STANDARD SPECIFICATION FOR DESIGN

AND CONSTRUCTION OF CONCRETE STRUCTURES

—1986, PART 1 (Design)

(CONCRETE LIBRARY SPECIAL PUBLICATION 1)

Prepared by JSCE Committee on

STANDARD SPECIFICATION FOR DESIGN AND

CONSTRUCTION OF CONCRETE STRUCTURES

Japan Society of Civil Engineers

YOTSUYA 1-CHOME, SHINJUKU-KU

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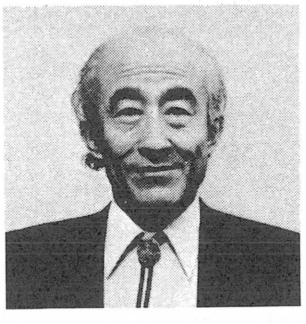

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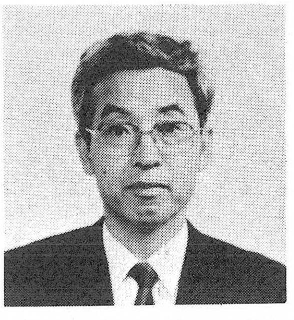

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Introduction

The Standard Specification for Design and Construction of Concrete Structures written by Japan Society of Civil Engineers forms a base line of the state of the practice of the public works construction in Japan. The document consists of Part 1 Design, Part 2 Construction, Part 3 Pavement, Part 4 Dam, and Part 5 JSCE Standards.

A major revision was made for the 1986 edition to incorporate recent developments in structural and construction technologies and, for the first time, to adopt a limit state design method.

The JSCE Subcommittee on standard specification for design and construction of concrete structures was chaired by Dr. Yoshiro Higuchi. Twenty three Working Groups respectively prepared chapters dealing with:

general; cement; admixture; aggregate; proportion,quality; temperature; durability; under water concrete,prepacked concrete; batching,mixing; transporting,placing; spacing; manufacturing product; lightweight concrete; design general; general, design principle, load, structural analysis; ultimate capacity, flexure, shear, torsion; crack, fatigue, deflection, durability; materials, concrete, steel; seismic design; prestressed concrete; design of members; steel reinforced concrete; pavement; and dam.

Each chapter consists of the main text and comments.

English translation was made for Part 1 Design and for Part 2 Construction by a committee chaired by Dr. Hajime Okamura. The English translation covers the subject areas listed above except Part 3 Pavement and Part 4 Dam. The English translation are in two volumes and are titled;

" Standard Specification for Design and Construction of Concrete Structures 1986, Part 1 (Design), First Edition,"

Concrete Library International Special Publication, C.L.I. SP-1,Japan Society of Civil Engineers,244p.

" Standard Specification for Design and Construction of Concrete Structures 1986, Part 2 (Construction), First Edition,"

Concrete Library International Special Publication, C.L.I. SP-2,Japan Society of Civil Engineers,306p.

They can be ordered for purchase to Japan Society of Civil Engineers.

The chapters contained in the Part I Design are as follows.

STANDARD SPECIFICATION FOR DESIGN AND CONSTRUCTION
OF CONCRETE STRUCTURES 1986 PART 1 (Design)

CONTENTS

On the following pages appear the excerpts from the Part 1 Design. They are;

Preface to the original Japanese edition,
Preface to English Edition,
Main texts of,
CHAPTER 2 GENERAL REQUIREMENTS
CHAPTER 6 ULTIMATE LIMIT STATE
CHAPTER 7 SERVICEABILITY LIMIT STATES
CHAPTER 8 FATIGUE
(in this excerpt the comments to the main texts were not included)

The contents of those chapters are the Japanese codification for the first time of the results of original researches in this country and the results based on evaluation of available knowledge throughout the world of structural concrete performance.

Preface to the original Japanese edition of the
" Standard Specification for design and construction
of concrete structures 1986 "

PREFACE

The Standard Specifications for Design and Construction of Concrete Structures have been established by the Concrete Committee of the Japan Society of Civil Engineers (JSCE).

The Concrete Committee was formed in 1930 as the Board of Concrete Investigation (Chairman, Dr. Muneharu OHKAWATO). The chairmanship had since then been succeeded by Dr. Tokujiro YOSHIDA (1948-1960) and Dr. Masatane KOKUBU (1960-1982) before the present committee was organized in 1982.

The previous revision of Standard Specifications was made in 1974 and further amendment in 1980. The followings are main backgrounds for publishing the 1986 Edition incorporating many basic revisions.

1. Almost all necessary preparation was completed for introducing the limit state design method.
2. Recent technical development and improvement in the fields of material and construction methods were remarkable.
3. Social demands for environmental preservation, saving of energy and natural resources, etc. became stronger than ever. In particular, serious questions were raised concerning the durability of concrete structures constructed during the high growth period of the Japanese economy.

One of the distinguishing features of the present Standard Specifications is that many basic revisions were made taking into account of the above-mentioned backgrounds. However, in order to avoid confusions which may result from the abrupt revisions, such consideration as to provisionally accept the allowable stress design method in the former standard Specifications was made. Another feature is that "Maintenance and Control of Structures (Draft)" is compiled as an appendix, based on the judgement that the maintenance and the repair of concrete structures will be important issues in the future. Considering that basic provisions have been revised, and that concrete technology is progressing day by day, it is obviously necessary to make further efforts to make the contents of the Standard Specifications more appropriate. The Concrete Committee intends to continue investigations and researches for the improvement of the Specifications.

Completion of the Standard Specifications and their comments with many basic revisions was possible due to the efforts of the committee members, especially the chairmen and the secretaries of the subcommittee and the working groups, who exerted their utmost efforts. I would like to express my profound gratitude to them.

October, 1986

Yoshiro HIGUCHI, Chairman
Concrete Committee
The Japan Society of Civil Engineers

PREFACE TO ENGLISH EDITION

The Standard Specifications for Design and Construction of Concrete Structures consist of the following five parts,

Part 1 Design

Part 2 Construction

Part 3 Paving

Part 4 Dam

Part 5 JSCE Standards

Of above, Part 1 and Part 2 have been translated into English.

Part 3 covers the design and construction of concrete slabs for concrete paving, and Part 4 covers those of dam concrete.

Part 5 (JSCE Standards) contains materials' quality and testing methods specified by the JSCE Concrete Committee.

Translation into English was made to be as faithful to the original edition as possible under the responsibility of the Subcommittee on English Edition of the Standard Specifications established in the Concrete Committee.

The English edition was created as a result of the devoted efforts of the subcommittee members. For publishing it, kind cooperations were given by the Editing Department of the JSCE and Gihodo Corporation. We sincerely extend our appreciation to them.

December, 1987

Hajime OKAMURA, Chairman
Jun YAMAZAKI, Secretary
Subcommittee on English Edition
of the Standard Specifications

CHAPTER 2 GENERAL REQUIREMENTS

2.1 Objectives of design

Structures shall be designed and constructed safely and economically as well as properly for the purposes. The objective of design is to assure that the structure and the members possess adequate safety against all loads during both service and construction stages, and provide their functions efficiently for ordinary services. In addition, it shall be considered that structures possess sufficient durability, and match the environment during lifetime.

2.2 Design lifetime

The design lifetime of a structure shall be determined with consideration for required service period of the structure, environmental conditions and durability of the structure.

