CONCRETE LIBRARY OF JSCE NO.4, DECEMBER 1984

RECOMMENDATIONS FOR LIMIT STATE DESIGN OF CONCRETE STRUCTURES

(Reprint from the CONCRETE LIBRARY No.52 published by JSCE, November 1983)

JSCE Concrete Committee on Limit States Design

Michiyuki KOHNO, Chairman

Hajime OKAMURA, Secretary

Committee Members

Keiichi AOKI
Yukio AOYAGI
Taisuke AKIMOTO
Shoji IKEDA
Hajime IKEDA
Tatsuo ISHIKAWA
Yuichi ISHIKAWA
Tadayoshi ISHIBASHI
Mitsuaki IZUMI
Takashi IDEMITSU
Masatoshi ITO
Shunji INOMATA
Yoshiaki IMAI
Tamon UEDA
Taketo UOMOTO
Takao ENDO
Toshiaki OHTA
Minoru OHTA
Koji OHTSUKA
Nobuaki OHTSUKI
Masayuki OHTSUBO
Sadatoshi OHNO
Takahisa OKAMOTO
Akira OKIFUJI
Yoshio OZAKA
Mitsumasa KAIEDA
Yoshio KAKUTA
Satoshi KASHIMA
Kiyoshi KATAWAKI
Masafumi KATOH
Souji KANAI
Michio KANAI
Naotaka KAWAGUCHI
Hiroyuki KAWATA
Masahiko KUNISHIMA
Nobuo KUBOTA
Minoru KURAUCHI
Kazunori KOH
Hisahiro KOGA
Takayuki KOJIMA
Tohru GOTO
Kazuo KOBAYASHI
Sigetoshi KOBAYASHI
Hitoshi KOMURA
Wataru KOYANAGI
Kazuto SAITO
Syoichi SAEKI
Noboru SAEKI
Nobuaki SAKAI
Kenji SAKATA
Michiaki SAKATE
Takashi SAKAMURA
Seiji SUGIURA
Motohiko SUZUKI
Motoyuki SUZUKI
Osamu SEIMIYA
Hiroshi SEKI
Isamu TAKUWA
Kohji TAKEWAKA
Tadaaki TANABE
Takashi CHOU
Yukikazu TSUJI
Seiichi TOTTORI
Tomohiko TOMITA
Toshihide TOYOFUKU
Akihiko NISHIMURA
Yoshinobu NISHIYAMA
Junichiro NIWA
Ryoji HAGIWARA
Takeshi HIGAI
Kouzoh HIGUCHI
Ikuo HIRASAWA
Manabu FUJII
Shinichi FUJITA
Minoru FUNAKOSHI
Koichi MAEKAWA
Shoichi MAEDA
Yasuo MAEHARA
Atsuhiko MACHIDA
Masashi MATSUI
Hiroshi MATSUSHIMA
Hiromichi MATSUSHITA
Yoshifumi MATSUDA
Susumu MATSUMOTO
Kyuichi MARUYAMA
Takashi MIURA
Kunihiko MIYAKAWA
Toyoaki MIYAGAWA
Syusuke MIYAZAKI
Hiroshi MUTSUYOSHI
Akiumi MOCHIZUKI
Syogo MORISHITA
Jun YAMAZAKI
Noriaki YAMADERA
Takeshi YAMATO
Yasuhiko YAMAMOTO
Yoshiyuki YAMAMOTO
Kiyoshi YAMURA
Yukio YOKOMIZO
Toshiharu YOSHIOKA
Yasuhiko YOSHIOKA
Asuo YONEKURA
Akira WATANABE
Yasumitsu WATANABE
Katsuya WADA

JSCE Concrete Committee on Limit States Design

1st working group (General requirements, Load)

Wataru KOYANAGI, Chairman	Michio KANAI, Secretary
Tadayoshi ISHIBASHI	Yoshio OZAKA
Syoichi SAEKI	Takashi CHOU
Minoru FUNAKOSHI	Manabu FUJII
Akira WATANABE	

2nd working group (Concrete)

Yukio AOYAGI, Chairman	Taketo UOMOTO, Secretary
Shoji IKEDA	Yoshiaki IMAI
Junichiro NIWA	Yasuhiko YAMAMOTO
Yasuhiko YOSHIOKA	Asuo YONEKURA

3rd working group (Steel)

Taisuke AKIMOTO, Chairman	Akiumi MOCHIZUKI, Secretary
Yukio AOYAGI	Masatoshi ITO
Shoji IKEDA	Tadayoshi ISHIBASHI
Minoru KURAUCHI	Hitoshi KOMURA
Nobuaki SAKAKI	Syusuke MIYAZAKI
Yoshiyuki YAMAMOTO	

4th working group (Structural analysis, Deflection)

Yoshio KAKUTA, Chairman	Noboru SAEKI, Secretary
Yukio AOYAGI	Keiichi AOKI
Satoshi KASHIMA	Wataru KOYANAGI
Tadaaki TANABE	Koichi MAEKAWA
Jun YAMAZAKI	

5th working group (Flexure and axial loads)

Manabu FUJII, Chairman	Kenji SAKATA, Secretary
Taisuke AKIMOTO	Shoji IKEDA
Hajime OKAMURA	Yoshio KAKUTA
Naotaka KAWAGUCHI	Seiji SUGIURA
Tadaaki TANABE	Ikuo HIRASAWA
Hiromichi MATSUSHITA	

6th working group (Shear, Torsion)

Mitsuaki IZUMI, Chairman	Kyuichi MARUYAMA, Secretary
Yuichi ISHIKAWA	Tamon UEDA
Sadatoshi OHNO	Takahisa OKAMOTO
Takayuki KOJIMA	Junichiro NIWA
Takeshi HIGAI	Minoru FUNAKOSHI
Masashi MATSUI	Hiroshi MATSUSHITA
Kunihiko MIYAKAWA	

7th working group (Crack)

Hiroshi SEKI, Chairman	Takeshi YAMATO, Secretary
Taisuke AKIMOTO	Tatsuo ISHIKAWA
Tadayoshi ISHIBASHI	Koji OHTSUKA
Nobuaki OHTSUKI	Masayuki OHTSUBO
Takahisa OKAMOTO	Kiyoshi KATAWAKI
Hisahiro KOGA	Koji TAKEWAKA
Shinichi FUJITA	Syougo MORISHITA
Kiyoshi YAMURA	

8th working group (Fatigue)

Tadayoshi ISHIBASHI, Chairman	Hiromichi MATSUSHITA, Secretary
Keiichi AOKI	Taisuke AKIMOTO

Takashi IDEMITSU | Tamon UEDA
Hajime OKAMURA | Koji OHTSUKA
Nobuaki OHTSUKI | Yoshio KAKUTA
Kenji SAKATA | Isamu TAKUWA
Ikuo HIRASAWA

9th working group (Development of reinforcement)

Takashi MIURA, Chairman | Koji OHTSUKA, Secretary
Hajime OKAMURA | Masayuki OHTSUBO
Nobuo KUBOTA | Shigetoshi KOBAYASHI
Hiromichi MATSUSHITA | Yukio YOKOMIZO

10th working group (Structural details)

Shigetoshi KOBAYASHI, Chairman | Yoshifumi MATSUDA, Secretary
Shoji IKEDA | Hajime OKAMURA
Nobuaki OHTSUKI | Isamu TAKUWA
Takashi MIURA | Yukio YOKOMIZO
Katsuya WADA

11th working group (Design of members)

Hajime OKAMURA, Chairman | Yukikazu TSUJI, Secretary
Taisuke AKIMOTO | Akira OKIFUJI
Masahiko KUNISHIMA | Tohru GOTOH
Motohiko SUZUKI | Tomohiko TOMITA
Shoichi MAEDA | Susumu MATSUMOTO
Yoshifumi MATSUDA | Jun YAMAZAKI
Yukio YOKOMIZO | Yasumitsu WATANABE

12th working group (Prestressed concrete)

Jun YAMAZAKI, Chairman | Takashi IDEMITSU, Secretary
Shoji IKEDA | Hajime IKEDA
Syunji INOMATA | Tadayoshi ISHIBASHI
Mitsuaki IZUMI | Nobuaki OHTSUKI
Yoshio KAKUTA | Sohji KANAI
Masahiko KUNISHIMA | Takayuki KOJIMA
Kazuo KOBAYASHI | Kazunori KOH
Kazuto SAITOH | Michiaki SAKATE
Takashi SAKAMURA | Seiji SUGIURA
Motohiko SUZUKI | Tomohiko TOMITA
Toshihide TOYOFUKU | Yoshinobu NISHIYAMA
Manabu FUJII | Yoshiyuki YAMAMOTO

13th working group (Steel and reinforced concrete)

Shoji IKEDA, Chairman | Isamu TAKUWA
Taisuke AKIMOTO | Mitsuaki IZUMI
Toshiaki OHTA | Minoru OHTA
Shigetoshi KOBAYASHI | Kohzoh HIGUCHI
Noriaki YAMADERA | Toshiharu YOSHIOKA

14th working group (Seismic design)

Atsuhiko MACHIDA, Chairman | Tadaaki TANABE, Secretary
Tadayoshi ISHIBASHI | Takao ENDOH
Yoshio OZAKA | Minoru OHTA
Masafumi KATOH | Hiroyuki KAWATA
Motoyuki SUZUKI | Osamu SEIMIYA
Seiichi TOTTORI | Akihiko NISHIMURA
Ryoji HAGIWARA | Yasuo MAEHARA
Kyuichi MARUYAMA | Hiroshi MUTSUYOSHI

15th working group (Definition, Notation)

Kazuo KOBAYASHI, Chairman | Mitsumasa KAIEDA, Secretary
Toyoaki MIYAGAWA

PREFACE

In recent years design methods of concrete structure have been improved significantly, and research results have also been accumulated. The Concrete Committee in Japan Society of Civil Engineers formed the Committee on Ultimate Strength Design in 1968, which started to investigate the recent studies on the design method of concrete structure and has reported the results.

(1)Concrete Library No.34, "References of Ultimate Strength Theories of Reinforced Concrete," August 1972

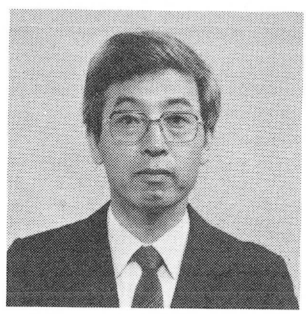
(2)Concrete Library No.41, "Recent Trend of Design Method of Reinforced Concrete," Nov. 1975

(3)Concrete Library No.48, "Proposal for Limit States Design of Concrete Structures," April 1981

In 1981 the Committee on Ultimate Strength Design was reformed to be nation wide and was renamed as Committee on Limit States Design. This recommendation has been proposed by this Committee.

Each section from the first to the eighteenth was discussed in the Committee members, but the nineteenth and twentieth sections were described on the responsibility of the corresponding working groups.

Michiyuki KOHNO, Chairman



Hajime OKAMURA, Secretary

Michiyuki KOHNO is a president of Japan Transportation Consultants, Inc. He was a research engineer of Japanese National Railways in Japan from 1941 to 1966 and was a superintendent of Structural Design Office of JNR from 1963 to 1966. He is a member of JSCE.

Hajime OKAMURA is a professor of Civil Engineering at the University of Tokyo. He was awarded JSCE Most Valuable Paper Prize in 1981 and JCI Valuable Paper Prize in 1981 and in 1982. He obtained his doctor at the University of Tokyo in 1966 and has been a faculty member since then. He has written many scientific articles on fatigue and shear of reinforced concrete members, and authored a book 'Limit State Design of Concrete Structures'.
Prof. Okamura is a current chairman of JCI Committee on Shear and FEM Analysis, and a member of working commission III of IABSE since 1982.

