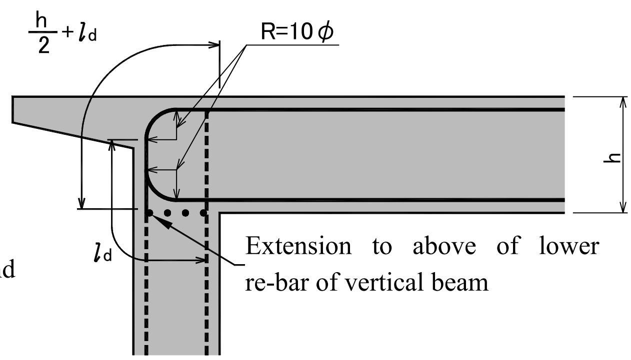


Fig. C.12.4 Anchorage of longitudinal reinforcement in an upper layer horizontal beam[1)]

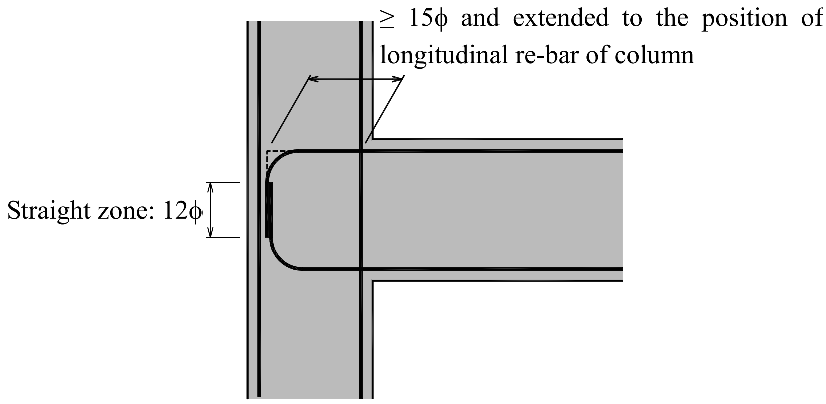


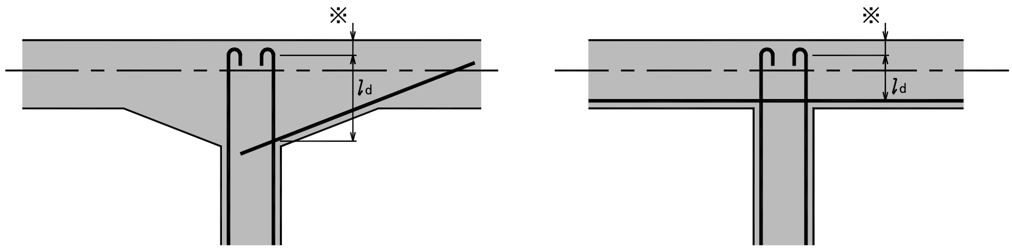
Fig. C.12.5 Anchorage of longitudinal reinforcement in middle layer beam[1)]

(c) Column

An example of anchorage to the upper layer beam is shown in Fig. C.12.6. Hooks are attached at four corners of the longitudinal reinforcement in the column.

When anchoring to an underground beam, the longitudinal reinforcement in the column should be prolonged to the reinforcement in the lower part of the underground beam and semicircular or right-angled hooks should be attached to all reinforcement in principle (refer to Fig. C.12.7).

When anchoring to the footing, the longitudinal reinforcement in the column should be prolonged to the top of the reinforcement in the lower part of the footing. For the extra length of anchorage l', the smaller of a half of the column width and 10ϕ (ϕ : diameter of longitudinal reinforcement in the column) should be specified (refer to Fig. C.12.8 (a) and (b)).



* Prolonged to the bottom of the primary reinforcement in the upper part of the beam (200 to 300 mm below the slab surface)

(a) With haunches (b) With no haunches

Fig. C.12.6 Anchorage of primary reinforcement in column to upper layer beam[1)]

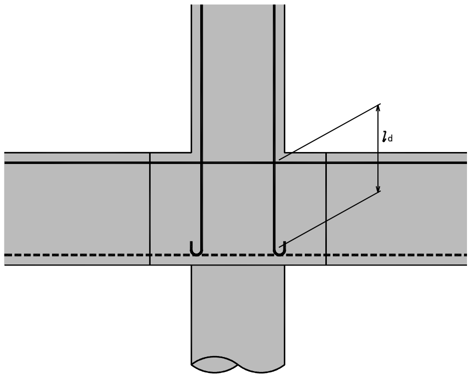


Fig. C.12.7 Anchorage of primary reinforcement in column to underground beam[1)]