2.3 Prerequisite of design

It is assumed for design by this specification that construction will be executed appropriately at all time at the construction sites.

2.4 Design principles

(1) It is a general principle in design to examine the structure and the structural members at all limit states where the required function is impaired and the design purposes may not be satisfied. Considerations shall be extended to construction, maintenance and repairs, and appearance.

(2) The limit states are classified into the ultimate limit state, the serviceability limit state and the fatigue limit state.

(3) The examination at the limit states may be conducted using the characteristic values of material strengths and loads, and the safety factors specified in Section 2.5.

2.5 Safety factors and modification factors

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

(2) Modification factors are material modification factors, ρ_m, and load modification factors, ρ_f, which are for converting specified values or nominal values into characteristic values.

(3) The values for safety factors and modification factors shall be given

properly for the limit state under consideration.

(4) Material factors, γ_m, shall be determined considering the unfavorable deviations of material strengths from the characteristic values, the differences of material properties between test specimens and actual structures, the influences of material properties on the limit states, and the time dependent variations of material properties. It is recommended that the material factor for concrete, γ_c, is given as 1.3 for both the ultimate and the fatigue limit states, and 1.0 for the serviceability limit state. The material factor for steel, γ_s, may be 1.0 for any limit state.

(5) Material modification factors, ρ_m, shall be determined in consideration of the differences between characteristic values and specified ones of material strengths.

(6) Load factors, γ_f, shall be determined considering the unfavorable deviations of loads from the characteristic values, the uncertainty in evaluation of loads, the influences of nature of loads on the limit states, and the variations of environmental actions.

The recommended values for load factors are as follows :

1.0—1.2 ; for the ultimate limit state.
(1.0 - 0.8 when the smaller values are critical.)

1.0 ; for both the serviceability and the fatigue limit states.

(7) Load modification factors, ρ_f, shall be determined in consideration of the differences between characteristic values and specified or nominal values of loads.

(8) Load combination factors, ψ, shall be determined in consideration of the probability of simultaneous existence of various loads.

(9) Structural analysis factors, γ_a, shall be determined in consideration of the uncertainty in structural analysis for computation of member forces.

The factors, γ_a, may be in between 1.0 and 1.2.

(10) Member factors, γ_b, shall be determined in consideration of the uncertainty in computation of capacities of members, the seriousness of dimensional error of members, and the importance or the influence of members on the entire structure when the member reaches a certain limit state.

The member factors, γ_b, depend on the equations for capacities of members.

(11) Structure factors, γ_i, shall be determined in consideration of the importance of the structure, and the influence on society when the structure would reach the limit state.

The factors, γ_i, may be in between 1.0 and 1.2.

2.6 Documentation of design calculations

(1) The calculation processes, in which the structure and the structural members are examined in terms of safety and serviceability, shall be expressed clearly in the report.

(3) The calculated results shall be precise in three significant digits unless any other directions are indicated.

CHAPTER 6 ULTIMATE LIMIT STATE

6.0 Notation

A_c : area of concrete section, or
: area of concrete shear plane in Section 6.3.7
A_e : area of concrete section surrounded by spiral reinforcement
A_m : effective area for torsion
A_{mi} : torsional effective area of each component rectangle
A_{pw} : total amount of area of prestressing steel as shear reinforcement over the interval s_p
A_{sp} : area of spiral reinforcement
A_{spe} : converted area of spiral reinforcment
A_{st} : total amount of axial reinforcement
A_{tl} : area of the longitudinal reinforcement that works effectively as torsion reinforcement
A_{tw} : area of a single transverse reinforcing that works effectively as torsion reinforcement
A_w : total amount of area of shear reinforcement over the interval s_s
$A_{w\,min}$: mimimum steel for vertical stirrups
b : width of member
b_0 : length of the shorter side of transverse reinforcement
b_w : web width
C'_d : design diagonal compressive force per unit width in concrete
C'_{ud} : design compressive capacity per unit width in concrete
d : effective depth
d_0 : length of the longer side of transverse reinforcement for a rectangular cross section and diameter of concrete cross section enclosed by transverse reinforcement for circular and annular cross sections
d_{sp} : diameter of cross section surrounded by spiral reinforcement
E_s : modulus of elasticity of reinforcement
F_d : design load
F_{ty}, F_{by} : tensile forces at yielding of upper and lower longitudinal reinforcement, respectively
f'_{cd} : design compressive strength of concrete (kgf/cm^2)
f_d : design strength
f_{ld}, f_{wd} : design yield stresses of longitudinal and transverse reinfor-

cement, respectively

f_{pyd} : design tensile yield strength of spiral reinforcement or design yield strength of prestressing steel as shear reinforcement

f_{td} : design tensile strength of concrete

f_{wyd} : design yield strength of shear reinforcement

f_{yd} : design yield strength of tension reinforcement

f'_{yd} : design compressive yield strength of axial reinforcement

H_r : horizontal bearing force obtained from the frictional and cohesion forces between the bottom surface of the structure and the ground, horizontal resistance forces of piles, horizontal bearing forces obtained from the passive earth pressure acting on the front surface of the structure. For the loads in obtaining the horizontal bearing force, nominal values are used.

H_{rd} : design resistance force for the horizontal support

H_{sd} : design horizontal force

K_t : torsional constant shown in Table 6. 4. 1

l_c, l_b : lengths of cantilever slab and net spacing of beam, respectively

M_d : design moment

M_0 : decompression moment necessary to cancel the fiber stress due to axial force at the tension fiber corresponding to design moment M_d

M_r : resistance moment obtained by using the nominal values of loads

M_{rd} : design resistance moment against turn-over at the end of the bottom of a structure

M_{sd} : design turn-over moment at the end of the bottom surface of a structure against turn-over

M_{td} : design torsional moment

M_{tcd} : design pure torsional capacity

M_{tud} : design torsional capacity given in Section 6. 4. 2.

$M_{tu\,min}$: smaller value of M_{tcud} and M_{tyd}

M_{ud} : design flexural capacity

M_x, M_y : design flexural capacity concrening x and y axis, respectively, subjected to biaxial flexural moments

N'_d : design axial compressive force

N'_{ud} : axial compressive capacity

N'_{oud} : upper limit of design axial compressive capacity

N_1, N_2 : principal in-plane forces. N_1 is in tension, not less than N_2 (Design values)

P_{ed} : effective prestress force in longitudinal tendon
p_b : balanced ratio of reinforcement
p_x, p_y : ratio of x and y reinforcement, respectively
R : computed capacity of member cross section
R_d : design capacity of member cross section
R_{sd} : design resistance
S : computed member force
S_d : design member force
S_{sd} : design applied load
s : pitch of spiral reinforcement, or
: longitudinal spacing of transverse reinforcement that works effectively as torsion reinforcement
s_p : spacing of prestressing steel as shear reinforcement
s_s : spacing of shear reinforcement
T_{xd}, T_{yd} : design tensile force per unit width in reinforcement in x and y direction, respectively
T_{xyd}, T_{yyd} : design tensile yield capacity per unit width in reinforcement in x and y direction, respectively
t : thickness of member
t_i : average thickness of flange
u : peripheral length of loaded area, or
: length of the centerline of transverse reinforcement
V_{hd} : design additional shear force produced by variation of member depth in the direction of member axis
V_{cd} : design shear capacity of member without shear reinforcement
V_{cwd} : design shear transfer capacity
V_d : design shear force
$(V_{odi})_{\min}$: minimum value of the in-plane shear capacity of each wall
V_{pcd} : design punching shear capacity
V_{ped} : design additional shear force produced by inclination of longitudinal tendon
V_r : vertical bearing force of the foundation or the piles
V_{rd} : design resistance force of the foundation or the piles
V_{sd} : design shear capacity provided by shear reinforcement, or
: design reaction force of the foundation or piles
V_{wcd} : design ultimate diagonal compressive capacity of web concrete
V_x, V_y : design shear capacity concerning x and y axis, respectively, subjected to biaxial shear
V_{yx}, V_{yy} : design uniaxial shear capacity concerning x and y axis, respectively