CONTENTS

6 STRUCTURAL ANALYSIS
- 6.1 Notation
- 6.2 General
- 6.3 Determination of member forces for check for ultimate limit states
- 6.4 Determination of member forces for check for serviceability limit states
- 6.5 Determination of member forces for check for fatigue limit states
- 6.6 Determination of member forces for check for seismic requirements
- 6.7 General requirements for linear analyses

7 FLEXURE AND AXIAL LOADS
- 7.1 Notation
- 7.2 General
- 7.3 Strength of the cross section of linear members
- 7.4 Computation of stresses in member cross sections
- 7.5 Computation of member forces in planar members subjected to flexure and in plane forces (in preparation)
- 7.6 Computation of deformation capacity of member cross sections (in preparation)

8 SHEAR
- 8.1 Notation
- 8.2 General
- 8.3 Check for linear members
- 8.4 Check for planar members subjected to transverse shear
- 8.5 Check for planar members subjected to in plane shear (in preparation)

9 TORSION
- 9.1 Notation
- 9.2 General
- 9.3 Check for linear members

10 CRACK
- 10.1 Notation
- 10.2 General
- 10.3 Definition for environmental conditions
- 10.4 Permissible crack width
- 10.5 Check for flexural crack width
- 10.6 Check for shear crack width

11 DEFLECTION
- 11.1 Notation
- 11.2 General
- 11.3 Permissible deflection
- 11.4 Computation of short term deflection
- 11.5 Computation of long term deflection

12 FATIGUE
- 12.1 Notation
- 12.2 General
- 12.3 Method of check
- 12.4 Computation of stresses due to variable loads

1 GENERAL

1.1 Scope

This Recommendation shows the fundamentals for general design of concrete structures based on the limit states design method.

1.2 Definition

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

(1) Design lifetime
The length of time specified in design during which the structures or members continue to fulfil their service requirements.

(2) Ultimate limit state
The limit states where structures or members fail, or lose stabilities or function due to overturning, buckling or excessive deformations.

(3) Service limit state
The limit states where structures or members become inadequate for normal service, or lose durability due to excessive cracking, displacements, deformations or vibration.

(4) Fatigue limit state
The limit states where structures or members fail in fatigue due to repetitive application of variable loads.

(5) Load
All actions that cause variations in deformations or stresses in structures or members.

(6) Permanent load
The loads which variation is almost nonexistant or negligible.

(7) Variable load
The loads which variation is not negligible and frequent or continuous.

(8) Accidental load
The loads which occurrences during the design lifetime are rare, but the consequences are grave if one should occur.

(9) Guaranteed value of load
The value of the load which the probability for sample values to exceed or the probability for sample values to fall below can be determined to be smaller than a given value.

(10) Guaranteed value of tested strength
The value of tested strengths which the probability for sample values to fall below can be determined to be smaller than a given value.

(11) Characteristic value for load
The values on the dangerous side of the guaranteed values of the loads.

(12) Characteristic value for materials strength
The guaranteed values of tested strengths.

(13) Specified value for load
The values for loads specified by design codes or specifications for the structures such as highway bridges or railroad bridges, apart from the characteristic values for loads.

(14) Specified value for materials strength
The values for materials strengths specified by design codes or specifications for the structures such as highway bridges or railroad bridges, apart from the characteristic values for loads.

(15) Material factor

A safety factor to take account of the variations in materials strengths from characteristic values toward undesirable directions, due to the causes such as differences in materials strengths in the structures from those determined by test specimens, influences of materials properties on the limit states, sensitivity against environments and time dependent change in materials properties.

(16) Load factor

A safety factor to take account of the variations in loads from characteristic values toward undesirable directions, due to the causes such as uncertainty in evaluation of loads, influences of nature of the loads on the limit states, or variations in environmental actions.

(17) Structure factor

A safety factor to take account of importance of a structure and its influence on society when the structure attained the ultimate limit states.

(18) Structural analysis factor

A safety factor to take account of uncertainties in structural analyses for computation of member forces.

(19) Member factor

A safety factor to take account of uncertainties in computation of strengths of members, influence of scatter of dimensions of members, or importance of members, that is, the influence on the entire structure when a member attained limit states.

(20) Load combination factor

A factor to take account of probability of combinations of the loads.

(21) Load modification factor

A factor to transform the specified values for loads into characteristic values.

(22) Material modification factor

A factor to transform the specified values for materials strengths into characteristic values.

(23) Design value for loads

A sum of all terms of combination of loads using load modification factors after multiplying characteristic values by respective load factors for check for a certain limit state.

(24) Design value for materials strength

A characteristic value for materials strengths divided by material factor for check for a certain limit state.

(25) Computed value for member force

Member forces computed using design values for loads for check for a certain limit state.

(26) Computed value for strength of member cross section

Strengths of member cross sections computed using design values for materials strengths for check for a certain limit state.

(27) Design value for member force

Computed values of member forces multiplied by structural analysis factor for check for a certain limit state.

(28) Design value for strengths of member cross section

Computed values of strengths of member cross sections divided by member factor for check for a certain limit state.

(29) Linear analysis

Structural analyses based on elastic primary theory assuming linear stress-strain relation of materials and neglecting secondary effect of deformations.

(30) Nonlinear analysis
Structural analyses taking account of nonlinear effects, such as, nonlinear stress-strain relation of materials, change in stiffness due to cracking or yielding of steel, or secondary effects due to deformations.

(31) Static linear analysis
Linear analyses where appropriate static effects are substituted for dynamic effects of earthquakes.

(32) Static nonlinear analysis
Nonlinear analyses where appropriate static effects are substituted for dynamic effects of earthquakes.

(33) Natural period
The lowest period of free oscillation of foundations or structures.

(34) Linear damage rule
A hypothesis that assumes linear relation of the number of application of constant amplitude of stress and degree of damage as in Miner's rule or modified Miner's rule.

(35) Redistribution of moment
Change in distribution of member forces in statically indeterminate structures due to cracking or formation of plastic hinges.

(36) Crack width
Crack width on the surface of concrete.

(37) Main reinforcement
Reinforcement which area is determined based on loads.

(38) Positive reinforcement
Main reinforcement arranged to resist tensile stress due to positive bending moment in slabs or beams.

(39) Negative reinforcement
Main reinforcement arranged to resist tensile stress due to negative bending moment in slabs or beams.

(40) Distribution reinforcement
Reinforcement which purpose is to distribute stresses and is usually placed transverse to positive or negative reinforcement.

(41) Web reinforcement
Reinforcement to resist diagonal tensile stress in slabs or beams.

(42) Stirrup
Web reinforcement placed at right angle or near right angle from positive or negative reinforcement and enclosing the latter.

(43) Bent bar
Web reinforcement formed by bending up positive reinforcement or bending down negative reinforcement.

(44) Tie
Transverse reinforcement enclosing longitudinal reinforcement at prescribed intervals.

(45) Spiral reinforcement
Reinforcement continuously wound in the form of cylindrical helix and enclosing longitudinal reinforcement.

(46) Precaution reinforcement
Auxiliary reinforcement for precaution against cracking due to temperature or drying shrinkage.

(47) Prestressing steel

High strength steel used for prestressing.

(48) Tendon

Single or bundled prestressing steel arranged for prestressing.

(49) Sheath

Sheath to form voids in concrete to accomodate tendons in prestressed concrete members of post tensioning system.

(50) Anchor

Device for anchoring ends of tendons in concrete.

(51) Coupler

Device to join tendon to tendon or anchor to anchor.

(52) Effective depth

Distance between extreme compressive fiber of member cross section and centroid of positive or negative reinforcement.

(53) Effective cross sectional area of concrete

Area of cross section of members bounded by extreme compressive fiber and centroid of cross section of positive or negative reinforcement.

(54) Tensile reinforcement ratio

Ratio of cross sectional area of main tensile reinforcement and effective cross sectional area of concrete.

(55) Compressive reinforcement ratio

Ratio of cross sectional area of main compressive reinforcement and effective cross sectional area of concrete.

(56) Balanced strain reinforcement ratio

Tensile reinforcement ratio of a cross section where attainment of design value of yield strength of main tensile reinforcement coincides with attainment of ultimate compressive strain for the extreme compressive fiber strain in concrete.

(57) Development length of reinforcement

Embedded length of reinforcement required to develop design value for steel stress at a design cross section.

(58) Space

Spaces between adjacent reinforcement, tendons or sheaths.

(59) Concrete cover

Minimum thickness of concrete cover between concrete surface and surface of reinforcement, tendons or sheaths.

(60) Clear span

Distances between opposing front surfaces of supports for slabs or beams.

(61) One-way slab

Rectangular slab supported by opposing two sides.

(62) Two-way slab

Rectangular slab supported by four sides.

(63) Deep beam

Beam which depth is relatively greater than span and distribution of stress across the depth is nonlinear.

(64) Corbel

Cantilevers attached to columns and span to depth ratio of which is less than or equal to one.

(65) Column

Member subjected to compressive forces, oriented vertical or near vertical and the length of which is greater than 3 times the least transverse dimension.

(66) Effective length of column
The length between adjacent points of zero moment in the mode shapes of buckling.

(67) Slenderness ratio
Ratio of effective length of column and radius of gyration.

(68) Wall
Vertical planar member which horizontal dimension is greater than 4 times the thickness.

1.3 Notation

The following notations are used in this Recommendation.

A : gross area of concrete section
A_a : bearing area
A_m : effective area of concrete to resist torsion, $b_0 \times d_0$ for rectangular section and $\pi d_0^2/4$ for circular section
A_s : area of reinforcement provided, or area of reinforcing steel in tensile zone
A_{sc} : area of reinforcement required by calculation
A_{ss} : area of steel shape in tensile zone
A_t : area of transverse reinforcement placed normal to surface of bond failure of bars to be anchored
A_{tb} : amount of torsion reinforcement in balanced condition at ultimate
A_{tbl} : amount of longitudinal torsion reinforcement in balanced condition at ultimate
A_{tbw} : amount of transverse torsion reinforcement in balanced condition at ultimate
A_{tl} : area of longitudinal torsion reinforcement
A_{tw} : area of a closed stirrup or a tie
A_w : area of one unit of shear reinforcement
a : constant
b_0 : length of short side of stirrups or ties
b_w : width of web of member
c : concrete cover of main tensile reinforcement or main sheath, or the smaller of concrete cover to reinforcing bar to be anchored and one half the space between bars to be anchored
c_ϕ : center-to-center distance of reinforcing steel
c_a : coefficient for correction of concrete cover for environmental condition
c_e : coefficient for correction of concrete cover for construction tolerances
c_m : minimum concrete cover
c_n : standard value for concrete cover
c_q : coefficient for correction of concrete cover for quality of concrete
d : effective depth
d_0 : diameter of concrete cross section enclosed by stirrups or ties for circular cross section, or the length of longer side of stirrups or ties 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 steel shape

F : load

F_k : characteristic value for 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 : modulus of rupture of concrete

f_{bm} : mean of modulus of rupture of concrete

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

f'_c : compressive strength of concrete

f'_{cm} : mean of compressive strength of concrete

f_k : characteristic value for material strength

f_l : yield strength of longitudinal torsion reinforcement

f_m : mean of material strength

f_n : specified value for material strength

f_{pu} : tensile strength of prestressing steel

f_{py} : yield strength of prestressing steel

f_r : fatigue amplitude strength

f_{rc} : fatigue amplitude strength of concrete

f_{rp} : fatigue amplitude strength of prestressing steel

f_{rs} : fatigue amplitude strength of steel

f_t : tensile strength of concrete

f_{tm} : mean of tensile strength of concrete

f_u : tensile strength of steel

f_v : apparent shear strength of concrete used for obtaining and

f_{v0} : shear strength of concrete

f_{vy} : yield strength in shear of steel

f_w : yield strength of transverse torsion reinforcement

f_{wy} : yield strength of shear reinforcement

f_y : tensile yield strength of steel

f'_y : compressive yield strength of steel

G_c : shear modulus of concrete

I_{cr} : moment of inertia of cross section excluding concrete in tensile zone

I_e : moment of inertia of transformed cross section

I_g : moment of inertia of gross cross section

K : constant

K_c : coefficient to take account of effects of concrete cover and transverse reinforcement in development of reinforcement stress