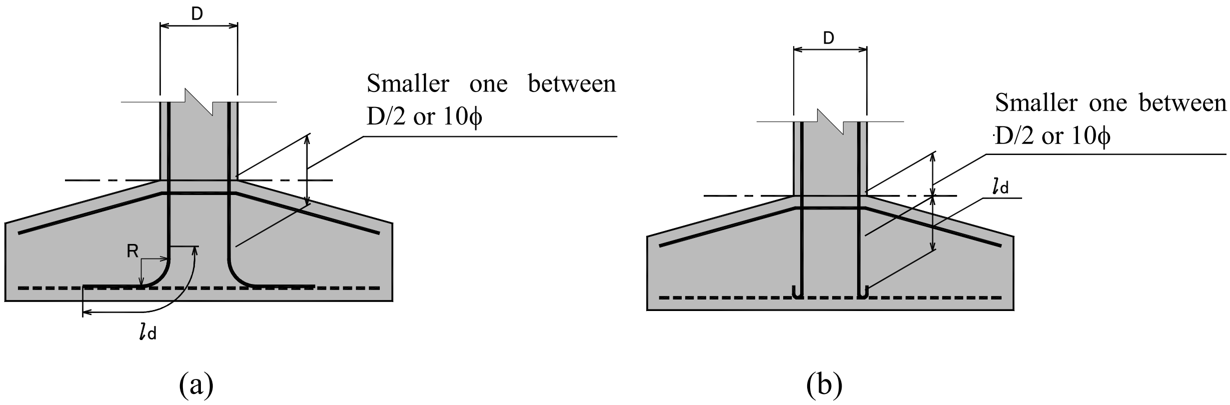
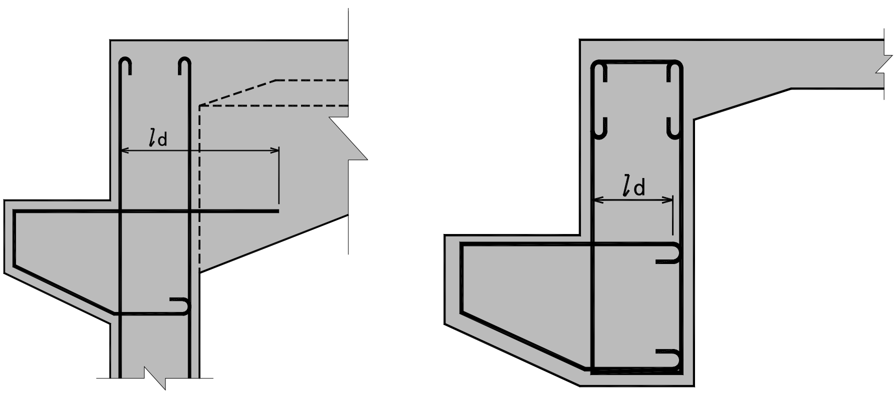


Fig. C12.8 Anchorage of longitudinal reinforcement in column to footing[1)]

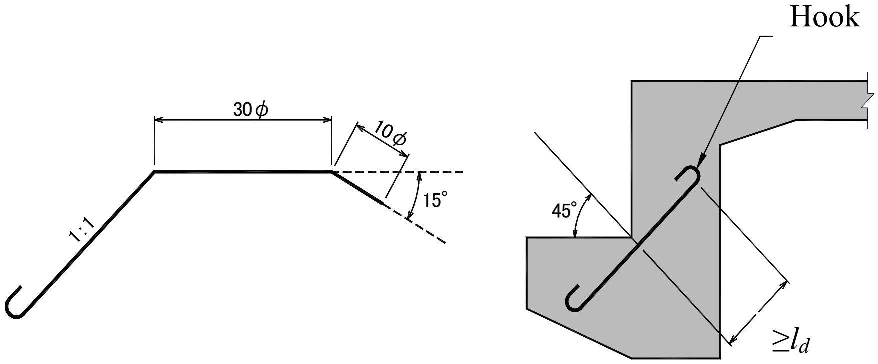
(d) Arrangement of reinforcing bar in beam support

Examples of reinforcement arrangement in a beam support are shown in Fig. C.12.9. The reinforcement against diagonal tension (lifting reinforcing bars) should be arranged as shown in Fig. C.12.10 (a) in principle. If necessary, however, the reinforcement may be arranged as shown in Fig. C.12.10 (b).



(a) With no hooks (b) With hooks

Fig. C.12.9 Examples of reinforcement arrangement in beam support[1)]



(a) With no hooks (b) With hooks

Fig. C.12.10 Examples of dimensions of lifting reinforcement[1)]

(e) Underground beam

An example of anchorage at the end of longitudinal reinforcement in an underground beam is shown in Fig. C.12.11. The top longitudinal reinforcing bar should be bent downward to the top of the reinforcement in the lower part of the beam for anchorage. The bottom longitudinal reinforcing bar should be bent upward with the straight section exceeding 20 ϕ (ϕ: diameter of the reinforcing bar to be anchored).

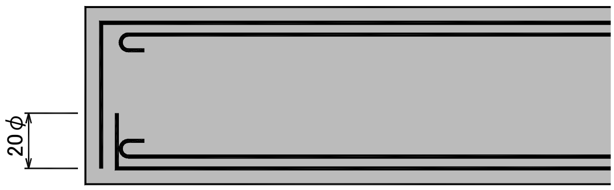


Fig. C.12.11 Example of anchorage of longitudinal re-bars in underground beam[1)]