V_{yd} : design shear capacity

z : distance from compression resultant to centroid of tension steel

α : angle between the direction of principal in-plane force N_1 and x-reinforcement

α_c : angle between compression fiber and member axis

α_p : angle between prestressing steel as shear reinforcement (or longitudinal tendon) and member axis

α_s : angle between shear reinforcement and member axis

α_t : angle between tension steel and member axis

β_d : coefficient to consider influence of effective depth on shear capacity

β_n : coefficient to consider influence of axial force on shear capacity

β_{nt} : coefficient to consider influence of axial force on torsional moment capacity

β_p : coefficient to consider influence of longitudinal reinforcement on shear capacity

β_r : coefficient to consider influence of loaded area on punching shear capacity

γ_a : structural analysis factor

γ_b : member factor

γ_i : structure factor

γ_h : safety factor for the horizontal stability determined by considering the variation of the horizontal bearing force from the characteristic value in the unfavorable direction, and others

γ_0 : safety factor for turn-over, determined by considering the variation of the nominal values of loads in the unfavorable direction, the uncertainty of the calculation method of loads, the uncertainty involved in the calculation of the resistance moment due to the deformation of the foundation, and others

γ_v : safety factor for the vertical stability determined by considering the variation of the vertical bearing force from the characteristic value in the unfavorable direction, and others

λ_t : one-side effective flange width for torsion

ξ : M_{tydi}/A_{mi} of the component rectangle with maximum torsional effective cross section

ε'_{cu} : ultimate compressive strain of concrete

σ_{pw} : tensile stress in prestressing steel as shear reinforcement at yield of other shear reinforcement

σ'_{nd} : average compressive normal stress to the shear plane (kgf/cm²), or

: average working compressive stress by axial force

σ_{td} : average tensile stress due to axial tensile forces

σ_{wpe} : effective tensile stress in prestressing steel as shear reinforcement

θ : angle between shear plane and reinforcement

6. 1 General

(1) Examination of ultimate limit state for failure of cross sections shall be made by confirming that the ratio of design capacity of member cross section R_d to design member force S_d is not less than a structure factor γ_i.

$$R_d/S_d \geqq \gamma_i \qquad (6.1.1)$$

(i) The design capacity of cross section R_d shall be determined by dividing computed capacity of member cross section R by a member factor γ_b. The computed capacity of member cross section R is obtained based on design material strengths f_d.

$$R_d = R(f_d)/\gamma_b \qquad (6.1.2)$$

Generally, the member factor γ_b prescribed in Section 6. 2 and other sections forth going may be used.

(ii) The design member force S_d shall be a member force S multiplied by a structural analysis factor γ_a. The computed member force S shall be based on design loads F_d.

$$S_d = \gamma_a S(F_d) \qquad (6.1.3)$$

(2) Examination of ultimate limit state for rigid body motion stability of structures shall be made by confirming that the ratio of design resistance R_{sd} to design applied load S_{sd} is not less than the structure factor γ_i.

$$R_{sd}/S_{sd} \geqq \gamma_i \qquad (6.1.4)$$

(3) Examination of ultimate limit states for displacement, deformation, collapse mechanism and others shall be made by appropriate methods, if necessary.

6. 2 Flexural moment and axial forces

6. 2. 1 General

(1) Members subjected to axial compressive force shall be designed so that the ratio of the upper limit of design axial compressive capacity N'_{oud} to design axial compressive force N'_d is not less than the structure factor γ_i. The member factor γ_b is generally taken as 1.3.

(2) Design for combined flexural moment M_d and axial force N'_d shall be made by confirming that the design flexural capacity M_{ud} based on the constant $e = M_d/N'_d$ satisfies Eq. (6. 1. 1). In this case, the member factor γ_b may be generally taken as 1.15.

(3) When the small effect of axial force, where e/h may be equal to or greater than 10 (h : depth of cross section), is assumed, ultimate capacity of member cross sections may be calculated as flexural members.

6.2.2 Design capacity of member cross section

(1) When design capacity of member cross sections subjected to flexural moment or combined flexural moment and axial loads are computed in directions corresponding with member forces, design shall be made according to the assumptions (i)-(iv) below. The member factor γ_b may be generally taken as 1.15.

(i) Fiber strain is proportional to the distance from neutral axis.

(ii) Tensile stress of concrete is neglected.

(iii) Stress-strain relationship of concrete is according to Fig. 3.2.1 as a rule.

(iv) Stress-strain relationship of steel is according to Fig. 3.3.1 as a rule.

(2) Compressive stress distribution of concrete may be assumed equivalent to rectangular compressive stress distribution (equivalent stress block) as shown in Fig. 6.2.1 except when the compressive strain is distributed over the member cross section.

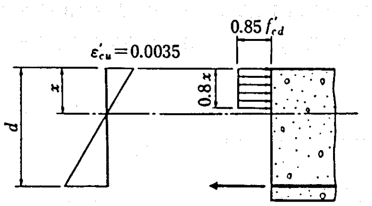


Fig. 6.2.1 Equivalent stress block

(3) The design capacity of member cross sections subjected to biaxial flexural moments and axial loads may be determined by the assumptions in (1).

6.2.3 Structural details

(1) Minimum reinforcement

(i) For a reinforced concrete member with a dominant effect of axial force, longitudinal reinforcement of not less than 0.8 % of concrete area, which is necessary according to computation, shall be arranged.

The concrete area necessary according to computation is the minimum concrete area which is necessary to support the axial force only.

Even in the case of concrete area greater than that necessary according to computation, it is desired to arrange longitudinal reinforcement not less than 0.15 % of the concrete area.

(ii) For a linear member with a dominant effect of flexural moment, longitudinal tension reinforcement not less than 0.2 % of concrete area shall be arranged as a general rule. For a T-type cross section, longitudinal reinforcement equal to or greater than 0.3 % of effective concrete area shall be placed. The effective area of concrete is the product of an effective depth d of a cross section and a web width b_w.

(2) Maximum reinforcement

(i) For a reinforced concrete member with a dominant effect of axial force, longitudinal reinforcement shall be equal to or less than 6 % of the concrete area in principle.

(ii) For a linear member with a dominant effect of flexure moment, tension reinforcement shall be equal to or less than 75 % of the balanced strain reinforcement ratio in principle.

6.3 Shear

6.3.1 General

(1) Design for shear shall be made by appropriate method in taking account of kinds of members such as linear and planar ones and acting direction of shear. Generally, examination may be made by items (2)-(5) below.

(2) For a linear member, the safety for shear shall be examined regarding design shear capacities V_{yd} and V_{wcd} obtained by Section 6.3.3, respectively.