K_t : section modulus for torsion

k : coefficient determined by the shape of distribution function of test values and the probability of obtaining test values smaller than the characteristic value, or constant

k_1 : coefficient for reduction in strength due to sustaining load, or coefficient to take account of differences between permanent load and variable load in influencing crack width and corrosion of steel

k_2 : coefficient to determine the stress value in equivalent stress block

k_3 : coefficient to determine depth of equivalent stress block, or coefficient expressing influence of bond behavior of reinforcement

k_{hd_1} : seismic force coefficient to cause failure of member cross section

k_{hd_2} : seismic force coefficient of the magnitude 2 to 3 times the

k_p : coefficient for reduction due to fraction of permanent load and environmental condition

l_b : span length of flange between girders

l_c : span length of flange

l_d : basic development length of reinforcement

l_0 : development length of reinforcement

l_s : distance to shift a reference point for evaluation of development length in the direction reducing moment of longitudinal reinforcement in flexural member

M : bending moment

M_{cr} : moment at the limit of cracking in a section

$M_{d\max}$: maximum moment for calculation of deflections

M_0 : moment at the limit of inducing tensile stress in a section

M_t : torsional moment

M_{tc} : torsional moment strength without torsion reinforcement

M_{tu} : torsional moment strength

M_{ty} : torsional moment strength at the yield of torsion reinforcement

M_u : moment strength

m_t : ratio of longitudinal to transverse torsion reinforcement

N : fatigue life, or equivalent number of cycles of fatigue load

P_e : effective prestress in tendon

p : ratio of tension reinforcement

p' : ratio of compression reinforcement

p_w : ratio of longitudinal reinforcement to area of web concrete, $A_s/b_w d$

r : 1/4 of peripheral length of loaded surface, or inside radius of bend

R : strength of cross section

R_r : fatigue amplitude strength of cross section

S : member force

S_e : member force for check for crack width $(S_e=S_p+k_1 S_r)$

S_p : member force due to permanent load

S_r : member force due to variable load

s : space of shear reinforcement, torsion reinforcement, or transverse reinforcement

u : peripheral length of bar, or $2(b_0+d_0)$

u_p : effective peripheral length in resisting slab punching, or peripheral length of bearing area of concentrated load or reaction plus πd

V : shear force

V_c : shear strength of members without shear reinforcement

V_{cp} : punching shear strength

V_h : force component acting parallel to applied shear due to variation in member depth

V_o : shear force due to the load being considered

V_p : shear force due to permanent load

V_{pe} : shear force component acting parallel to applied shear due to effective prestress in deflected tendon

V_{pp} : shear force in slab due to permanent load

V_r : shear force due to variable load

V_{rc} : fatigue amplitude shear strength provided by effects other than shear reinforcement

V_{rp} : fatigue amplitude slab punching shear strength

V_{ry} : shear strength provided by reinforced concrete section of SRC (Steel and Reinforced Concrete)

V_s : shear strength provided by shear reinforcement

V_{sry} : Shear strength of SRC (Steel and Reinforced Concrete) members of additive strength type
V_{sy} : shear strength provided by steel shape
V_{wc} : calculated shear strength provided by concrete when web crushes
V_y : shear strength of member when shear reinforcement yields
w : crack width
w_{lim} : permissible crack width
x : distance from support
z : distance between point of application of compression resultant and centroid of tension reinforcement
α : angle between shear reinforcement and member axis
α_1 : in development of reinforcement, coefficient to take account of difference in compression and tension
α_2 : in development of reinforcement, coefficient to take account of influence of position of reinforcement in member
α_3 : in development of reinforcement, coefficient to take account of influence of concrete cover and transverse reinforcement
α_c : angle between compression fiber and member axis
α_p : angle between tendon and member axis
α_t : angle between tension reinforcement and member axis
β_{cr} : coefficient for reducing characteristic value of tensile strength according to degree of danger of diagonal tension cracking
β_d : coefficient to take account of influence of effective depth on shear strength
β_{ds} : coefficient to take account of influence of effective depth on shear strength of slabs or footings
β_n : coefficient to take account of influence of axial force on shear strength
β_{nt} : coefficient to take account of influence of axial force on torsional moment strength
β_p : coefficient to take account of influence of longitudinal reinforcement on shear strength
β_r : coefficient concerning slab punching shear strength
β_x : coefficient for modifying design shear force near direct support
γ : rate of apparent relaxation of prestressing steel, or k_{hds}/k_{hd1}
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
δ : coefficient of variation of test results, or response displacement
δ_y : displacement corresponding to seismic force factor equals to k_{hd1}
ε'_c : compressive strain in concrete
ε'_{cc} : compressive creep strain in concrete
ε'_{cu} : ultimate compressive strain in concrete
η : square root of ratio of total area of concrete to bearing area
λ_t : effective width of flange on one side to resist torsion
ρ_f : load modification factor
ρ_m : material modification factor
σ_r : applied amplitude stress
σ'_{cp} : compressive stress in concrete due to permanent load

σ_p : stress in concrete due to permanent load
σ_{sp} : stress in steel due to permanent load
σ_w : stress in shear reinforcement
σ_{wr} : stress in shear reinforcement due to variable load
σ_{wp} : stress in shear reinforcement due to permanent load
σ_{pe} : increase in stress in prestressing steel for check for crack width
σ_{pp} : increase in stress in prestressing steel due to member force caused by permanent load
σ_{se} : increase in stress in steel for check for crack width
σ_{sp} : increase in stress in steel due to member force caused by permanent load
φ : creep factor for concrete
ϕ : diameter of steel
ψ : load combination factor

Commentary to Chapter 1
Meanings of symbols of frequent use are as follows.

A : area
b : width
c : cover, space
d : effective depth
E : modulus of elasticity
F : load
f : material strength
G : shear modulus
I : moment of inertia
l : span length, development length
M : moment
N : fatigue life
P : prestress in tendon
p : reinforcement ratio
R : strength of cross section
S : member force
s : space
u : peripheral length
V : shear
w : crack width
x : distance from support
α : angle from member axis
β : coefficient for shear strength
γ : safety factor, rate of relaxation
δ : coefficient of variation, displacement
ε : strain
ρ : modification factor
σ : stress
φ : creep factor
ϕ : diameter
ψ : load combination factor

Meanings of suffixes of frequent use are as follows.

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

Characteristic value for load and material strength is indicated by suffix k.

Design value for member force and strength of cross section is indicated by suffix d .

When it is obvious that characteristic value or design value are meant, suffix k or d are dropped.

Tension is indicated by positive sign and compression by negative sign.

A prime (') attached to the right shoulder of symbol means the value is in compression without negative sign.

2 GENERAL REQUIREMENTS FOR DESIGN

2.1 Notation

2.2 Objectives of design

The objectives of design is to attain that the strucure is suited for its purposes, is safe and economical, that the structure and the members should develop functions sufficient for ordinary services, and posses sufficient durability during its lifetime, as well as adequate safety against all loads during service and construction stage. The concordance of the structure with the environment must also be considered.

2.3 Lifetime assumed in design

The lifetime of the structure assumed in design is to be determined based on service life required for the structure and the durability performance of the structure.

2.4 Supposition of design

These recommendations for design suppose that adequate control of construction is executed in the construction site.

2.5 Limit states

(1) In design check must be made for all limit states where the structure or members can no more function or satisfy the design purposes.

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

(3) The check for limit states, as a rule, should be made using characteristic values of materials strength, loads and safety factors given in Section 2.11.

2.6 Check for ultimate limit state

(1) Ultimate limit state for failure of member cross sections

(i) Check for ultimate limit state for failure of cross sections due to flexure, axial load, shear, torsion and their combinations should be made by ensuring that the ratio of the design value for the strengths of member cross sections R_d and the design value for the member forces is greater than the structure factor γ_i .

$$R_d/S_d \geqq \gamma_i$$

(ii) A design value for the strength of the cross section R_d should be determained by dividing a computed strength of the member cross sections by a member factor γ_b . The computed strength of the member cross sections is obtained based on design values of materials strengths . f_d is obtained by dividing a characteristic value for materials strength f_k by a materials factor γ_m .

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

$$f_d = f_k/\gamma_m$$

(iii) When specified values for the materials strengths f_n are given instead of the characteristic values, the characteristic values for the materials strengths f_k should be a specified strength f_n multiplied by a materials modification factor ρ_m .

$$f_k = \rho_m f_n$$

(iv) A design value for the member forces S_d should be a member force S multiplied by a structural analysis factor γ_a . The computed member force S should be based on the design values for the loads F_d which are the products of characteristic values of the loads F_k , load factors γ_f and load combination factors ψ .

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

$$F_d = \Sigma \gamma_f \psi F_k$$

(v) When specified values for the load F_n are given instead of the characteristic values, the characteristic values for the loads should be the products of those specified values F_n and load modification factors ρ_f .

$$F_k = \rho_f F_n$$

(2) Other ultimate limit states

When it is necessary check for ultimate limit states for rigid body motion stability, collapse mechanism, buckling, deformation, displacements and others should be made by appropriate methods.

2.7 Check for serviceability limit states

Check for serviceability limit states, in general, should be made by ensuring that stresses, crack widths or displacements for the members do not exceed the respective permissible values at the design values of the loads for which check is to be made.

2.8 Check for fatigue limit states

(1) Check for fatigue limit states should be made, as a rule, by ensuring that the ratio of a design value of fatigue amplitude strengths R_{rd} and a design value of variable member forces S_{rd} is greater than a structures factor γ_i .

$$R_{rd}/S_{rd} \geqq \gamma_i$$

(2) A design value for the fatigue amplitude strengths R_{rd} should be a computed fatigue amplitude strength of a member cross section $R_r(f_d)$ divided by a

members factor γ_b . The quantity R_{rd} should be obtained based on design values for materials strengths f_{rd} which are characteristic values for materials strengths f_{rk} divided by materials factors γ_m .

$$R_{rd}=R_r(f_{rd})/\gamma_b$$

$$f_{rd}=f_{rk}/\gamma_m$$

(3) A design value for variable member forces (member force amplitudes) should be a product of a computed variable member force $S_r(F_{rd})$ and a structural analyses factor γ_a. The computed variable member forces should be based on the design values of the loads F_{rd} which are the products of the characteristic values for the variable loads F_{rk} , the load factors , and load combinations factors.

$$S_{rd}=\gamma_a S_r(F_{rd})$$

$$F_{rd}=\Sigma\gamma_f \varphi F_{rk}$$

(4) Check for fatigue limit states may be made by ensuring that the ratios of the design values for fatigue amplitude strengths f_{rd} and the design values for variable stresses σ_{rd} are greater than $\gamma_a \cdot \gamma_b \cdot \gamma_i$

$$f_{rd}/\sigma_{rd} \geqq \gamma_a \cdot \gamma_b \cdot \gamma_i$$

2.9 Characteristic values

(1) A characteristic value for materials strengths f_k is a guaranteed value for the tested strengths. The guaranteed value is a value that most of the tested values exceed, and is defined by the following equation taking account of scatter of test data .

$$f_k=f_m(1-k\delta) \qquad (2.9.1)$$

where, k may, in general, be assumed to be 1.64.