(2) Shear reinforcement in a beam or column of a rigid-frame structure required for checking should be arranged throughout the length of the member. In the element at the end of the member or at other locations where a plastic hinge is formed, transverse restraining reinforcement should be arranged to control the buckling of the longitudinal reinforcement. Stirrups for beams or ties for columns are arranged as shear and transverse confining reinforcement, respectively. At the columns and beams of rigid frame bridges, large moments and shear forces occur at the same time during earthquakes, and moreover they act to the columns and beams in alternate directions, so it often results in intersected diagonal cracks of members. In cases where inadequate shear reinforcement is arranged, cracks rapidly cause failure. In order to increase ductility near the connection, therefore, adequate ties or stirrups should be arranged effectively. Especially, sufficient lateral ties shall preferably be provided near the upper portion of the column because construction joints for successive pours are often provided there, resulting in a decrease of strength of concrete due to bleeding and so forth. Especially for the columns, the ratio of lateral ties to be provided in the region from the under part of the beam haunch, of the connection with column and beam, shall preferably be not less than 0.25 %, and the spacing of lateral ties shall preferably be not greater than 1/4 of the minimum size of member (See Fig. C12.12). And also, the ratio of lateral ties to be provided from the top of footing to the upper point on column, shall preferably be not less

than 0.2 %. In the range of beam, the quantity of stirrups shall preferably be not less than 0.2 % or 1.2 times the value obtained through calculations, whichever is larger. And also, the maximum spacing of lateral ties shall preferably be not greater than 1/4 of the effective height of member. The same volume of reinforcement steel should be arranged also in the connection for the anchorage of primary reinforcement and for the reinforcement of the joint.

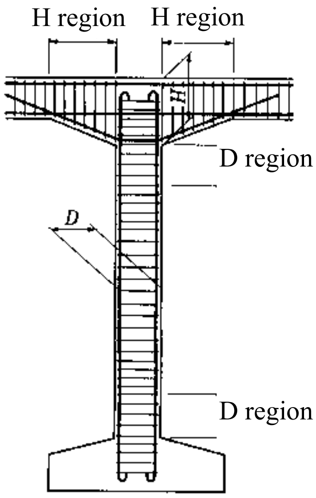


Fig. C.12.12 Reinforcement arrangement at inter support of rigid frame structure

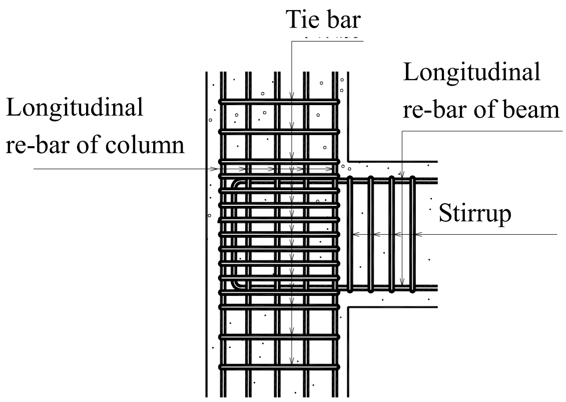


Fig. C.12.13 Reinforcement arrangement at joint

(4) The joint of members should maintain sufficient stiffness and strength even in cases where strength or plastic deformability develops in members in contact with the joint such as beams and columns. To that end, haunches should be installed in members or shear reinforcement should be arranged adequately in the joint so that sufficient strength and stiffness may develop at the joint (refer to Fig. C.12.13). Then, the joint is congested with reinforcing bars. Attention should therefore be paid also to the ease of concrete construction. Even in cases where strength and deformability develops in the joint to which members are connected, if sufficient stiffness and strength in the joint are verified in checking, the cross section and bar arrangement for the joint may be specified so that sufficient stiffness and strength may be provided. In cases where high strength reinforcement is used for the longitudinal reinforcement in columns or beams in particular, great forces act on the connection. Check should therefore be made of the joint by an appropriate method.

CHAPTER 13 RE-BAR ARRANGEMENT IN ARCH

(1) In principle, longitudinal reinforcement in an arch rib should be provided symmetrically along its top and bottom faces. The gross-sectional area of reinforcement shall not be less than 600 mm^2 per meter of the width of arch rib. Also, the total reinforcement for both the top and bottom faces shall be not less than 0.15 % of the cross-sectional area of concrete.

(2) Lateral reinforcement, which encases the longitudinal reinforcement, shall be provided in an arch rib. Diameter of lateral reinforcements shall not be less than 13 mm, and the center-to-center spacing of these reinforcements shall not be greater than 15 times the diameter of longitudinal reinforcements nor greater than the smallest dimension of cross-section of arch rib, whichever is smaller.

[Commentary] (1) Relatively large stresses are induced in arch rib due to temperature change, shrinkage of concrete, displacement of support and so forth. These stresses are released by plastic deformation of concrete, but stresses vary due to displacement of arch axis. In order to reduce these effects, the minimum amount of reinforcement has been prescribed as above.

(2) The lateral reinforcement is provided to keep the location of top and bottom longitudinal reinforcement, to resist secondary stresses induced in the normal direction to arch axis and to prevent the buckling of longitudinal reinforcement.

CHAPTER 14 RE-BAR ARRANGEMENT IN PRESTRESSTED CONCRETE MEMBER

14.1 Reinforcement of Anchorage Zone

(1) End of the member

Tensile stress damaging concrete at the end of the member may occur depending on the interval at which anchorages are installed and the sequence of tensioning of tendons. Re-bars should be applied for protection from the tensile stress.

(2) Intermediate section of the member

(i) When installing anchorages in an intermediate section, anchorages should be embedded in concrete and reinforced with reinforcing bars to prevent adverse effects on the performance of the anchorage zone.