(3) For a planar member subjected to transverse shear, examination for transverse shear shall be made according to the linear member. Punching shear failure shall be examined in accordance with Section 6.3.4 provided that a concentrated load V_d is applied on a localized area of the member.

(4) For a planar member subjected to in-plane shear, examination for in-plane shear shall be made according to Section 6.3.5 and Section 6.3.6.

(5) If shear force V_d must be transfered on placing joints or planes where the probability of crack occurence is large, examination for direct shear transfer on the shear planes shall be made according to Section 6.3.7.

6.3.2 Design shear forces of linear members

The design shear force in a linear member where the member depth is variable shall be calculated by subtracting components V_{hd} of bending compressive and tensile forces parallel to the shear force. The value of V_{hd} may be obtained by Eq. (6.3.1).

$$V_{hd}=(M_d/d)(\tan\alpha_c+\tan\alpha_t) \qquad (6.3.1)$$

The effective depth d for the cases where prestressing tendon and tension reinforcement coexist is taken as the effective depth corresponding to the composite cross section of steel. The angles α_c and α_t are defined possitive when the effective depth increases according to the increase in absolute value of moment.

6.3.3 Design shear capacity of linear members

(1) The design shear capacity V_{yd} may be obtained by Eq. (6.3.2). When both bent bars and stirrups are arranged as shear reinforcement, at least 50 % of the shear force provided by shear reinforcement shall be carried by stirrups.

$$V_{yd}=V_{cd}+V_{sd}+V_{ped} \qquad (6.3.2)$$

where, V_{cd} : design shear capacity of linear members without shear reinforcement, obtained by Eq. (6.3.3).

$$V_{cd}=f_{vcd}\cdot b_w\cdot d/\gamma_b \qquad (6.3.3)$$

$$f_{vcd}=0.9\,\beta_d\cdot\beta_p\cdot\beta_n\cdot\sqrt[3]{f'_{cd}}\ (\text{kgf/cm}^2) \qquad (6.3.4)$$

$\beta_d=\sqrt[4]{100/d}$ (d : cm), when $\beta_d>1.5$, β_d is taken as 1.5

$\beta_p=\sqrt[3]{100\,p_w}$ when $\beta_p>1.5$, β_p is taken as 1.5

$\beta_n=1+M_0/M_d$ $(N'_d\geq 0)$ when $\beta_n>2$, β_n is taken as 2.

$=1+2\,M_0/M_d$ $(N'_d<0)$ when $\beta_n<0$, β_n is taken as 0.

where, γ_b : 1.3 may be used in general.

$p_w=A_s/(b_w\cdot d)$

V_{sd} : design shear capacity carried by shear reinforcing steel and obtained by Eq. (6.3.5).

$$V_{sd}=[A_w f_{wyd}(\sin\alpha_s+\cos\alpha_s)/s_s + A_{pw}\sigma_{pw}(\sin\alpha_p+\cos\alpha_p)/s_p]z/\gamma_b \qquad (6.3.5)$$

where, $\sigma_{pw}=\sigma_{wpe}+f_{wyd}\leq f_{pyd}$

f_{wyd} : not greater than 4 000 kgf/cm^2.

z : generally, may be taken as $d/1.15$

γ_b : 1.15 in general

V_{ped} : component of effective tensile force of longitudinal tendon parallel to the shear force and obtained by Eq. (6.3.6).

$$V_{ped}=P_{ed}\sin\alpha_p/\gamma_b \qquad (6.3.6)$$

where, γ_b : 1.15 in general.

(2) When linear members are supported directly, no further examination for V_{yd} may be required over the span range of one-half the total depth of members h from the support faces. In this range, the shear reinforcement not less than reinforcement necessary for the cross section located $h/2$ from the support faces shall be arranged, and the depth at the support face may be used providing variable member depth. Provided that, the part of haunch whose gradient does not exceed 1 : 3 is considered to be effective.

When the check for planar members subjected to transverse shear is made as linear members by Section 6.3.1(3), the computation of the design shear capacity near the supports may be conducted by well-founded appropriate methods.

(3) The design diagonal compressive capacity V_{wcd} of web concrete to shear force may be determined by Eq. (6.3.7).

$$V_{wcd}=f_{wcd}\cdot b_w\cdot d/\gamma_b \qquad (6.3.7)$$

where, $f_{wcd}=4\sqrt{f'_{cd}}$ (kgf/cm^2)

γ_b : 1.3 in general

(4) Web width of members

(i) For the cases where a diameter of a duct of prestressed concrete members is equal to or greater than 1/8 times of the web width, the assumed width used in Eq. (6.3.3) shall be less than the actual web width b_w. As a rule, the web width may be reduced one-half the total amount of duct diameters ϕ spaced in the cross section to $b_w-1/2\sum\phi$.

(ii) For members with variable web widths in the direction of member depths 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 is taken as b_w. For members with solid or hollow circular cross sections, the web width will be defined as the side length of the square with the same area or the total width of webs of the square box having the same area respectively. In these cases, the area of axial 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 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.

These definitions of area for axial tensile steel shall not be applied to the computation for the flexural capacity.

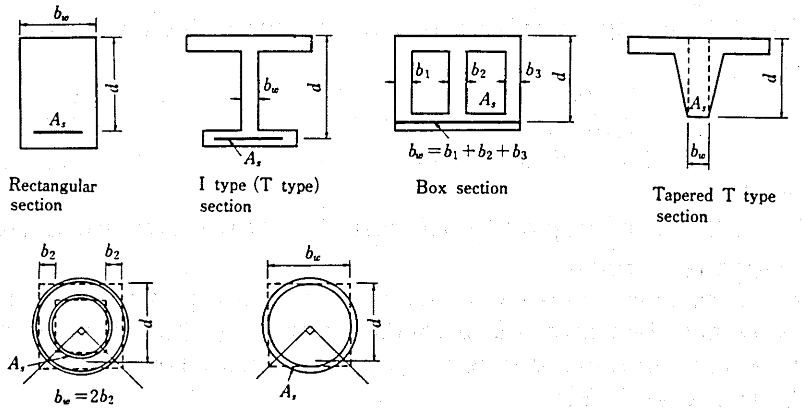


Fig. 6. 3. 1 Definitions of b_w and d concerning various shapes of cross sections

6. 3. 4 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} may be determined by Eq. (6. 3. 8).

$$V_{pcd}=f_{pcd}\cdot u_p d/\gamma_b \qquad (6.3.8)$$

where,

$$f_{pcd}=0.6\beta_d\cdot\beta_p\cdot\beta_r\cdot\sqrt{f'_{cd}} \quad \text{(kgf/cm}^2\text{)} \qquad (6.3.9)$$

$\beta_d=\sqrt[4]{100/d}$ (d : cm), when $\beta_d>1.5$, β_d is taken as 1.5

$\beta_p=\sqrt[3]{100\,p}$ when $\beta_p>1.5$, β_p is taken as 1.5

$\beta_r=1+1/(1+0.25\,u/d)$

u_p : peripheral length of the design cross section which is located $d/2$ from the loaded area

d, p : effective depth and reinforcement ratio which are defined as the average values for the reinforcement in two directions.

γ_b : 1.3 in genral

(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 taken into account.

(3) When the loads are applied eccentrically to the loaded area, the effects of flexure and torsion shall be considered.