When specified values for materials strengths f_n are given instead of the characteristic values, the characteristic values should be the products of those specified values and factors. Those materials modification factors ρ_m should be determined taking account of the relations between the characteristic values and the specified values.

(2) The characteristic values for the loads F_k should be the guaranteed values on dangerous side due to the maximum and minimum loads on the structure. The guaranteed values should be the expected values for the maximum and minimum loads occurring during construction stage and service of the structure and taking account of the scatter of the loads.

2.10 Computed values

(1) The function $R(f)$ to evaluate the computed values for the strengths of member cross sections should be, as a standard, such that leads to the average values of the strengths of the cross sections when all the materials factors are set equal to 1.0. Whereas, variations in $R(f)$ should be taken into account by γ_b .

(2) The function $S(F)$ to evaluate the computed values for member forces should be, as a standard, such that leads to the average values of member forces when all the load factors are set equal to 1.0. Whereas, variations in $S(F)$ should be taken into account by γ_a .

2.11 Safety factors and modification factors

(1) Safety factors are materials factors γ_m , load factors γ_f , structures factors γ_i , structural analyses factors γ_a , and members factors γ_b .

(2) Modification factors are load combinations factors ψ , materials modifications factors ρ_m and load modification factors ρ_f to take account of difference between characteristic values and specified values.

(3) For safety factors and modification factors the values chosen should be appropriate for the limit state being considered.

(4) Materials factors γ_m should be determined taking account of the variations of materials strengths from characteristic values toward undesirable directions, differences in materials properties of the structures from those of test specimens, influences of materials properties on the limit states, and time dependent variations of materials properties.

For check for the ultimate and the fatigue limit states the materials factor for concrete may be set equal to 1.3 and that for steel may be set equal to 1.0, in general.

For check for serviceability limit state materials factors may be set equal to 1.0, in general.

(5) Materials modification factors ρ_m should be determined taking account of differences between characteristic values and guaranteed values of materials strengths.

(6) Load factors γ_f should be determined taking account of variations of loads from characteristic values toward undisirable directions, uncertainty in evaluation of loads, and influences of nature of the loads on the limit states.

The load factors γ_f may be set equal to

1.1-1.2 for check for ultimate limit state,

1.0 for serviceability limit state and

1.0 for fatigue limit state.

(7) Load modification factors ρ_f should be determined taking account of differences between characteristic values and specified values of the loads.

(8) Load combinations factors ψ should be determined taking account of probability of combinations of the loads.

(9) Structures factors γ_i should be determined taking account of importance of a structure and its influence on society when the structure attained ultimate limit states. The structures factors γ_i may be set equal to 1.0-1.15, in general.

(10) Structural analyses factors γ_a should be determined taking account of uncertainties in structural analyses for computation of member forces. The structural analyses factors may be set equal to 1.0, in general.

(11) Members factors γ_b should be determined taking account of uncertainties in computation of strengths of members, influence of scatter of dimensions of members, and importance of members, namely, the influence on the entire structure when a member attained a limit state. The values for γ_b should be determined corresponding to respective equations for computing strengths of members.

3 CONCRETE

3.1 Notation

3.2 Quality

Concrete shall be a material whose strengths, deformational characteristics, durability, property to protect steel and collaboration with steel in a body are proper for structures or members. The scatter of quality of concrete shall be limited as small as possible.

3.3 Strength

(1) The characteristic values for strengths of concrete are guaranteed values for tested 28-day strengths in general.
Compressive test shall be made in accordance with JIS A 1108 "Testing method for compressive strength of concrete".
Tensile test shall be made in accordance with JIS A 1113 "Testing method for tensile strength of concrete".
Bending test shall be made in accordance with JIS A 1106 "Testing method for bending strength of concrete".
(2) When the ready-mixed concrete meeting the prescriptions in JIS A 5308 is used, the nominal strength specified by a purchaser may be the characteristic value of compressive strength f'_{ck} in general.
(3) The characteristic value for the tensile, bending, bond and bearing strengths of concrete, as a rule, may be respectively obtained based on the characteristic value of the compressive strength as follows.

(i) The characteristic value for tensile strength, f_{tk}

$$f_{tk}=0.5 f'^{2/3}_{ck} \qquad (3.3.1)$$

(ii) The characteristic value for bending strength, f_{bk}

$$f_{bk}=0.9 f'^{2/3}_{ck} \qquad (3.3.2)$$

(iii) The characteristic value for bond strength, f_{bok}

The characteristic value for bond strength of deformed bars conforming to the prescriptions of JIS G 3112 (1975) shall be the value shown in Table 3.3.1.

For normal round bars, the characteristic value for bond strength shall be taken as 40% of that for deformed bars, in which, however, hooks shall be provided at the ends of round bars.

Table 3.3.1 **The characteristic value for bond strength** (kg/cm²)

f_{ck}	180	240	300	400~
f_{bok}	19	23	27	33

(4) The characteristic value for bearing strength, f_{ak}

$$f_{ak}=\eta\cdot f'_{ck} \qquad (3.3.3)$$

where

$$\eta=\sqrt{A/A'}\leqq 2$$

3.4 Fatigue strength

(1) The characteristic value for the fatigue amplitude strength shall be the guaranteed value for the tested fatigue amplitude strength with the consideration of kinds of concrete and the exposure condition of structures.

(2) The characteristic values for the compressive, flexural-compressive, tensile and flexural-tensile fatigue amplitude strengths of concrete, f_{rk} , may be determined as a function of fatigue life, N and stress in concrete due to permanent load, σ_p in accordance with Eq.(3.4.1).

$$f_{rk}=k_1 f_k(1-\sigma_p/f_k)(1-\log N/K) \qquad (3.4.1)$$

(i)The value of K for normal weight concrete which is continuously or often satulated by water and for light weight concrete shall be be 10.
In other cases, K shall be 17.

(ii) k_1 may be, in general, defined as follows.
For compressive and flexural-compressive fatigue amplitude strengths,
k_1 =0.85
For tensile and flexural-tensile fatigue amplitude strengths,
k_1 =1.0

(iii) The stress in concrete due to permanent load σ_p may be, in general, set equal to 0 when the repeated load is considered.

3.5 Stress-strain diagram

(1) The stress-strain diagram shall be assumed to have a proper form for the purpose considered.

(2) For check for ultimate limit state of flexure and axial loads, the uniaxial stress-strain diagram shown in Fig.3.5.1 may be used in general.

(3) For check for serviceabiliy limit state, the stess-strain diagram of concrete may be assumed a straight one in which modulus of elasticity should be in accordance with Section 3.6.

(4) The stress-strain diagram of concrete under biaxial and triaxial stress conditions considerably differs from that shown in Fig.3.5.1, and accordingly account shall be taken of the effect of the difference. For check for the serviceability limit state, however, concrete may be elastic and its modulus of elasticity and Poisson's ratio may be the values specified in 3.6 and 3.7.

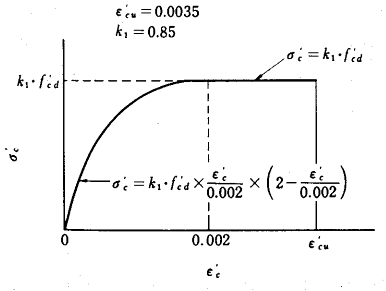


Fig. 3.5.1 The stress-strain diagram for concrete under uniaxial stress state

3.6 Modulus of elasticity

(1) It is a principle that the modulus of elasticity of concrete should be the

mean value of tested secant modulus between the origin and the one-third point of the compressive strength on the stress-strain diagram prescribed in JIS A 1108 "Testing method for compressive strength".
(2) The modulus of elasticity used in computation of static indeterminate forces or elastic deformations may be the value stipulated in Table 3.6.1.

Table 3.6.1 Modulus of elasticity for concrete (the value for calculating redundant forces or elastic deformations)

f_{ck} (kg/cm²)	180	240	300	400	500
E_c(10^5 kg/cm²)	2.4	2.7	3.0	3.3	3.5

(3) The ratio of modulus of elasticity of steel to that of concrete ($n=E_s/E_c$) used in stress evaluations may be set equal to 7 in general.

3.7 Poisson's ratio

Poisson's ratio of concrete may be 0.2 within the elastic limit in general. However, when cracking of concrete due to tensile forces is permitted, Poisson's ratio shall be 0.

3.8 Thermal properties

It is a principle that the thermal properties of concrete are determined based on experiments or previously reported data. For normal concrete with ordinary properties, the values shown in Table 3.8.1 may be used in general.

Table 3.8.1 Thermal properties of concrete

Coefficient of thermal expansion	10×10^{-6}/°C
Heat conductivity	2.0 kcal/(mh°C)
Specific heat	0.25 cal/g/°C
Heat diffusivity	0.003 m²/h

3.9 Drying shrinkage

(1) The drying shrinkage of concrete, as a rule, shall be determined taking account of the influences of humidity arround structures, geometry and size of member section and proportions of concrete.
(2) The value stipulated in Table 3.9.1 may be taken as the strain of drying shrinkage of normal concrete.

Table 3.9.1 Strain of drying shrinkage ($\times 10^{-5}$)

Age of concrete / Environmental condition	less than 3 days	4-7 days	28 days	3 months	1 year
Outdoors	25	20	18	16	12
Indoors	40	35	27	21	12

(3) The drying shrinkage strain of concrete for computing static indeterminate forces by the theory of elasticity may be 15×10^{-5} in general. When this value is used, however, the contribution of creep shall not be added.

3.10 Creep

(1) Creep strain of concrete, which is assumed to be in proportion to the elastic strain due to applied stress, may be obtained in accordance with Eq.(3.10.1).

$$\varepsilon'_{cc}=\varphi\cdot\sigma'_{cp}/E_c \qquad (3.10.1)$$

(2) It is a principle that creep coefficient of concrete shall be determined taking account of the effects of the humidity arround structures, geometry and size of member section, proportions and age when stresses act on concrete.

(3) Creep coefficient applied for prestressed concrete may be the value shown in Table 3.10.1. When the creep strain is analyzed in accordance with Eq.(3.10.1) with the value in Table 3.10.1, 28-day modulus of elasticity, shall be specified.

Table 3.10.1 Creep coefficient of concrete

Kinds of cement	Environmental condition	Age of concrete when the prestressing is introduced				
		4-7 day	14 days	28 days	3 months	one year
High-early portland	Outdoor	2.6	2.3	2.0	1.7	1.2
	Indoor	4.0	3.3	2.8	2.1	1.3
Normal portland	Outdoor	2.8	2.5	2.2	1.9	1.4
	Indoor	4.3	3.6	3.1	2.4	1.6

4 STEEL

4.1 Notation

4.2 Quality

(1) Steel shall be a material whose strengths, stress-strain characteristics, workability, weldability, forms, and sizes are proper for concrete structures or members.

(2) It is a principle that steel shall be adapted by JIS standards.

4.3 Strength in tension, compression, and shear

(1) The characteristic values for the tensile yield strength of steel, f_y and that for the tensile strength, f_u are guaranteed values for those strengths. Tensile test shall be made by JIS Z 2241 "Tensile testing method for metalic materials".

(2) The characteristic value for the tensile yield strength of steel, f_y and that for the tensile strength, f_u may be the lower limit values of JIS specific values. The area of steel for design, in general, may be the nominal value.

(3) The characteristic value for the compressive yield strength of steel, f'_v may be equal to that for the tensile yield strength, f_v .

(4) The characteristic value for the shear yield strength of steel, f_{vy} may be obtained by the following equation.

$$f_{vy}=f_y/\sqrt{3} \qquad (4.3.1)$$

4.4 Fatigue strength

(1) The characteristic value for the fatigue amplitude strength of steel shall be the guaranteed value for the fatigue amplitude strength given by the tests with the consideration of kinds of steel, surface geometry, size of cross-section, methods for jointing, magnitudes and frequencies of working stresses, and environmental conditions.