(ii) In cases of anchorage to a projection or notch, reinforcing bars should be applied for reinforcement to prevent the variation of stress due to sudden change of cross section from adversely affecting the characteristics of section of the member.

(3) Reinforcing bars behind the anchorage fixture

For the reinforcing bars against the tensile forces occurring behind the anchorage fixture, reinforcement for the anchorage fixture, spiral reinforcement or reinforcement arranged in a grid pattern should be used.

[Commentary] In the anchorage zone, multiple anchorages are generally arranged in one and the same cross section and the stress condition is complicated. In cases where anchorages are installed in an intermediate section, projections or notches need to be provided in the cross section of the member. Sudden change of cross section frequently causes the stress to concentrate near the anchorage zone. Studies should therefore be made in design to prevent the dimensions of the anchorage zone or stress condition from adversely affecting the characteristics of the cross section of the member. Reinforcing bars should be applied if necessary.

(1) Typical stress distributions near the anchorage zone are shown in Figs. C.14.1 (a) and (b). Near the anchorage zone, tensile stress at right angles to the tendon ((i)), tensile stress at right angles to the tendon between anchorages ((ii)) or tensile stress at the corner ((iii)) occurs. Examples of reinforcing bars against these tensile stresses are shown in Figs. C.14.1 (b) and (d). The method of reinforcement has been set for each method of anchorage. The tensile force at right angles to the tendon ((i)) may be considered during the design of the anchorage as Fig. C.14.2.

(2) In cases where anchorages are embedded, or used as dead anchors, reinforcement should be arranged not only along the tendon but also in the longitudinal direction of the member.

In the case of projection anchorage, the tensile force indicated in Fig. C.14.3 by an arrow is expected to act on the concrete near the anchorage zone. Tensile forces T_1, T_2 and T_6 may be obtained by

$$T_1=0.25P\,(a-a')\,/a$$
$$T_2=0.25P\,(b-b')\,/b \qquad (C.14.1)$$

$T_6 = P \sin a$

where, P is the tensioning force used for anchorage, a and b, and a' and b' indicate the sectional area and size of bearing area of a concrete block cut out of the anchorage zone, respectively (refer to Fig. C.14.4).

For the coefficients in the equations for T_1 and T_2, 0.30 instead of 0.25 may sometimes be used. T_1 and T_2 may be calculated by other methods.

Tensile force T_3 generated at the corner by tensioning force P for anchorage is generally said to be 10% of P.

P causes high compressive force to act on the concrete at the notch. The compressive force may sometimes induce tensile force in front of the anchorage zone. At present, no formula has been proposed for calculating T_4. T_4 should be calculated on the assumption of a force that is proportional to P. In a method of anchorage, for example, calculation is made on the assumption of $T_4 = 0.5P$.

T_5 may be obtained as the tensile force generated due to the action of P that works as an eccentric axial compressive force.

An example of reinforcement in the projection anchorage zone is shown in Fig. C.14.5. Reinforcement F_1 is against T_1, F_2 against T_2 and T_3, F_3 against T_4 and T_5, and F_4 against T_6. An example of reinforcement in the notch anchorage zone is shown in Fig. C.14.6. Reinforcement F_1, F_2 and F_3 are against the tensile force at the notch and F_4 is against the tensile force behind the anchorage fixture.

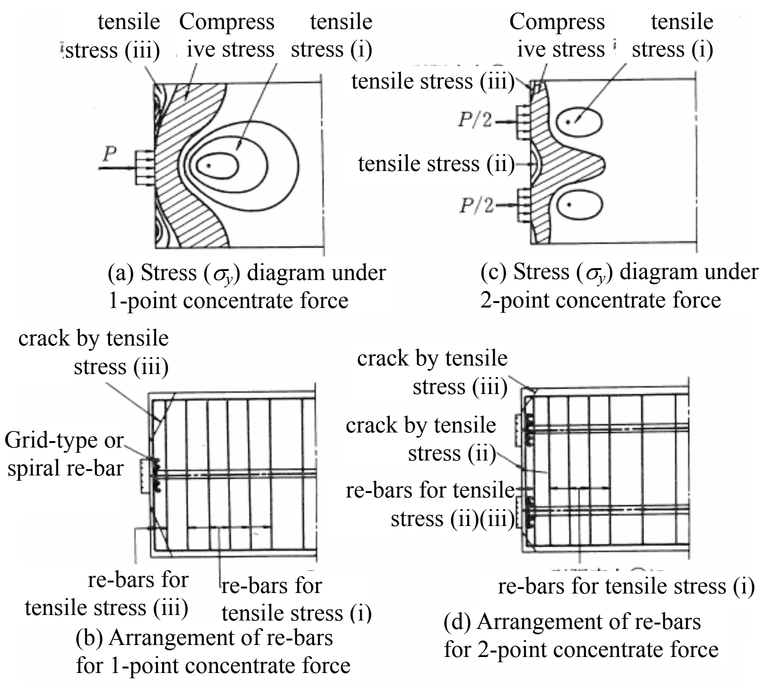


Fig.C.14.1 Stress distribution in concrete at the vicinity of anchorage and example of reinforcement