6. 3. 5 Design member forces of planar members subjected to in-plane forces

For orthogonally reinforced concrete planar members subjected to in-plane forces, tensile forces T_{xd}, T_{yd} obtained by Eq. (6. 3. 10) and Eq. (6. 3. 11) in each reinforcing direction and diagonal compressive force C'_d

by Eq. (6.3.12) applied to concrete may be used as the design in-plane forces.

$$T_{xd}=N_1\cos^2\alpha+N_2\sin^2\alpha+(N_1-N_2)\sin\alpha\cos\alpha \quad (6.3.10)$$
$$T_{yd}=N_1\sin^2\alpha+N_2\cos^2\alpha+(N_1-N_2)\sin\alpha\cos\alpha \quad (6.3.11)$$
$$C'_d=2(N_1-N_2)\sin\alpha\cos\alpha \quad (6.3.12)$$

where, $\alpha\leq45^\circ$.

6.3.6 Design capacity of planar members subjected to in-plane forces

For examination of in-plane forces to design member forces by Section 6.3.5, design yield capacities of reinforcement T_{xyd}, T_{yyd} and the design compressive capacity of concrete C'_{ud} may be obtained by Eqs. (6.3.13), (6.3.14) and (6.3.15).

(1) Design yield capacity of reinforcement

$$T_{xyd}=p_x\cdot f_{yd}\cdot b\cdot t/\gamma_b \quad (6.3.13)$$
$$T_{yyd}=p_y\cdot f_{yd}\cdot b\cdot t/\gamma_b \quad (6.3.14)$$

where,

p_x and p_y : x and y reinforcement ratios (A_s/bt)

γ_b : may be generally taken as 1.15

(2) Design compressive capacity of concrete

$$C'_{ud}=f'_{ucd}\cdot b\cdot t/\gamma_b \quad (6.3.15)$$

where,

$$f'_{ucd}=9\sqrt{f'_{cd}} \quad (\text{kgf/cm}^2) \quad (6.3.16)$$

γ_b : may be generally taken as 1.3

6.3.7 The design capacity for shear transfer

When reinforcement is spaced over a shear plane, the design capacity for shear transfer V_{cwd} on the shear plane may be computed by Eq. (6.3.17).

$$V_{cwd}=\mu(pf_{yd}\sin^2\theta+\sigma'_{nd})A_c/\gamma_b \quad (6.3.17)$$

where, $\mu=1.1\sqrt{f'_{cd}}/(pf_{yd}\sin^2\theta+\sigma'_{nd})^{2/3}\leq0.12\sqrt{f'_{cd}}$

p : reinforcement ratio along the shear plane, where reinforcement with sufficient developement length in both directions from the shear plane is taken into account.

γ_b : 1.3 in general

6.3.8 Structural details

(1) For a linear member, stirrups not less than 0.15% of the concrete area shall be arranged over the member length. The spacing shall not be greater than 3/4 of the effective depth of a member nor 40 cm in general. This provision needs not be applied to planar members.

(2) For a linear member, if reinforcement is required by computation, then stirrups shall be arranged with space equal to or less than 30 cm and 1/2 of the effective depth, within the range necessary, and also

within the length of the effective depth at its each outer side when shear reinforcement is calculatedly necessary.

(3) Ends of stirrups and bent bars shall be embedded in compressive side of concrete.

6.4 Torsion

6.4.1 General

(1) For structural members not significantly influenced by torsional moment and those subject to compatibility torsional moment, all of the examinations specified in Section 6.4 may be deleted. Here, the structural members not significantly influenced by torsional moment means those of which the ratio of the design torsional moment, M_{td}, to the design pure torsional capacity, M_{tcd}, specified in Section 6.4.2 for no torsion reinforcement is less than 0.2 for all of the cross sections.

(2) When the design torsional moment, M_{td}, satisfies Eq. (6.4.1), the examinations specified in Section 6.4.3 may be deleted, if the minimum torsion reinforcement is provided in accordance with Section 6.4.4.

$$0.5\, M_{tud}/M_{td} \geq \gamma_i \qquad (6.4.1)$$

(3) When the design torsional moment, M_{td}, does not satisfy Eq. (6.4.1), torsion reinforcement shall be provided in accordance with Section 6.4.3.

6.4.2 Design torsional capacity for members without torsion reinforcement

(1) When a linear member without torsion reinforcement is subject to torsional moments only, the design torsional capacity, M_{tud}, is obtained by Eq. (6.4.2).

$$M_{tud} = M_{tcd} \qquad (6.4.2)$$

where

$$M_{tcd} = \beta_{nt} \cdot K_t \cdot f_{td} / \gamma_b \qquad (6.4.3)$$

K_t : See Table 6.4.1

$\beta_{nt} = \sqrt{1 + \sigma'_{nd}/(1.5\, f_{td})}$

$\sigma'_{nd} \leq 7\, f_{td}$

$\gamma_b = 1.3$ (in general)

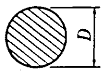
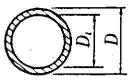
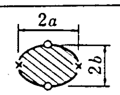
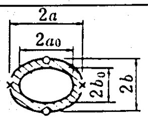
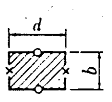
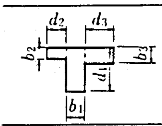
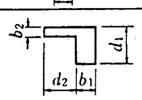
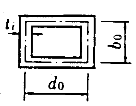
(2) When flexural moment, M_d, is applied simultaneously, the design torsional capacity, M_{tud}, may be obtained by Eq. (6.4.4).

$$M_{tud} = M_{tcd}(0.2 + 0.8\sqrt{1 - M_d/M_{ud}}) \qquad (6.4.4)$$

(3) When shear force, V_d, is applied simultaneously, the design torsional capacity, M_{tud}, may be obtained by Eq. (6.4.5).

$$M_{tud} = M_{tcd}(1 - 0.8\, V_d/V_{yd}) \qquad (6.4.5)$$

Table 6. 4. 1 Fators for torsion

Shape of cross section	K_t	Remark
	$\frac{\pi D^3}{16}$	
	$\frac{\pi(D^4-D_i^4)}{16D}$	
	$\bigcirc: \pi a b^2/2$ $\times: \pi a^2 b/2$	
	$\bigcirc: \pi a b^2(1-q^4)/2$ $\times: \pi a^2 b(1-q^4)/2$	$\bigcirc: q=a_o/a$ $\times: q=b_o/b$
	$\bigcirc: b^2 d/\eta_1$ $\times: b^2 d/(\eta_1 \eta_2)$	$\eta_1=3.1+\frac{1.8}{d/b}$ $\eta_2=0.7+\frac{0.3}{d/b}$
 	$\Sigma \frac{b_i^2 d_i}{\eta_{1i}}$ 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.
	$2A_m t_t$ 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_t : Web thickness

6. 4. 3 Design torsional capacity for members with torsion reinforcement

(1) The design capacity for diagonal compression failure of web concrete against torsion, M_{tcud}, is obtained by Eq. (6. 4. 6).

$$M_{tcud}=K_t \cdot f_{wcd}/\gamma_b \qquad (6.4.6)$$

where

$f_{wcd}=4\sqrt{f'_{cd}}$ (kgf/cm²) (6.4.7)

K_t : See Table 6.4.1

$\gamma_b=1.3$ (in general)

(2) The design torsional capacity of rectangular, circular, and annular cross sections, M_{tyd}, is obtained by Eq. (6.4.8).

$$M_{tyd}=2A_m\sqrt{q_w \cdot q_l}/\gamma_b \quad (6.4.8)$$

where

A_m : $b_0 d_0$ (rectangular), $\pi d_0^2/4$ (circular, annular)

$q_w=A_{tw}\cdot f_{wd}/s$

$q_l=\sum A_{tl}\cdot f_{ld}/u$

u : $2(b_0+d_0)$ (rectangular), πd_0 (circular, annular)

$\gamma_b=1.3$ (in general)

When $q_w \geq 1.25\,q_l$, $q_w=1.25\,q_l$, and when $q_l \geq 1.25\,q_w$, $q_l=1.25\,q_w$.