(2) The characteristic value for the fatigue amplitude strength of steel, f_{rsk} may be obtained, as a function of fatigue life, N and stress in steel due to permanent load, σ_{sp} , by Eq.(4.4.1).

$$f_{rsk}=(1-\sigma_{sp}/f_u)10^a/N^k(\mathrm{kg/cm^2}) \qquad (4.4.1)$$

where, $f_{rsk}\leqq f_y-\sigma_{sp}$

(i) It is a principle that the values, a and k should be determined by tests.

(ii) The values, a and k for steel bar whose size is equal to or smaller than D32 may be those shown in Table 4.4.1.

Table 4.4.1 Values of a and k for steel bar with size equal to or smaller than D 32

	Straight bar		Pressed joint		Mechanical joint	
	a	k	a	k	a	k
$N\leqq 2\times10^6$	4.3	0.18	4.2	0.18	4.5	0.25
$N>2\times10^6$	4.0	0.13	3.9	0.13	—	—

(iii) In the cases of reinforcement composed of welded and bent-up bars, the characteristic values for the fatigue amplitude strength are to be 50% of that for the straight bar itself.

(iv) In the cases of prestressing steel, the characteristic value for the fatigue amplitude strength may be obtained, supposing the values, a and k equal to those in Table 4.4.2.

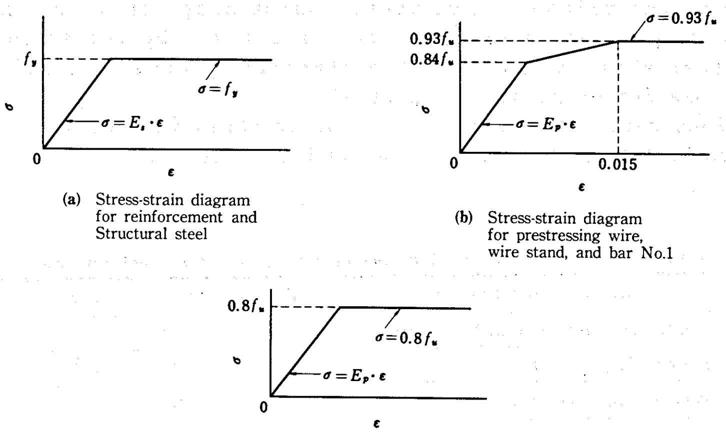
Table 4.4.2 Values of α and k for prestressing steel

	Prestressing steel bar		Prestressing steel wire · wire strand	
	α	k	α	k
$N \leqq 2 \times 10^6$	4.7	0.22	5.5	0.38
$N > 2 \times 10^6$	4.3	0.16	4.1	0.16

4.5 Stress-strain diagram

(1) The stress-strain diagram shall be assumed to have a proper form for the purpose considered.

(2) For the check of the ultimate limit state, the stress-strain diagram shown in Fig.4.5.1 may be used in general.



(a) Stress-strain diagram for reinforcement and Structural steel

(b) Stress-strain diagram for prestressing wire, wire stand, and bar No.1

(c) Stress-strain diagram for prestressing bar No.2

Fig. 4.5.1 Stress-strain diagram for steel

(3) For the check of the serviceability limit state, steel may be elastic and its modulus of elasticity and Poisson's ratio may be the values specified in Sections 4.6 and 4.7.

4.6 Modulus of elasticity

(1) It is a principle that the characteristic value for the modulus of elasticity should be calculated from the stress-strain diagram given by JIS Z 2241 "Tensile testing method for metalic materials".

(2) The characteristic value for the modulus of elasticity, in general, may be the values shown in Table 4.6.1.

Table 4.6.1 **Young's Modulus of steel** ($\times 10^6 kg/cm^2$)

kind of steel		
reinforcement and structural steel	E_s	2.1
prestressing bar	E_p	2.0

4.7 Poisson's ratio

Poisson's ratio for steel may be 0.3.

4.8 Modulus of heat expansion

The modulus of heat expansion for steel may be $10\times10^{-6}/°C$ in general.

4.9 Relaxation of prestressing steel

(1) The rate of the relaxation of prestressing steel shall be the guaranteed value of the tripled value of 1,000 hour test given by relaxation tests. The relaxation test shall be made by JSCE standard (proposed) testing method for long-time relaxation of prestressing steel.

(2) The fictious rate of the relaxation of prestressing steel, in general, may be assumed to be those shown in Table 4.9.1 in order to calculate the loss of prestress.

Table 4.9.1 Rates of relaxation corresponding to initial tensile stress, γ

Kinds of prestressings stêel	Fictious rates of relaxation, γ
Prestressing steel wire and wire strand	5%
Prestressing steel bar	3%
Prestressing steel with low relaxation	1.5%

5 LOAD

5.1 Notation

5.2 General

In design of structures, all loads during construction stage and service stage should be considered in adequate combination of loads.

5.3 Characteristic values

(1) Characteristic values for loads F_k should be guaranteed values on dangerous side due to the maximum and minimum loads on structures. Guaranteed values are expected values for the maximum and minimum loads occurring during construction stage and service stage of structures and taking account of scatter

of loads.

(2) Characteristic values should be determined respectively for the limit states which must be checked. In general, ultimate limit state, serviceability limit state and fatigue limit state may be considered as the limit states which should be checked.

(3) When specified values of loads F_n are given instead of characteristic values, characteristic values for loads should be products of those specified values F_n and load modification factors ρ_f .

5.4 Design values

(1) Design values for load F_d should be determined taking account of load factors γ_f and load combination factor ψ . Load factors γ_f represent variations of loads from characteristic values toward undesirable directions, uncertainty in evaluation of loads and influences of nature of loads on the limit states. Load combination factors ψ represent probability of combinations of loads which should be checked.

(2) Load factors γ_f which are being multiplied by characteristic values, in general, may be set equal from 1.1 to 1.2 for check for ultimate limit state and to 1.0 for serviceability limit state and for fatigue limit state.

(3) Combination of loads should be put together one governing load and other loads which act simultaneously. In order to combine other loads, they should be multiplied by load combination factor ψ taking account of probability of occurring simultaneously.

6 STRUCTURAL ANALYSIS

6.1 Notation

6.2 General

(1) For structural analyses appropriate analysis models should be selected corresponding to geometry of the structures, support conditions, state of the loads and the limit states to be checked for.

(2) Structures may be analysed by assuming structural models such as slabs, beams, columns, frames, arches, shells or their combinations corresponding to their geometry, in general. However, consideration must be made so that the behavior of the structure agrees with the assumptions for boundary conditions, connectivity conditions, and other conditions for analysis. Especially when local stresses do not appear due to a modeling, reinforcement must be provided for those stresses when it is necessary.

(3) For structural analyses loads also may be modeled to give equivalent or conservative effects by simplifying the manner of distribution of loads or by replacing dynamic loads by static loads.

(4) Methods for determining spans of beams, effective widths of T beam flanges, spans of slabs, widths of distribution of concentrated loads and member axes for frames and arches should follow the requirements in Chapter 18.

(5) The member forces and other quantities such as bending moments, shear, axial force and torsional moment should be computed by appropriate analytical theories

correspondig to the limit states.

6.3 Determination of member forces for check for ultimate limit states

(1) Linear analyses may be used for determination of member forces for check for ultimate limit states.

(2) When analysis methods other than linear analysis are to be used, it should be confirmed that such methods give safe results.

(3) When check is to be made for moments at supports or nodal joints in continuous beams, continuous slabs and frames under accidental loads, redistribution may be made within 15% of the values obtained by linear analysis. In such cases the moments at all cross sections must be greater than 70% of the values before redistribution is made. Also, the reinforcement ratio p-p' for all cross sections must be smaller than 50% of the balanced reinforcement ratio.

(4) The member forces associated with forced deformations due to temperature changes, creep and drying shrinkage in ordinary conditions may be neglected. For this simplification the reinforcement ratio p-p' at all cross sections must be smaller than 50% of the balanced reinforcement ratio.

6.4 Determination of member forces for check for serviceability limit states

Determination of member forces used for check for serviceability limit states should be based on linear analyses, as a rule.

6.5 Determination of member forces for check for fatigue limit states

Determination of member forces used for check for fatigue limit state should be based on linear analyses, as a rule.

6.6 Determination of member forces for check for seismic requirements

Determination of member forces used for check for seismic requirements should follow the requirements of Chapter 15.

6.7 General requirements for linear analyses

In linear anlyses the moment of inertia for member cross sections and materials constants should be determined as follows.

(1) Moments of inertia for member cross sections

The moment of inertia for member cross sections may be computed based on the gross cross sections neglecting effects of steel, in general.

(2) Materials constants

The modulus of elasticity and Poisson's ratios for concrete and steel should be respective characteristic values.

7 FLEXURE AND AXIAL LOADS

7.1 Notation

7.2 General

(1) Structures must be designed for flexure moment and axial loads so that their purposes of service are not impaired.
In general, check of safety for ultimate limit state should be made according to Section 7.3 and check for serviceability limit state should be made according to Chapter 10 and Chapter 11. If necessary, check for fatigue limit state should be made according to Chapter 12.
(2) Check for flexure moment and axial loads resulting from earthquake should be made according to Chapter 15.
(3) In check for serviceability limit state or fatigue limit state, when evaluation of stress intensity of section of members are necessary, the provisions of Section 7.4 should be followed.
(4) Check for planar members subject to transverse bending or in plane forces should be made according to Section 7.5.
(5) In check for ultimate limit state, when evaluation of deformation ability of section of members are necessary, the provisions of Section 7.6 should be followed.

7.3 Strength of cross section of linear members

(1) Evaluation of strength of cross section of linear members subject to bending moment, axial loads and combination of bending moment and axial loads should be made according to following assumptions.

(i) Fiber strain is proportional to distance from neutral axis.
(ii) Tensile stress of concrete is neglected.
(iii) Stress - strain relationship of concrete is according to Section 3.5. In the relationship, ε'_{cu} and k_1 is set equal to 0.0035 and 0.85, respectively.
(iv) Stress - strain relationship of steel is according to Section 4.5.

(2) Stress - strain relationship of confined concrete in transverse direction is determined taking account of experimental data.
When concrete strains of section of members are not all compression, compressive stress distribution of concrete may be considered to be equivalent to rectangular compressive stress distribution (equivalent stress block) as shown in Fig.7.3.1. When breadth of cross section is constant or becomes large as it approaches to compression extreme fiber, $k_1 \cdot k_2$ may be set equal to 0.85 and k_3 may be set equal to 0.80. When breadth of cross section becomes small as it approaches to compression extreme fiber, $k_1 \cdot k_2$ may be set equal to 0.80 and k_3 may be set equal to 0.80.

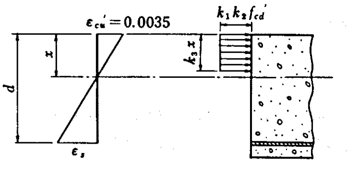


Fig. 7.3.1 Concrete strain and rectangular compressive stress distribution (equivalent stress block)

(3) In general, member factor γ_b may be set equal to 1.15.

7.4 Evaluation of stress intensity of section of members

In check for serviceability limit state or fatigue limit state, evaluation of stress intensity of concrete and steel in section of members should be made according to following assumptions.

(i) Fiber strain is proportional to distance from neutral axis.
(ii) Both concrete and steel are dealt with as elastic body.
(iii) Tensile stress of concrete is neglected.
(iv) The ratio of Young's modulus of steel to concrete may be set equal to 7 for both reinforced concrete and prestressed concrete.