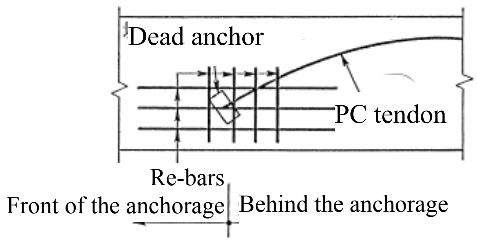
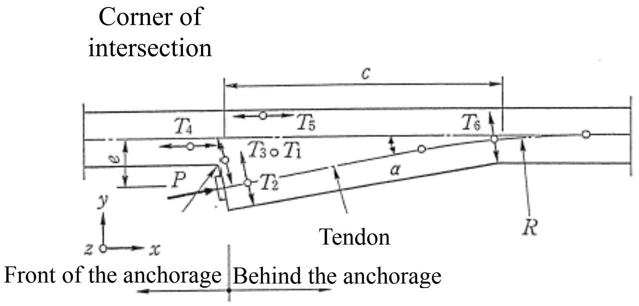


Fig.C.14.2 Example of reinforcement by dead anchorage



Where, P : preseressed force

e : eccentricity of P

c : length of rib-anchorage

R : bending radius of tendon

α : bending angle of tendon

T_1 : tensile force occurring behind the anchorage in z-direction (vertical to this paper)

T_2 : tensile force occurring behind the anchorage in y-direction

T_3 : tensile force at the corner of intersection

T_4 : tensile force generating in front of anchorage

T_5 : tensile force induced by bending moment occurred by prestress force($Me = P \cdot e$)

T_6 : tensile force generating at the bending section of tendon $= P \cdot \sin\alpha$

Fig.C.14.3 Tensile force at rib-anchorage

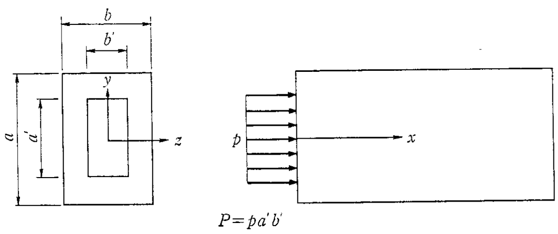


Fig.C.14.4 Concrete block

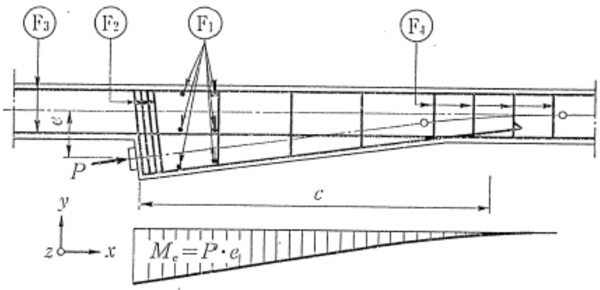


Fig.C.14.5 Example of reinforcement at rib-anchorage

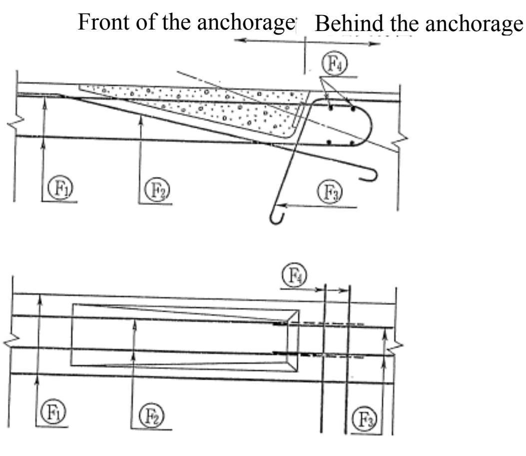


Fig.C.14.6 Example of reinforcement at anchorage with cut-out portion

(3) Tensile force of tensile stress (i) shown in Fig. C.14.1 acts behind the anchorage fixture. Against the tensile force, reinforcement for the anchorage fixture, spiral reinforcement or reinforcement arranged in a grid pattern should be applied. The reinforcement is specified according to the method of anchorage. The reinforcement should be arranged in accordance with "the Recommendations for Design and Construction of Prestressed Concrete" or other documents. Stirrups or other materials arranged in the member should double as the reinforcement against tensile forces.

When using large anchorage fixtures, the area and arrangement of reinforcement should be examined by FEM analysis or other means.

References

1) Railway Technical Research Institute: Guidelines for Reinforcing Bar Arrangement, Design Standards for Railway Structures, Nov. 2004.

2) e.g. Shima, Zhou and Okamura: Bond characteristics of deformed reinforcing bars after yielding, Proceedings of the Japan Society of Civil Engineers, No. 378/V-6, pp. 213-220, Feb. 1987.