(3) The design torsional capacity for T, L, and I sections may be obtained by dividing the cross section into rectangular sections and summing up in accordance with the following (i)—(iv) the torsional capacity of each component rectangle, M_{tydi}, given by Eq. (6.4.8). However, each M_{tydi} shall not exceed $\xi \cdot A_{mi}$.

(i) A_{mi} may be considered as the area enclosed by the transverse reinforcement, as shown in Fig. 6.4.1.

(ii) 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 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 of flange is different, the smaller value of reinforcement shall be taken as the limit.

(iv) The one-side effective flange width for torsion, λ_t, is computed by Eq. (6.4.9).

$$\lambda_t=3\,t_i \quad (6.4.9)$$

where

$\lambda_t \leq l_c$ (for the cantilever part)

$\lambda_t \leq l_b/2$ (for the middle part)

(4) A box section for which the minimum 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, (7) below shall be followed 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.

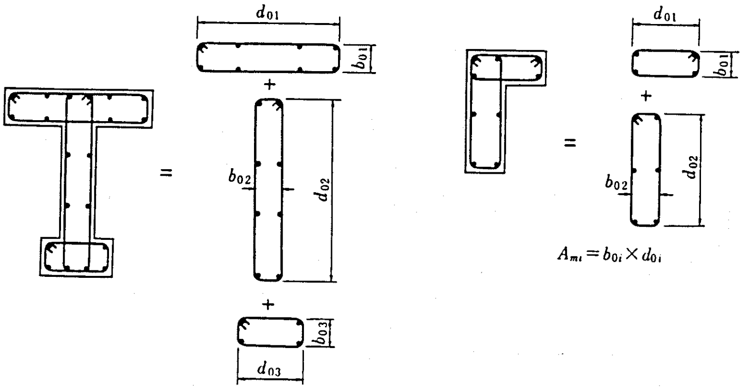


Fig. 6.4.1 Calculation scheme of A_{mi} for T and L sections

(5) When a member is subject to flexural moment, M_d, simultaneously, the design torsional capacity, M_{tud}, for rectangular, circular, and annular sections may be obtained by Eqs. (6.4.10), (6.4.11), and (6.4.12).

$$M_{tud}=M_{tu\,\min} \quad (\text{for } M_{ud}\geq M'_{ud} \text{ and } |M_d|\leq M_{ud}-M'_{ud}) \qquad (6.4.10)$$

$$M_{tud}=(M_{tu\,\min}-0.2\,M_{tcd})\sqrt{(M_{ud}-|M_d|)/M'_{ud}}+0.2\,M_{tcd} \quad (\text{for } M_{ud}\geq M'_{ud} \text{ and } M_{ud}-M'_{ud}\leq |M_d|\leq M_{ud}) \qquad (6.4.11)$$

$$M_{tud}=(M_{tu\,\min}-0.2\,M_{tcd})\sqrt{1-|M_d|/M_{ud}}+0.2\,M_{tcd} \quad (\text{for } M_{ud}<M'_{ud} \text{ and } |M_d|\leq M_{ud}) \qquad (6.4.12)$$

where

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, simultaneously, the design torsional capacity, M_{tud}, for rectangular, circular, and annular cross sections, may be obtained by Eq. (6.4.13).

$$M_{tud}=M_{tu\,\min}(1-V_d/V_{yd})+0.2\,M_{tcd}V_d/V_{yd} \qquad (6.4.13)$$

(7) For a box section for which 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 design torsional capacity, M_{tyd}, may be obtained by Eq. (6.4.14).

$$M_{tyd}=2\,A_m(V_{odi})_{\min} \qquad (6.4.14)$$

In this case, for the joint part of each wall and the anchorage method of reinforcement, the structural details in Section 6.4.4 shall be adopted.

When a member is subject to flexural moments or shear forces together with torsional moments, the torsional capacity may be computed in the similar way as the rectangular cross section.

6. 4. 4 Structural details

(1) The minimum torsion reinforcement for a prismatic member is calculated by Eq. (6. 4. 15).

$$\sum A_{tl} = M_{tud} \cdot u / (3 \cdot A_m \cdot f_{ld}) \quad \text{(longitudinal reinforcement)}$$

$$A_{tw} = M_{tud} \cdot s / (3 \cdot A_m \cdot f_{wd}) \quad \text{(transverse reinforcement)} \qquad (6.4.15)$$

(2) The torsion reinforcement is composed of closed transverse reinforcement and the longitudinal reinforcement perpendicular to the transverse reinforcement, as shown in Fig. 6. 4. 2.

(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. When the distance between the position of the transverse reinforcement and the outer perimeter of the cross section is more than 0. 2 times the member width, as a rule, it is not considered as transverse reinforcement for torsion reinforcement.

(5) For a rectangular cross section, at least one longitudinal reinforcing bar shall be arranged at each corner of the cross section, as shown in Fig. 6. 4. 2(a). For a circular cross section, at least six longitudinal reinforcing bars shall be arranged at equal intervals, as shown in Fig. 6. 4. 2(b).

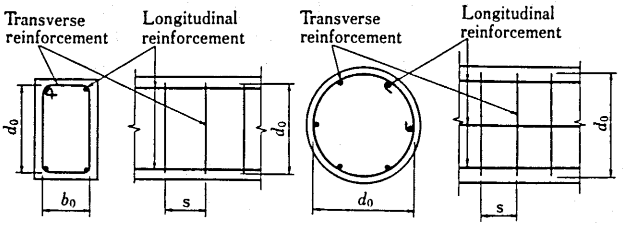


Fig. 6. 4. 2 Arrangement of torsion reinforcement

(6) The transverse reinforcement for a box section shall be arranged as shown in Fig. 6. 4. 3.

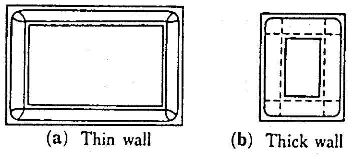


Fig. 6. 4. 3 Arrangement of transverse reinforcement for a box section

(7) The torsion reinforcement shall be arranged in the portions where the calculated volume of reinforcement is required and in the adjacent segments whose length is equal to the overall height or the diameter of the cross section of the member. For the rest of the member, the minimum amount of reinforcement is required.

6. 5 Rigid body stability

The safety for rigid body stability in the design of ground-based structures is examined for turn-over, and horizontal and vertical stability.

(1) The safety for turn-over is examined by confirming that the ratio between the design resistance moment and the design turn-over moment at the end of the bottom of structures is not less than the structure factor, γ_i. The design resistance moment is calculated by dividing the resistance moment obtained from the nominal values of loads by the safety factor, γ_o.