7.5 Evaluation of strength of section of planar members subject to transverse bending and in plane forces (in preparation)

7.6 Evaluation of deformation ability of section of members (in preparation)

8 SHEAR

8.1 Notation

8.2 General

(1) Structures must be designed for shear so that their purposes of service are not impaired.
(2) Check for ultimate limit state should be made according to Section 8.3, in general.
(3) For check of safety at ultimate limit state it must be ensured that safety against shear is greater than safety against flexure and axial loads.
(4) When a ratio of permanent load to the total load is great, check must be made for serviceability limit state according to Chapter 10.
(5) When a ratio of variable load to the total load is great, and when the number of repetition of the variable load is great, check must be made for fatigue limit state according to Chapter 12.
(6) In severe earthquakes when reversible loads accompanying large deformations are applied the provisions of Chapter 15 shall be followed.
(7) The structural analyses factor γ_a may be set equal to 1.0 in general.

8.3 Check for linear member

8.3.1 General

(1) For linear members design of the cross sections must be made so that yield in shear reinforcement precedes crush of concrete in webs due to diagonal compression.
(2) For check of linear members for ultimate limit state it should be ensured

that the ratios of design values for shear strength to the design values for shear are greater than the structures factor γ_i .

(3) When the provision in (2) is satisfied by using the design value for shear strength which is the contribution of the factors other than shear reinforcement, the shear reinforcement may be proportioned according to Section 14.4.

8.3.2 Design value for shear

The design values for shear in the members where the depth of the member is variable and containing prestressing tendons which have angles with member axis may be obtained by Eq.8.3.1.

$$V_d = V_{od} - V_{hd} - V_{ped} \tag{8.3.1}$$

where,

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

The effective depth d for the cases where prestressed steel and ordinary steel coexist should be determined for the composite cross section of both steel.

$$V_{ped} = P_{ed}\cdot(\sin\alpha_p - \tan\alpha_t) \quad \cdots\cdots\cdots\cdots \quad \alpha_t \geqq 0$$
$$= P_{ed}\cdot\sin\alpha_p \quad \cdots\cdots\cdots\cdots \quad \alpha_t < 0$$

8.3.3 Design value for shear strength

(1) Design values for shear strength must be obtained taking account of concrete strength, shear span, cross sectional dimensions of the member, longitudinal reinforcement ratio, and type, quantity and arrangement of shear reinforcement. In general, items (2) through (4) of Section 8.3.3 may be used.

(2) Design values for shear strength V_{cd} contributed by factors other than shear reinforcement should be obtained by Eq.8.3.2.

$$V_{cd} = f_{vd}\cdot b_w\cdot d/\gamma_b \tag{8.3.2}$$

where,

$$f_{vd} = f_{vod}(1+\beta_d+\beta_p+\beta_n)$$
$$f_{vod} = f_{vk}/\gamma_c (\mathrm{kg/cm^2})$$
$$f_{vk} = 0.94\, f'^{1/3}_{ck} (\mathrm{kg/cm^2})$$
$$\beta_d = \sqrt[4]{100/d} - 1 \geqq 0 \quad (d : \mathrm{cm})$$
$$\beta_p = \sqrt{100\, p_w} - 1 \leqq 0.73$$
$$\beta_n = M_0/M_d \leqq 1$$

(i) In general γ_b may be set equal to 1.15, and γ_c equal to 1.3.

(ii) For the members subjected to axial tension and when the axial tensile stress is so small that cracking due to that stress does not occur, it should be that $\beta_n = -1$. When cracking occurs due to axial tension, it should be that $V_{cd} = 0$.

(3) Design values for shear strength V_{yd} corresponding to yield of shear reinforcement should be obtained by Eq.8.3.3.

$$V_{yd} = V_{cd} + A_w\, f_{wyd}\, z(\sin\alpha + \cos\alpha)/s/\gamma_b \tag{8.3.3}$$

(i) γ_b may be set equal to 1.15 in general.

(ii) z may be set equal to $d/1.15$.

(iii) V_{cd} should be obtained by Eq.(8.3.2).

(iv) When bent up bars are used as a part of shear reinforcement, more than 50% of the shear carried by shear reinforcement should be carried by stirrups.

(4) Design values for shear strength V_{wcd} governed by compression failure in web concrete due to diagonal compression should be obtained by Eq.(8.3.4).

$$V_{wcd}=0.3f'_{cd}\,b_w\,d/\gamma_b \qquad (8.3.4)$$

γ_b may be set equal to 1.5 in general.

8.3.4 Computation of stress in shear reinforcement

(1) Stresses in shear reinforcement to be used in check for serviceability limit state may be obtained by Eq.(8.3.5).

$$\sigma_{wd}=\frac{(V_d-V_{cd})s}{A_w\,z(\sin\alpha+\cos\alpha)} \qquad (8.3.5)$$

(2) Stresses in shear reinforcement to be used in check for fatigue limit state may be obtained by Eqs.(8.3.6) and (8.3.7).

$$\sigma_{wrd}=\frac{(V_{pd}+V_{rd}-0.5V_{cd})s}{A_w\,z(\sin\alpha+\cos\alpha)}\,\frac{V_{rd}}{V_{pd}+V_{rd}+V_{cd}} \qquad (8.3.6)$$

$$\sigma_{wpd}=\frac{(V_{pd}+V_{rd}-0.5V_{cd})s}{A_w\,z(\sin\alpha+\cos\alpha)}\,\frac{V_{pd}+V_{cd}}{V_{pd}+V_{rd}+V_{cd}} \qquad (8.3.7)$$

8.3.5 Check for ultimate limit state near supports

(1) For check of safety at ultimate limit state of members near their supports the reduced design values of shear may be used according to Eq.(8.3.8).

$$V_d=(V_{od}-V_{hd}-V_{ped})/\beta_x \qquad (8.3.8)$$

where, V_d :Reduced design values of shear near supports

β_x :Modification factor depending on the distance from the support

$= 5/[1+2(x/d)] \quad (0<x/d\leqq 2.0)$

x :Distance from the support

d :Effective depth

V_{hd} and V_{ped} are given by Eq.(8.3.1).

(2) When the design values for shear given by Eq.(8.3.8) do not exceed V_{cd} given by Eq.(8.3.2), design of cross sections should be the same as that for the cross section located at two times the effective depth from the support.

(3) When the design values of shear given by Eq.(8.3.8) exceed V_{cd}, design of cross sections should be the same as that for the cross section located where the difference of the design shear and V_{cd} is greatest. Shear reinforcement must have the amount which can carry the shear V_{sd} computed by Eq.(8.3.9), and be provided throughout the length of two times the effective depth from the support.

$$V_{sd}=(V_{od}-V_{hd}-V_{ped})-V_{cd}\,\beta_x \qquad (8.3.9)$$

where, V_{sd} : Shear to be carried by shear reinforcement computed at the section where the difference between the reduced shear and V_{cd} is greatest.

8.4 Planar members subject to transverse shear

8.4.1 General

For planar members subject to transverse shear due to concentrated loads check must be made of safety against punching shear failure in addition to the check for linear members.

8.4.2 Design value for punching shear strength

(1) Design values for punching shear strength must be determined taking account of concrete strength, geometry and positioning of loaded area, effective depth of the member, amount and arrangement of longitudinal reinforcement and eccentricity of the load.

(2) When the eccentricity of the load is small, design values for punching shear strength may be determined by Eq.(8.4.1).

$$V_{cpd}=f_{vd}\cdot u_p\cdot d/\gamma_b \qquad (8.4.1)$$

where, γ_b = 1.15 in general.

$$f_{vd}=\beta_r(1+\beta_{ds}+\beta_p)f_{vod}$$

$$\beta_r=2.0(0.85+0.4\,d/r)$$

$$\beta_{ds}=1.0-0.15\,d\geqq 0.60 \qquad (d: \mathrm{cm})$$

Definitions of f_{vod} and β_p are the same as given in Section 8.3.3(2).
d and p_w to be used in computing β_{ds} and β_p are the average values for the reinforcement in two directions. u_p is the peripheral length of the design cross section which is located $d/2$ from the periphery of the loaded area.

8.5 Check for planar members subject to in plane shear (in preparation)

9 TORSION

9.1 Notation

9.2 General

Structures must be designed for torsion so that their purposes of service are not impaired.
In general, check for ultimate limit state should be made according to Section 9.3.

9.3 Check for linear members

9.3.1 General

(1) When the influence of torsion is small or the torsion is compatibility torsion, the provisions of this Chapter may be omitted.

The occasion when the influence of torsion is small implies the case when the ratio of the design value for torsional moment Mtd to the design value for torsional moment strength Mtcd which doesn't take account of torsion reinforcement determined by Section 9.3.2 is less than or equal to 0.2.
(2) In check of safety for ultimate limit state, it must be ensured that safety against torsion is greater than safety against flexure and axial loads.
(3) In order to avoid rapid failure, design of cross sections must be made so that yield in torsion reinforcement precedes crush of concrete due to diagonal compression.

9.3.2 Design value for torsional moment strength

(1) Design value for torsional moment strength should be obtained taking account of concrete strength, cross sectional shapes and dimensions of members, ratio of applied torsional moment to applied shear or applied flexure and axial loads and type, quantity and arrangement of torsion reinforcement.
(2) When torsion reinforcement is not taken account of, design value for torsional moment strength may be obtained according to following provisions from (i) to (iii).

(i) Design value for torsional moment strength in member which doesn't take account of torsion reinforcement Mtcd may be obtained by Eq.(9.3.1).

$$M_{tcd}=\beta_{nt}\cdot K_t\cdot f_{td}/\gamma_b \qquad (9.3.1)$$

where, γ_b may be set equal to 1.15, in general.

$\beta_{nt}=\sqrt{1+\sigma'_{cd}/f_{td}}$ where, $1.0\leqq\beta_{nt}\leqq 2.5$

σ'_{cd} is average axial compressive stress of cross section of member.
K_t is section modulus for torsion as shown in Table 9.3.1.

(ii) When members which don't take account of torsion reinforcement are subject to combination loads of torsional moment and bending moment, design value for torsional moment strength may be obtained by Eq.(9.3.2).

$$M_{tud}=M_{tcd}\left[\sqrt{0.64\left(1.0-\frac{M_d}{M_{ud}}\right)}+0.2\right] \qquad (9.3.2)$$

where, $\frac{M_{td}}{M_{tcd}}\geqq 0.2$
is design value for flexure strength of member according to Section 7.2.

(iii) When members which don't take account of torsion reinforcement are subject to combination loads of torsional moment and shear force, design value for torsional moment strength may be obtained by Eq.(9.3.3).

$$M_{tud}=M_{tcd}\left(1-0.8\frac{V_d}{V_{cd}}\right) \qquad (9.3.3)$$

where, $\frac{M_{td}}{M_{tcd}}\geqq 0.2$

V_{cd} is design value for shear strength of member obtained by Eq.(8.3.2).

(3) When torsion reinforcement is taken account of, the upper limitation values of torsion reinforcement should be obtained by following provision of (i) and design values for torsional moment strength of members may be obtained by following provisions from (ii) to (iv).