PART 6 STRUT-AND-TIE MODEL

CHAPTER 1 GENERAL

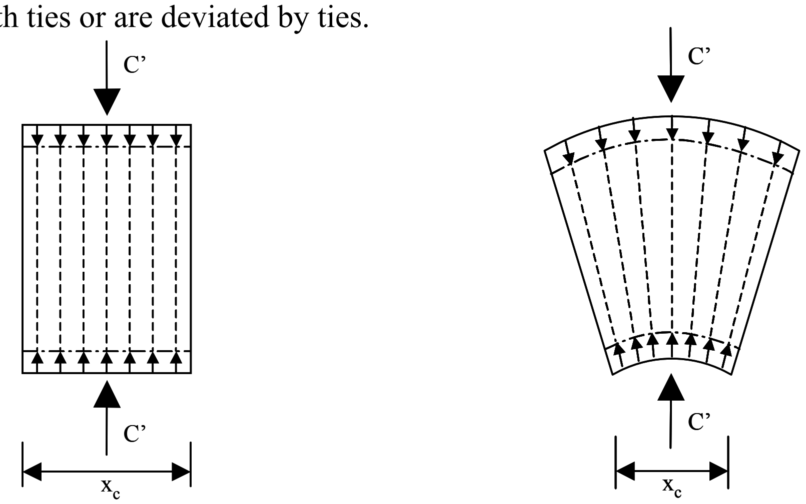
1.1 Scope

(1) This Specification should be applied to cases where the limit state in the case of a sectional failure is examined using a strut-and-tie model defined in Section 9.2.2 of "Design: General Requirements."

(2) The strut-and-tie model should be applied to examine the ultimate limit state of structural concrete in discontinuity regions, such as regions in which the flow of internal forces changes significantly, regions with abrupt changes of section, regions near frame corners, regions with openings, regions near steel anchorages, or regions where the general beam or plate theory is not applicable, when proof experiments for the ultimate capacity or analyses with high accuracy are not performed.

(3) When using the strut-and-tie model to estimate the ultimate capacity of structural members, it should be ensured that the equilibrium conditions between the compression in struts and the tension in the ties at all nodes are satisfied.

[Commentary] The strut-and-tie model is a useful structural tool to estimate the resistance of a structure due to the applied load and how the internal forces flow. In this model, concrete structures or members are discretely modeled into an assembly of one-dimensional struts and ties, and nodes connecting these struts and ties. The ultimate capacity corresponding to the assumed flow of forces is calculated based on the static equilibrium conditions and the individual strength of the struts and ties. The tie is normally modeled on the basis of the resultant force from a layer of reinforcing bars or prestressing steel. The strut represents the resultant of either a uniform compressive stress field, such as a compression chord or a diagonal compression strut in webs of beams, or a fan-shaped compressive stress field, such as over supports or under concentrated loads (Fig. C1.1). The node represents a certain amount of concrete volume, in which struts either intersect with ties or are deviated by ties.



(a) Uniform compressive stress field (b) Fan-shaped compressive stress field

Fig. C1.1 Concrete struts

As far as structural members, such as beams, columns, or slabs, whose design and construction procedures are well established, and structural details of cross-section and detailing of reinforcement are widely and empirically accepted, are concerned, a reasonably safe design can be achieved by following the provisions laid down in either Chapter 9 of “Design: General Requirements”, or those in the present Specification. However, examination for ultimate capacity of some of the structural members including those in the regions with abrupt changes of section, corners of framed structures, or with openings, cannot be carried out using standard methods given in Chapter 9 of “Design: General Requirements”. The present understanding and practice of reinforcement arrangement in regions where the flow of internal forces changes significantly are not necessarily sufficient. In such cases, it is desirable to perform proof experiments or nonlinear analyses with sufficient accuracy to examine the safety of structural members with an assumed geometry and material properties. However, once the resisting mechanism for the design load is predetermined, the design for detailing of the reinforcement and the properties of the materials used, can be accordingly carried out, using methods such as the strut-and-tie model.

The strut-and-tie model described in this Specification should be applied only to examine the ultimate limit state. Since the strut-and-tie model does not strictly consider the compatibility condition in deformations, when deciding the location of struts and ties, it should be ensured that the compatibility of deformations is adequately maintained by providing sufficient deformation ability to the structural members. This ensures the transfer of forces according to the resisting mechanism assumed in the ultimate limit state.

CHAPTER 2 STRENGTH OF TIE

2.1 Strength of Steel Tie

The strength of steel ties shall be obtained by multiplying the cross-sectional area of steel by the design yield strength.

[Commentary] At the ultimate limit state, it may be assumed that the steel of the tie yields. Therefore, the strength of each tie can be obtained by multiplying the cross-sectional area of steel in the tension tie by the design yield strength of steel. When high strength steel or prestressing steel is used and the steel does not yield at the ultimate limit state, the force of tension ties shall be evaluated using the actual stress in the steel at the ultimate limit state.

2.2 Strength of Concrete Tie

In the case of concrete ties, it shall be examined whether the expected tension resistance of concrete still remains at the ultimate limit state. If this examination is skipped, it is not allowed to assume concrete ties.

[Commentary] Although reinforcement is normally provided to resist tensile forces in structural concrete, the tensile resistance of concrete, such as in the case of planar members without shear reinforcement or for bond and anchorage of steel, is often expected even at the ultimate limit state. When concrete ties are assumed, it shall be examined whether the tension resistance of concrete still remains at the ultimate limit state. This is the prerequisite to apply a concrete tie in the strut-and-tie model.