(2) The safety for the horizontal stability is examined by confirming that the ratio between the design horizontal resistance force and the design horizontal force which makes the structure slide along the bottom surface is not less than the structure factor, γ_i.

The design horizontal resistance force is calculated by dividing the horizontal bearing force obtained from the friction and cohesion forces between the bottom surface of the structure and the foundation ground, and the horizontal resistance of piles and the passive earth pressure at the front surface of the structure by a safety factor, γ_h.

(3) The safety for the vertical stability is examined by confirming that the ratio between the design bearing force of the ground in contact with the structure or that of the piles and the design reaction force of the ground or the piles is not less than the structure factor, γ_i.

The design vertical bearing force of the ground or the piles is computed by dividing the vertical bearing force of the ground or the piles obtained from loading tests or mechanics formula by a safety factor, γ_v.

CHAPTER 7 SERVICEABILITY LIMIT STATES

7.0 Notation

A_b : area of bent bar
A_w : area of one unit of shear reinforcement
c : concrete cover (cm)
c_s : center-to-center distance of steel (cm)
d : effective depth
f_{wd} : design yield strength of transverse torsion reinforcement
I_{cr} : moment of inertia of cracked section transformed to concrete
I_e : effective moment of inertia for computation of displacement and deformation
I_g : moment of inertia of gross cross section
k_1 : constant to take into account the influence of bond characteristics of steel, which may be set equal to 1.0 for deformed bars, 1.3 for plain bars and prestressing steel
k_2 : constant to take into account differences of degrees which crack widths by permanent loads and those by variable loads which affect the corrosion of reinforcement
M_{crd} : cracking moment, which causes the design bending stress given by Eq. (3.2.2) at the tension fiber
$M_{d\,max}$: maximum design moment in computation of displacement and deformation
M_{tcd} : design pure torsional capacity without torsion reinforcement given by Eq. (6.4.3)
M_{tpd} : design torsional moment produced by permanent loads
M_{tyd} : design torsional capacity given by Eq. (6.4.8)
S_p : member force produced by permanent loads
S_r : member force produced by variable loads
s : spacing of shear reinforcement
s_b : spacing of bent bar
V_{cd} : design shear capacity of concrete given by Eq. (6.3.3)
V_{pd} : design shear force produced by permanent loads
V_{yd} : design shear capacity given by Eq. (6.3.2)
z : distance from compression resultant to centroid of tension reinforcement
α : angle between shear reinforcement and longitudinal axis of member
α_b : angle between bent bar and member axis

ε'_{cs} : compressive strain for evaluation of increment of crack width due to drying shrinkage and creep in concrete

φ : creep factor

δ_t : long-term displacement and deformation

δ_{ep} : short-term displacement and deformation due to permanent load

ϕ : diameter of steel bar (cm)

7.1 General

(1) In order to preserve sufficient functions of structures or members during their lifetime, serviceability limit states for cracks, displacements, deformations, and vibrations shall be set up and examined by appropriate design methods.

(2) Other required serviceability limit states shall be set up and examined by appropriate methods if necessary.

7.2 Computation of stress

In serviceability limit states, stresses in concrete and reinforcement in the cross section of members shall be computed based on the following assumptions (i)~(iv).

(i) Fiber strain is proportional to the distance from the neutral axis.

(ii) Both concrete and reinforcement are elastic bodies.

(iii) Tensile stress in concrete is negligible.

(iv) Elastic moduli of concrete and reinforcement are given in accordance with CHAPTER 3.

7.3 Examination for cracks

7.3.1 General

(1) It shall be examined by an appropriate method that the functions, durability, and appearances of structures are not impaired by cracking in concrete.

(2) This clause may be applied to the examination for crack due to flexural moment, shear force, torsional moment, and axial load.

(3) Examination of cracks for durability shall be made as a rule by controlling crack widths on the surface of concrete to stay within the permissible crack widths for corrosion of reinforcement determined by environmental conditions, concrete cover, service life, and other factors. Examination of cracks may not be required, in general, when the service life of structures is apparently short, the surface of structures is protected, and the structures are temporary works.

(4) When watertightness is important, examination for cracking shall be made by an appropriate method. In this case, the method for

examination may be the one similar to that given in (3), setting up the permissible crack widths to certain values which are considered acceptable for required watertightness.

(5) When appearances of structures are important, a method for examination may be the one similar to that indicated in (3), setting up the permissible crack widths considered acceptable to appearances.

7. 3. 2 Classification of environmental conditions

(1) When the examination for limit state of crack width determined by durability, environmental conditions where the structure is exposed shall be taken into account.

(2) Environmental conditions regarding the corrosion of reinforcement are, in general, classified into "normal condition", "corrosive condition", and "severely corrosive condition" as shown in Table 7. 3. 1.

Table 7. 3. 1 Classification of environmental conditions

Normal environment	Outdoors of ordinaly condition, underground, etc.
Corrosive environment	1. As compared with normal environment, reinforcement subject to detrimental influences, such as severe alternate wetting and drying and structure below level of underground water containing harmful substances. 2. Marine structures submerged in seawater and structures exposed to mild marine environment, etc.
Severely corrosive environment	1. Reinforcement subject to detrimental influences considerably. 2. Marine structures exposed to tides, sprashes, severe ocean winds, etc.

7. 3. 3 Permissible crack width

(1) Permissible crack width, w_a, is determined as a rule by considering, such as, the use purposes of structures, environment conditions, and conditions of members.

(2) Permissible crack widths for corrosion of reinforcement may be determined, in general, as the values proposed in Table 7. 3. 2 according to environmental conditions, concrete cover and type of reinforcement. The concrete cover, c, in Table 7. 3. 2 may be applicable when it is not greater than 10 cm.

Table 7.3.2 Permisible crack width w_a (cm)

Type of reinforcement	Environmental conditions for corrosion of reinforcement		
	Normal environment	Corrosive environment	Severely corrosive environment
Deformed bars and plain bars	0.005 C	0.004 C	0.0035 C
Prestressing steel	0.004 C	———	———

7.3.4 Examination for flexural cracks

(1) Examination for flexural cracks shall be made, in general, that the crack width, w, obtained from Eq. (7.3.1) is not greater than the permissible crack width, w_a, in Table 7.3.2.

Increase of stress in reinforcing bars, σ_{se}, and that in prestressing steel, σ_{pe}, shall be obtained by member force, S_e, which may be derived from Eq. (7.3.2).

$$w = k_1\{4c + 0.7(c_s - \phi)\}\left[\frac{\sigma_{se}}{E_s}\left(\text{or } \frac{\sigma_{pe}}{E_p}\right) + \varepsilon'_{cs}\right] \qquad (7.3.1)$$

$$S_e = S_p + k_2 S_r \qquad (7.3.2)$$

(2) Reinforcing bars and prestressing steel to be examined for flexural cracks are, as a rule, the tension reinforcement located nearest to the concrete surface. Stresses shall be obtained in accordance with Section 7.2.

7.3.5 Examination for shear cracks

(1) For members subject to shear forces, it may not be required to examine shear cracks when the design shear force, V_d, is smaller than the design shear capacity of concrete, V_{cd}.

(2) When examination for shear crack is necessary, examination shall be made by an appropriate method.