Table 9.3.1 Factors concerning torsion

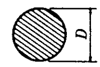
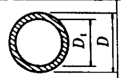
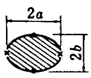
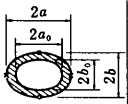
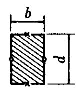
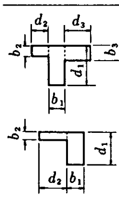
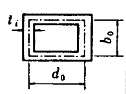
shape of section	K_t	J	remark
	$\frac{\pi D^3}{16}$	$\frac{\pi D^4}{32}$	
	$\frac{\pi(D^4-D_i^4)}{16D}$	$\frac{\pi(D^4-D_i^4)}{32}$	
	○点 $\pi ab^2/2$ ×点 $\pi a^2 b/2$	$\frac{\pi a^3 b^3}{a^2+b^2}$	
	○点 $\pi ab^2(1-q^4)/2$ ×点 $\pi a^2 b(1-q^4)/2$	$\frac{\pi a^3 b^3(1-q^4)}{a^2+b^2}$	$q=a_0/a$ $=b_0/b$
	○点 $b^2 d/\eta_1$ ×点 $b^2 d/(\eta_1 \cdot \eta_2)$	$\frac{b^3 d}{\eta_3}$	η_1, η_2, η_3 is shown in **Table 9.3.2**, respectively.
	$\Sigma \frac{b_i^2 d_i}{3.5\eta_{1i}}$ b_i, d_i is shorter side length and longer side length of each divided rectangular section, respectively.	$\Sigma \frac{b_i^3 d_i}{\eta_{3i}}$ η_{3i} is η_3 of each divided rectangular section.	
	$2A_m t_i$	$\frac{4A_m^2}{\Sigma(b_i/t_i)}$	A_m is area enclosed by center line of wall thickness. t_i is web thickness.

Table 9.3.2

d/b	1.0	1.2	1.5	2.0	2.5	3	4	5	7	10	20	∞
η_1	4.80	4.57	4.33	4.07	3.88	3.74	3.55	3.43	3.30	3.20	3.10	3.00
η_2	1.00	0.930	0.859	0.795	0.766	0.753	0.745	0.743	0.742	0.742	0.742	0.742
η_3	7.11	6.02	5.12	4.37	4.01	3.80	3.56	3.43	3.30	3.30	3.10	3.00

(i) Torsion reinforcement for rectangular sections should be made to be less than or equal to the equilibrium reinforcement (A_{tb}) which is obtained by Eq.(9.3.4).

for longitudinal reinforcement

$$A_{tbl}=\frac{A_m f'_{cd}}{f_{ld}}\ \frac{900}{f_{ld}\cdot d_0/(b_0+d_0)+2\,200}$$

(9.3.4)

for transverse reinforcement

$$A_{tbw}=\frac{A_m f'_{cd}}{f_{wd}}\ \frac{900(1+b_0/d_0)}{f_{wd}+4\,400}$$

For circular sections, b_0 in Eq.(9.3.4) may be set equal to d_0 .

(ii) In members which take account of torsion reinforcement, design values for torsional moment strength of rectangular or circular sections M_{tyd} may be obtained by Eq.(9.3.5).

$$M_{tyd}=1.6\,A_m\sqrt{m_t}\,\frac{A_{tw}f_{wd}}{s}\gamma_b \qquad (9.3.5)$$

where, γ_b , in general, may be set equal to 1.15.

$$m_t=\frac{\sum A_{tl}\cdot f_{ld}\cdot s}{A_{tw}\cdot f_{wd}\cdot u} \qquad 0.8\leqq m_t\leqq 1.25$$

Longitudinal reinforcement $\sum A_{tl}$ should be determined by Eq.(9.3.6).

$$\sum A_{tl}=m_t\frac{f_{wd}\cdot u}{f_{ld}\cdot s}A_{tw} \qquad (9.3.6)$$

(iii) For T shape, L shape and I shape sections, original section should be divided into small rectangular sections and Eq.(9.3.5) may be applied for each divided sections.

Design value for applied torsional moment of each divided rectangular sections should be determined by Eq.(9.3.7).

$$M_{tdi}=(G_{ci}\cdot J_i)/(\sum G_{ci}\cdot J_i)M_{td} \qquad (9.3.7)$$

where, M_{tdi} : design value for applied torsional moment of divided rectangular section

$\sum G_{ci}\cdot J_i$: summation value of torsional rigidity of divided rectangular sections

$G_{ci}\cdot J_i$: torsional rigidity of divided rectangular section

J : deformation modulus for torsion

Dividing original T, L and I sections into small rectangular sections should be made as shown in Fig.9.3.1 so that torsional rigidity may increase.

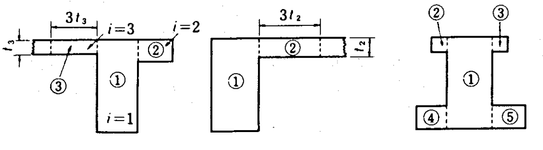


Fig. 9.3.1 dividing original T,L and I sections into small rectangular sections

Effective width of flange on one side to resist torsional moment (λ_t) should be determined as follows.

$$\lambda_t=3t_i \qquad (9.3.8)$$

where, $\lambda_t\leqq l_c$ for cantilever portion

$\lambda_t\leqq l_b/2$ for intermediate portion

then, t_i is average thickness of flange.

l_c is span length of flange.

l_b is span length of flange between girders.

(iv) For box sections, original section should be regarded as solid rectangular section enclosed with center line of wall thickness and Eq.(9.3.5) may be applied for this rectangular section.

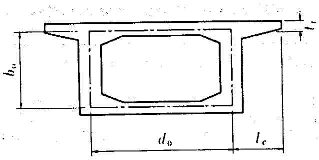


Fig. 9.3.2. box section

(v) When members which take account of torsion reinforcement are subject to combination loads of torsional moment and bending moment, design value for torsional moment strength may be obtained by Eq.(9.3.9).

$$M_{tud}=M_{tyd}\sqrt{1-\frac{M_d}{M_{ud}}} \qquad (9.3.9)$$

(vi) When members which take account of torsion reinforcement are subject to combination loads of torsional moment and shear force, design value for torsional moment strength may be obtained by Eq.(9.3.10).

$$M_{tud}=M_{tyd}\left(1-\frac{V_d}{V_{yd}}\right) \qquad (9.3.10)$$

where, V_{yd} is obtained by Eq.(8.3.3).

10 CRACK

10.1 Notation

10.2 General

(1) It must be checked that the purposes of service of structures are not impaired due to cracking in concrete.
(2) This Chapter deals with cracks due to bending moment, shear and axial loads.
(3) Check for crack is made as a rule by controling crack widths on the surface of concrete to stay below the permissible crack widths determined by environmental conditions and concrete cover.

10.3 Classification of environmental conditions

(1) When check for crack limit state is to be made, environmental conditions where the structure is exposed must be classified.
(2) Due to vulnerability to corrosion of reinforcement the environmental conditions are classified into the normal environment, the corrosive environment and the severely corrosive environment as shown in Table 10.3.1 in general.

Table 10.3.1 Classification of environmental conditions due to vulnerability to corrosion of reinforcement

Normal environment	Outdoors of ordinary condition
Corrosive environment	1. Marine structures submerged into seawater 2. When subject to detrimental chemical actions such as corrosive gases (e.q. SO_2), acids and salts
Severely corrosive environment	Marine structures exposed to the tides, sprashes or severe ocean winds

10.4 Permissible crack width

(1) Permissible crack widths w_{lim} should be determined taking account of importance of the structure, service life, scatter of crack widths, loading conditions, effects of axial loads, environmental conditions, concrete cover, type of reinforcing steel and stress conditions, in general.

(2) For reinforced concrete and prestressed concrete the values in Table 10.4.1 may be used based on environmental conditions, concrete cover and type of steel, in general. The variable c is concrete cover.

Table 10.4.1 Permissible crack width w_{lim} (unit : cm)

	Type of steel	Environmental conditions due to vulnerability to corrosion of reinforcement		
		Normal environment	Corrosive environment	Severely corrosive environment
Reinforced concrete	Deformed bars and round bars	0.005 c	0.004 c	0.0035 c
Prestressed concrete	Deformed bars	0.005 c	0.004 c	0.0035 c
	Prestressing steel	0.004 c	0.0035 c	0.003 c

10.5 Check for flexure cracks

(1) Prestressing steel and reinforcing bars which check for crack is to be made for should be those located nearest to the concrete surface. Stresses in prestressing steel and reinforcing bars should be obtained according to Section 7.4. The ordinary reinforcement to be used in prestressed concrete in longitudinal direction should be deformed bars.

(2) Check must be made so that increases in stress in reinforcing steel σ_{sp} and those in prestressing steel σ_{pp} due to member forces S_p caused by permanent load are smaller than the values in Table 10.5.1. σ_{sp} and σ_{pp} should be obtained as the variations from the values corresponding to the state where the stress in concrete adjacent to the steel is zero.

Table 10.5.1 Limiting values for the increases is stress in ordinary reinforcement σ_{sp} and those in prestressing steel σ_{pp} due to permanent load (unit : kg/cm²)

	Type of steel	Environmental conditions due to vulnerability to corrosion of reinforcement		
		Normal environment	Corrosive environment	Severely corrosive environment
Reinforced concrete	Deformed bars Round bars	1 200 1 000	1 000 800	800 600
Prestressed concrete	Deformed bars	1 200	1 000	800
	Prestressing steel	1 000	800	600

(3) It must be ensured that the ratios of permissible crack widths given in Table10.5.1 and the crack widths w obtained by Eq.(10.5.2) are greater than structures factor γ_i . In this case γ_i may be set equal to 1.0. The increases in stress in ordinary reinforcing steel σ_{se} and those in

prestressing steel σ_{pe} should be obtained in a same manner as given in (2), where member force S_e should be given by Eq.(10.5.3).

$$\frac{w_{lim}}{w} \geqq \gamma_i \qquad (10.5.1)$$

$$w = \gamma_a \frac{k_3 \sigma_s}{K} \sqrt[3]{c_\phi \left(c + \frac{\phi}{2} \right)^2} \qquad (10.5.2)$$

$$S_e = S_p + k_1 S_r \qquad (10.5.3)$$

where, γ_a should be equal to 1.2.

σ_s : σ_{se} , or σ_{pe}

k_1 : Constant to take account of influence of permanent and variable load on crack widths and corrosion of steel, which may be set equal to 0.5 in general.

k_3 : Constant to take account of influence of bond characteristics of bar, which may be set equal to 1.0 for deformed bars, 1.3 for round bars, and 1.3 for prestressing steel.

k : Constant, 600×10^3 (kg/cm²)

(4) When watertightness is important, check for cracking must be made by appropriate methods.

In such cases a method of check may be the one similar to that given in (3), but limiting the permissible crack widths to certain values smaller than those for the severely corrosive environment given in Table 10.4.1.

(5) When esthetics of structures is important, a method of check may be the one similar to that given in (3), but limiting the permissible crack widths to certain values considered acceptable for appearances.

10.6 Check for shear crack

When check for shear crack is necessary, check must be made by appropriate methods. A case where that check is necessary is when the ratios V_d/V_{cd} are greater than 2.

In general check is made by ensuring the stresses in shear reinforcement computed by Eq.(8.3.5) are smaller than the values in Table 10.5.1.

11 DEFLECTION

11.1 Notation

11.2 General

Deflections of a member must be checked not to impair functionality, beautiful sights, or durability of a structure. This check may be made for a serviceability limit state in general. In cases of member length sufficiently shorter than its hight, the check of deflections may be omitted.

11.3 Permisible deflection

Permissible deflections of a member shall be determined, considering a kind and purpose for use of a structure and kinds of loads.

11.4 Computation of short term deflection

(1) Short term deflections of concrete member, in which flexural cracking would not occur according to computation, may be calculated by theory of elasticity, assuming that a gross section is effective.

(2) Short term deflections of concrete member, in which flexural cracking would occur according to computation, shall be calculated with the consideration of the reduction in the stiffness.

In general, a moment of inertia of a transformed section may be calculated by Eq.(11.4.1) or Eq.(11.4.2).