CHAPTER 3 STRENGTH OF STRUTS

3.1 Strength of Concrete Strut

(1) In calculating the load carrying capacity of concrete struts in the strut-and-tie model, the effective strength of concrete $f'_{cd.eff}$ is desirable to be obtained using the design compressive strength of concrete f'_{cd} but appropriately reducing it using factors to account for the presence of cracks in a member, crack width and geometric disturbances. Equation (3.1) may be used the calculation of the effective strength.

$$f'_{cd.eff} = \nu_1 \cdot \nu_2 \cdot f'_{cd} \quad \textbf{(3.1)}$$

where, ν_1 is a reduction factor to consider the difference between the compressive strength of concrete in a structural member and the strength determined using laboratory specimens and may be taken to be 0.85. The reduction factor ν_2 may be taken as follows:

(a) $\nu_2 = 1.00$ **for uncracked struts with uniform strain distribution.**

(b) $\nu_2 = 0.80$ **for struts with cracks parallel to the strut or for struts intersecting bonded transverse reinforcement; the strength of struts shall be reduced considering the disturbances due to the transverse tension or the irregular crack surfaces.**

(c) $\nu_2 = 0.60$ **for struts transferring compression across cracks, such as in webs of beams.**

(d) $\nu_2 = 0.45$ **for struts transferring compression across large cracks, such as in members with axial tension or flanges in tension.**

(2) The load carrying capacity of a concrete strut F_{Rcd} shall be obtained using the effective strength of concrete $f'_{cd.eff}$ and the area of a concrete strut.

$$F_{Rcd} = A_c \cdot f'_{cd.eff} / \gamma_b \quad \textbf{(3.2)}$$

where, A_c **: area of a concrete strut (=** $x_c \cdot b$ **)**
x_c **: depth of a concrete strut**
b **: thickness of a concrete strut**
γ_b **: generally, may be taken as 1.3**

(3) The depth of a concrete strut x_c shall be determined considering the stress distribution and the direction of intersecting reinforcement.

[Commentary] (3) When a uniform stress distribution can be assumed as shown in Fig. C3.1, the depth of a concrete strut may be estimated as $x_c = z \cdot \cos\theta$, where θ is the inclination angle between the resultant compressive force and the member axis. When a concentrated load is applied near the support (Fig. C3.2), a fan-shaped stress distribution occurs over the support and the depth of a concrete strut changes depending on the location. In this case, if the minimum depth determined from the width of a support is used as the strut depth x_c, a conservative estimation for the ultimate capacity can be obtained. When transverse reinforcement intersecting a strut is provided (Fig. C3.3), the change of strut depth due to the location may be considered. Figure C3.4 is a typical example of a strut-and-tie model considering the arrangement of reinforcement around the anchorage of prestressing steel. It shows the steel ties required for the division of concrete struts, which support the tensile force in the prestressing steel and the approximation of tension forces *T* of the ties.