7.3.6 Examination for torsion cracks

(1) When design torsional moment in member, M_{td}, subjected to equilibrium torsional moment is less than design torsional moment capacity in member, M_{tud}, obtained from Section 6.4.2, which does not take into account the torsion reinforcement, and when the member is subjected to compatibility torsional moment, examination for torsion cracks need not be conducted.

(2) When examination for torsional cracks is necessary, examination shall be made by an appropriate method.

7.3.7 Structural details

(1) When it is necessary, additional reinforcement for the member shall be arranged to control cracking due to the change in temperature, drying shrinkage and others, in addition to the required reinforcement to control cracking due to loads.

(2) Reinforcement for crack control shall be arranged dispersively and closely adjacent to cross section of required members. In this case, the diameter and spacing of reinforcement shall be small as much as possible.

(3) Spacing of longitudinal reinforcement and transverse reinforcement shall not be greater than 30 cm as a rule.

7.4 Examination for displacement and deformation

7.4.1 General

(1) Displacements and deformations of structures or members shall be examined by an appropriate design method so that their functionality, serviceability, durability, and appearance of structures are not impaired.

(2) Short-term displacements and deformations, and long-term displacements and deformations shall be considered separately. Short-term displacements and deformations mean displacements and deformations that occur immediately on application of load. Long-term displacements and deformations are computed by adding short-term displacements and deformations, and displacements and deformations caused by permanent load.

(3) Short-term displacements and deformations, and long-term displacements and deformations of structures or members shall not be greater than each permissible displacement and deformation.

7.4.2 Permissible displacement and deformation

Permissible displacements and deformations of structures or members shall be determined considering the type and purpose of use of structures, and the type of loads.

7.4.3 Examination of displacement and deformation

(1) Short-term displacements and deformations of concrete members without flexural cracking may be computed using the theory of elasticity assuming that the gross cross section is effective.

(2) Short-term displacements and deformations of concrete members with flexural cracking shall be computed taking into account the reduction in stiffness due to cracking.

(3) Long-term displacements and deformations of concrete members shall be computed taking into account the effects of creep and drying shrinkage of concrete under permanent load.

CHAPTER 8 FATIGUE

8.0 Notation

A_b : area of bent bar within distance s_b
A_w : area of shear reinforcement within distance s
d : effective depth
F_{rd} : design variable load
f_{rd} : design fatigue strength
f_{rk} : characteristic value of fatigue strength for materials
N : equivalent number of cycles, or fatigue life
N_i : number of cycles to failure
n_i : number of cycles
R_{rd} : design fatigue strength of cross section
$R_r(f_{rd})$: fatigue strength of cross section of member
S_{rd} : design variable member force
$S_r(F_{rd})$: variable member force produced by design variable load
s : spacing of vertical stirrups
s_b : spacing of bent bars
V_{cd} : design shear capacity without shear reinforcement
V_{pd} : design shear force produced by permanent load
V_{pcd} : design punching shear capacity provided by concrete slab
V_{rcd} : design shear fatigue capacity of beam member without shear reinforcement
V_{rd} : design shear force produced by variable load
V_{rpd} : design punching shear fatigue capacity
z : distance from compression resultant to centroid of tension reinforcement
α_b : angle between bent bar and longitudinal axis of member
α_s : angle between shear reinforcement and longitudinal axis of member
γ_a : structural analysis factor
γ_b : member factor
γ_i : structure factor
γ_m : material factor
σ_p : stress due to permanent load
σ_{rd} : design variable stress
σ_{wpd} : design stress in shear reinforcement due to permanent load
σ_{wrd} : design stress in shear reinforcement due to variable load

8.1 General

(1) Examination of safety for fatigues shall be performed when the ratio of variable loads to total loads, or number of applied cycles are large.

(2) For beams, examination for fatigues shall be performed for tension and shear reinforcement, in general. However, for concrete members without shear reinforcement, or with lightweight aggregate, and under wet conditions, the examination for fatigue shall be conducted for concrete.

(3) For slabs, examination for fatigue in general shall be performed for tension reinforcement and punching shear.

(4) For columns, generally, examination for fatigue is not required. When the influence of flexural moment or axial tensile force are large, however, examination for fatigue shall be performed in accordance with that for beams.

8.2 Examination for fatigue

(1) In the examination for fatigue limit state, it shall be confirmed as a rule that the ratio of design fatigue strength, f_{rd}, to design variable stress, σ_{rd}, is greater than the structure factor, γ_i.

$$f_{rd}/\sigma_{rd} \geq \gamma_i \qquad (8.2.1)$$

where design fatigue strength, f_{rd}, is equal to the characteristic value of fatigue strength for materials, f_{rk}, divided by the material factor, γ_m.

$$f_{rd} = f_{rk}/\gamma_m \qquad (8.2.2)$$

(2) It is alternative to examine the fatigue limit state by confirming that the ratio of design fatigue capacity of cross section, R_{rd}, to design variable member force, S_{rd}, is greater than the structure factor, γ_i.

$$R_{rd}/S_{rd} \geq \gamma_i \qquad (8.2.3)$$

where design fatigue capacity of cross section, R_{rd}, is equal to fatigue capacity of cross section of member, $R_r(f_{rd})$, which is obtained from design fatigue strength of material, f_{rd}, divided by the member factor, γ_b. Design member force due to variable load, S_{rd}, is equal to the variable member force, $S_r(F_{rd})$, which is obtained from the design variable load, F_{rd}, multiplied by the structural analysis factor, γ_a.

(3) In order to compute the equivalent number of cycles of fatigue load to design variable stress or design member force due to variable load, the variable stress or member force due to variable load during the lifetime of structure may be converted to independent number of cycles of fatigue stress or member force by an appropriate method. Miner's hypothesis may be applied for this conversion.

8.3 Computation of stress due to variable load

(1) Tensile stress in reinforcement due to flexure may be computed in accordance with Section 7.2.

(2) Compressive stress in concrete due to flexure may be computed using the stress of the rectangular stress distribution whose resultant force acts at the same position as the resultant of triangular stress distribution obtained in Section 7.2.

(3) Stress in shear reinforcement may be computed by Eqs. (8.3.1) and (8.3.2), in general.

$$\sigma_{wrd}=\frac{(V_{pd}+V_{rd}-0.5\,V_{cd})s}{A_w z(\sin\alpha_s+\cos\alpha_s)}\frac{V_{rd}}{V_{pd}+V_{rd}+V_{cd}} \qquad (8.3.1)$$

$$\sigma_{wpd}=\frac{(V_{pd}+V_{rd}-0.5\,V_{cd})s}{A_w z(\sin\alpha_s+\cos\alpha_s)}\frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}} \qquad (8.3.2)$$

8.4 Fatigue strength of concrete members without shear reinforcement

(1) Design shear fatigue capacity of beam members without shear reinforcement, V_{rcd}, may be computed by Eq. (8.4.1), in general.

$$V_{rcd}=V_{cd}(1-V_{pd}/V_{cd})\left(1-\frac{\log N}{11}\right) \qquad (8.4.1)$$

where V_{cd} is given by Eq. (6.3.3).

(2) Design punching shear fatigue capacity, V_{rpd}, of reinforced concrete slabs as plane member may be computed by Eq. (8.4.2), in general.

$$V_{rpd}=V_{pcd}(1-V_{pd}/V_{pcd})\left(1-\frac{\log N}{14}\right) \qquad (8.4.2)$$

where V_{pcd} is derived from Eq. (6.3.8).