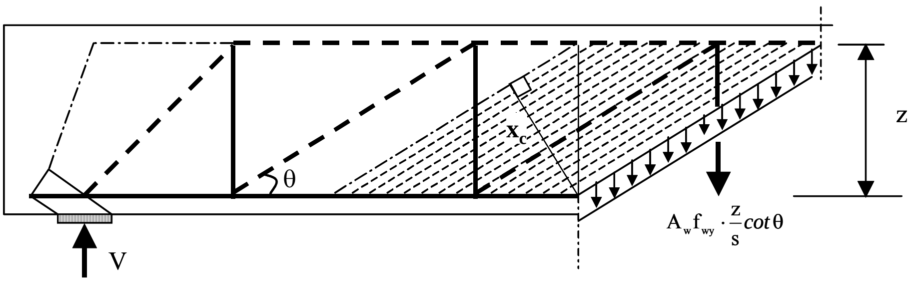


Fig. C3.1 Concrete strut for uniform compressive stress field

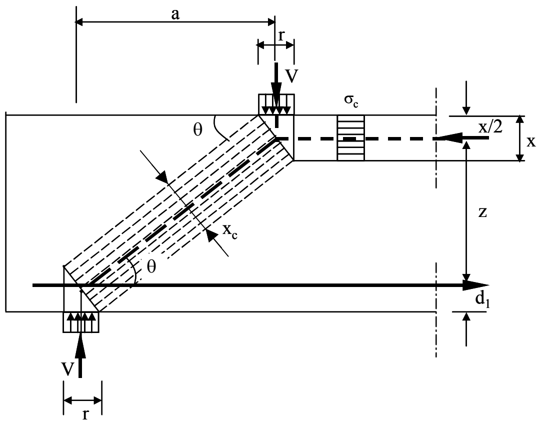


Fig. C3.2 Concrete strut for a concentrated load near a support

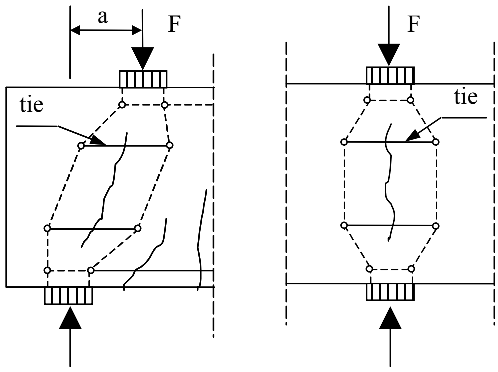


Fig. C3.3 Strut-and-tie model with horizontal reinforcement

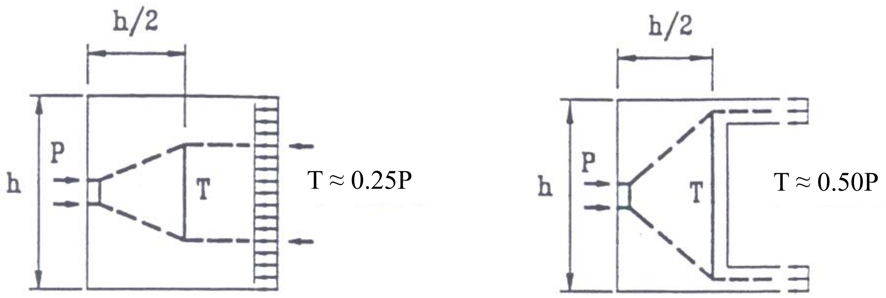


Fig. C3.4 Strut-and-tie model around the anchorage of prestressing steel

3.2 Strength of Concrete Strut with Reinforcement

(1) Reinforcements may be considered effective in compression struts only when they are placed parallel to the strut. Flexural compression reinforcement in a beam and longitudinal reinforcement in a column are examples of such reinforcement.

(2) These reinforcements shall be surrounded with adequate transverse reinforcement to provide the required resistance to buckling.

(3) The load carrying capacity of a concrete strut with reinforcement may be taken to be the sum of the load carrying capacity of the concrete strut and that of the reinforcement as shown in Eq. (13.3.3).

$$F_{Rcd} = A_c \cdot f'_{cd.eff} / \gamma_b + A_{sc} \cdot f_{yd} / \gamma_{bs} \quad \textbf{(3.3)}$$

where, γ_b and γ_{bs} may be taken to be 1.3 and 1.1, respectively.

3.3 Strength of Confined Concrete Strut

The load carrying capacity of a concrete strut may be increased by providing sufficient and proper transverse reinforcement to confine the concrete strut. In this case, the increase in strength should be appropriately assessed.

3.4 Reduction in Strut Thickness

(1) If a strut of thickness b is intersected by ducts or bars such that the sum of their diameters is larger than $b/6$, the resisting force of the strut shall be calculated using a reduced thickness b_r.

$$b_r = b - \eta \sum \phi \quad \textbf{(3.4)}$$

where, $\sum \phi$: sum of the diameters of ducts or bars

η : coefficient depending on the stiffness of ducts or bars; the value shall be taken to be 0.5 and 1.2 for grouted ducts or bonded bars and for ungrouted ducts or unbonded bars, respectively

(2) The load carrying capacity of the strut shall be calculated as given by Eq. (3.5).

$$F_{Rcd} = (x_c \cdot b_r) f'_{cd.eff} / \gamma_b \quad \textbf{(3.5)}$$

where, x_c : depth of the strut

(3) If the load does not act uniformly throughout the thickness of a member or the thickness of a member is not uniform, the reduced thickness of struts shall be used to calculate the load carrying capacity in compression.

CHAPTER 4 STRENGTH OF NODES AND ANCHORAGES OF REINFORCING BARS

4.1 General

(1) The nodes shall be designed in a manner that all forces are appropriately balanced and any ties are securely anchored.

(2) Appropriate additional reinforcement should be provided at the anchorage to take care of transverse tensile stresses that are likely to be carried by tensile resistance in concrete. In cases such reinforcement is not provided, the ultimate capacity of concrete in the anchorage region shall be thoroughly re-examined.

[Commentary] (1) Any anchorage of reinforcement in the anchorage zone produces transverse tension, which is often resisted by concrete tensile stresses. The stresses in concrete in a structural member could be biaxial or triaxial. For example, in the plane stress condition, the concrete is in biaxial compression stress state in C-C-C nodes (C = compression force in strut) connecting struts and is in biaxial tension-compression stress state in C-C-T or C-T-T nodes (T = tension force in tie).

The following verifications shall be carried out during the design of the nodes:

(a) verification for the anchorage of ties in nodes

(b) verification that the maximum compressive stress does not exceed the effective compressive strength of concrete

4.2 Compression Node

(1) In nodes connecting only compression struts, the increase in compressive strength of concrete under multiaxial compressive stress state may be taken into account when estimating the ultimate capacity. The increase in compressive strength of concrete under multiaxial compressive stress state shall be based on adequate and reliable experimental evidence.

(2) In special cases, such as in the anchorage zones of prestressing force, higher local strength of concrete may be utilized, if adequate confining reinforcement is provided in this portion.

4.3 Anchorages of Reinforcing Bar

The reinforcing bars shall be anchored in accordance with the provisions of Chapter 13 of "Design: General Requirements."

[Commentary] The ties have to be securely anchored to ensure that the reinforcement in the ties actually yields, as assumed in the design provisions. However, the real structures, where the strut-and-tie model is often applied, have a geometric layout that makes appropriate arrangement of reinforcement quite difficult. Therefore, if the provisions for the standard structural detailing cannot be followed, the performance of the anchorage should be verified in accordance with Chapter 13 of "Design: General Requirements."

In cases where the performance of the anchorage zone cannot be achieved by the method specified in Chapter 13 of "Design: General Requirements," the performance of the anchorage zone should be verified in accordance with the "2007 Guidelines for Anchorage and Joints for Reinforcing Bars."