(i) in cases of section stiffness to be changed according to bending moment

$$I_e=\left[\left(\frac{M_{crd}}{M_d}\right)^4 I_g+\left(1-\left(\frac{M_{crd}}{M_d}\right)^4\right)I_{cr}\right]\leqq I_g \qquad (11.4.1)$$

(ii) in cases of section stiffness to be kept constant all over the member length

$$I_e=\left[\left(\frac{M_{crd}}{M_{d\ max}}\right)^3 I_g+\left(1-\left(\frac{M_{crd}}{M_{d\ max}}\right)^3\right)I_{cr}\right]\leqq I_g \qquad (11.4.2)$$

where a moment of inertia of a transformed section in a continuous beam may be calculated by Eq.(11.4.2) at the section where the positive maximum moment takes place.

(3) The characteristic value shall be used for the modulus of elasticity.

11.5 Computation of long term deflection

Long term deflections must be analyzed by a proper method, considering the effect of shrinkage and creep due to permanent load.

12 FATIGUE

12.1 Notation

12.2 General

(1) Check of safety for fatigue must be made when the rate of variable load to total load is large and the frequency of variable load is high.

(2) For reinforced concrete beams, check for fatigue of tensile and shear reinforcement shall be made in general.

For reinforced concrete beams with light weight aggregate or under wet condition, however, check for fatigue of concrete as well as tensile and shear reinforcement shall be made.

(3) For reinforced concrete slabs, check for fatigue of main and distribution reinforcement shall be made in general.

(4) For reinforced concrete columns, check for fatigue may be omitted in general. In a case of large effect of bending moment or tensile axial force, however, check may be made as for reinforced concrete beams.

12.3 Method of check

(1) Check of safety for fatigue shall be made according to Section 2.8.

(2) In computing the equivalent cycles of loading, it is allowed that variable stresses are transformed to independent cyclic stresses by a proper method and that the linear damage rule (Miner's or the modified Miner's rule) is assumed.

12.4 Computation of stresses due to variable loads

(1) Tensile stress in steel due to bending shall be computed according to Section 7.4.

(2) Compressive stress in concrete due to bending shall be equal to the stress in the rectangular stress distribution whose resultant acts at the same point as the resultant of the triangular stress distribution obtained in Section 7.4.

(3) Stress in shear reinforcement shall be computed according to Section 8.3.4.

12.5 Design value of fatigue amplitude strength under variable loads

(1) A design value of fatigue amplitude strength in shear of member without shear reinforcement may be obtained by the following equation in general.

$$V_{rcd}=V_{cd}(1-V_{pd}/V_{cd})(1-\log N/11) \qquad (12.4.1)$$

where, V_{cd} is calculated by Eq.(8.3.2) and γ_b is 1.15.

(2) A design value of fatigue amplitude strength in punching shear of a reinforced concrete slab may be obtained by the following equation in general.

$$V_{rpd}=V_{cpd}(1-V_{pd}/V_{cpd})(1-\log N/14) \qquad (12.4.2)$$

where, V_{cpd} is calculated by Eq.(8.4.1) and γ_b is 1.15.

13 DEVELOPMENT OF REINFORCEMENT

13.1 Notation

13.2 General

An end of reinforcement must be embedded with the development length specified in Section 13.4 which is computed at the location specified in Section 13.3.

13.3 Location for computation of development length

Computation of development length for longitudinal tensile reinforcement in members for flexure shall be made, considering that the following location is a starting point for development, where l_s may be equal to an effective depth of member's section in general.

(1) Location at the distance of l_s from the section where flexural moment is at the peak value.

(2) Location at the distance of l_s in the direction of lessened flexural moment from the section where a part of reinforcement becomes unnecessary for flexural moment.

(3) Location at the distance of half a hight of section inside from the face of

footing in cases of lower end of column.
(4) Location at the distance of half an effective depth inside from face of supported part when an end of tensile reinforcement is confined by top and bottom sides of the supported part and location at the distance of an effective depth inside from the face of supported part when an end of tensile reinforcement is not confined by a top or bottom side of the supported part for a fixed end of a cantilever, etc.

13.4 Development length of reinforcement

(1) Development length of reinforcement, l_0 must be longer than basic development length calculated in Section 13.5, l_d . In cases of actually used reinforcement, A_s greater than reinforcement required by computation, A_{sc} , the development length of reinforcement may be reduced by Eq.(13.4.1).

$$l_0 \geqq l_d \cdot (A_c / A_s) \qquad (13.4.1)$$

where,

$l_0 \geqq l_d/3 \qquad l_0 \geqq 10\phi$

(2) How to evaluate development length of reinforcement whose embedded part is bent is as follows.

(i) When a radius of bend is equal to or greater than ten times the bar diameter, all the length of reinforcement including bent part is effective.
(ii) When a radius of bend is less than ten times the bar diameter, a straight part after bending is effective within a distance of ten times the bar diameter only when the straight part after bending is extended by the length equal to or greater than ten times the bar diameter.

(3) For a part, like a lower edge of column, where embedment of reinforcement is very important, it is a principle that a development length should be longer than a basic development length necessary for unhooked reinforcement and that hook should be placed at the end of reinforcement.
(4) A semicircular standard hook must be placed at the end of round bar.
(5) It is a principle that tensile reinforcement should be embedded in concrete subjected to no tensile stress. In case where one of the following two items is satisfied, however, tensile reinforcement may be embedded in concrete subjected to tensile stress. In this case the embedded part of tensile reinforcement must be extended by (l_d+l_s) from the section where tensile reinforcement becomes unnecessary according to computation.

(i) The design value of shear force at a section where reinforcement is cut is less than (2/(3 γ_i)) times the design value of shear capacity specified in Section 8.3.3.
(ii) Continuous reinforcement which is equal to or greater than twice the amount necessary for flexural moment is placed at a section where reinforcement is cut, and the design value of shear force at a section where reinforcement is cut is less than (3/(4 γ_i)) times the design value of shear capacity specified in Section 8.3.3.

(6) In cases of positive reinforcement in a slab or beam embedded over an edge support, the development length of the reinforcement must be started at the front face of the support and must be equal to or longer than the development length, l_0 for stress in reinforcement at the section which is at the distance of l_s from the front face of the support, and the reinforcement must be extended to the edge of member.

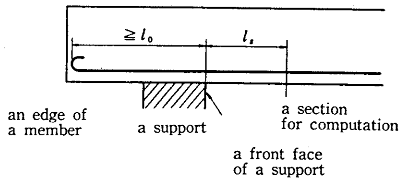


Fig. 13.4.1

(7) A development length of bent bar embedded in compressive concrete shall be equal to or greater than 15ϕ without hook and shall be equal to or greater than 10ϕ with hook.

13.5 Basic development length

Basic development length of reinforcement, l_d is obtained by Eq.(13.5.1).

$$l_d = \alpha_1\alpha_2\alpha_3 A_s f_{yd}/(u \cdot f_{bod}) \quad (13.5.1)$$

where, $f_{bod} = f_{bok}/\gamma_c$

The value of f_{bok} is given in Table 3.3.1. The value of γ_c may be 1.3 in general cases.

α_1 = 1.0 (in a case of tensile reinforcement)
= 0.8 (in a case of compressive reinforcement)

α_2 = 1.3 (in a case where embedded reinforcement is placed at the level greater than 30 cm from the bottom of concrete placed and with the angle less than 45 degree from the level)
= 1.0 (in all other cases)

α_3 = 1.0 (in case of $K_c \leqq 1.0$)
= 0.9 (in case of $1.0 < K_c \leqq 1.5$)
= 0.8 (in case of $1.5 < K_c \leqq 2.0$)
= 0.7 (in case of $2.0 < K_c \leqq 2.5$)
= 0.6 (in case of $2.5 < K_c$)

where, $K_c = (c/\phi) + (15 A_t/s\phi)$

In cases of tensile reinforcement with standard hook, a development length obtained by Eq.(13.5.1) may be reduced by 10ϕ. However, the development length is preferable to be equal to or greater than 20ϕ at least.

13.6 Lap splices

(1) When actually provided reinforcement is equal to or greater than twice reinforcement required by computation and the ratio of lap splices to total reinforcement is equal to or less than a half, lengths of lap splices must be equal to or greater than l_d .

(2) When one of the two conditions in (1) is not satisfied, lengths of lap splices must be equal to or greater than 1.3 l_d and the lap splices must be reinforced by transverse reinforcement or similar.

(3) When none of the two conditions in (1) is satisfied, length of lap splices must be equal to or greater than 1.7 l_d and the lap splices must be reinforced by transverse reinforcement or similar.

(4) In a case of lap splices subjected to low cycle fatigue, its length must be equal to or greater than 1.7 l_d and the lap splices must be reinforced by spiral reinforcement, metal devices for reinforcing connection, etc. as well as by placing hooks.

(5) Lengths of lap splices of stirrup shall be equal to or greater than $2l_d$.
(6) In any case lengths of lap splices shall be equal to or greater than 20ϕ.

14 STRUCTURAL DETAILS

14.1 Notation

14.2 General

General structural details which must be satisfied for design of reinforced and prestressed concrete structures will be shown. When structural details are determined separately for members or structures concerned, those details also must be satisfied.

14.3 Longitudinal reinforcement

14.3.1 Range of arrangement

Tensile reinforcement in a member for flexure shall have a development length starting at the point which is l_s away from the point where the tensile reinforcement becomes unnecessary according to computation, where the length, l_s is measured in direction in which flexural moment decreases and may be equal to an effective depth of member's section, d in general.

14.3.2 Minimum and maximum reinforcement

(1) For a reinforced concrete member with a great effect of axial force, longitudinal reinforcement equal to or greater than 0.8% of concrete area which is necessary according to computation must be arranged. The concrete area which is necessary according to computation is the miminum concrete area which is necessary to support the axial force alone. Even in a case of concrete area greater than that necessary according to computation, it is desired to arrange longitudinal reinforcement equal to or greater than 0.15% of the concrete area. The maximum longitudinal reinforcement must be arranged to satisfy spaces of reinforcement required by Section 14.3.3 and concrete covers required by Section 14.7 and must be equal to or less than 6% of the concrete area.
(2) For a reinforced concrete member with a great effect of flexure, longitudinal reinforcement equal to or greater than 0.2% of effective concrete area must be arranged. In case of longitudinal reinforcement equal to or greater than 4/3 times of the amount which is necessary according to computation, however, the amount may be reduced to 0.15%. The effective concrete area is the product of an effective depth of a beam, d and a web width, b_w . The maximum longitudinal reinforcement must be arranged to satisfy spaces of reinforcement required in Section 14.3.3 and concrete covers required in Section 14.7 and must be equal to or less than 75% of the balanced strain reinforcement ratio.

14.3.3 Spaces of reinforcement

(1) Horizontal spaces of longitudinal reinforcement in a beam must be equal to or greater than 2 cm, 4/3 times of the maximum size of aggregate, and the diameter of the reinforcement.

In case of longitudinal reinforcement arranged in two layers or more, its vertical space shall be equal to or more than 2 cm and the diameter of the reinforcement (see Fig.14.3.1).

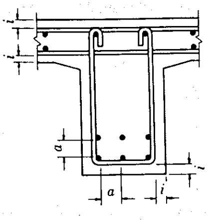
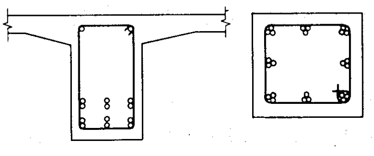


Fig. 14.3.1 Spaces of reinforcement and concrete covers

(2) Spaces of longitudinal reinforcement in a column must be equal to or greater than 4 cm, 4/3 of the maximum size of aggregate, and 1.5 of the diameter of the reinforcement.

(3) When deformed bars with a diameter of 32 mm or less are provided and the bars are arranged so complicatedly that insufficient compaction may be made, two horizontal longitudinal bars in a beam, slab, etc. may be bundled vertically and two or three vertical longitudinal bars in a column, wall, etc. may be bundled (see Fig.14.3.2).



(a) Beam (b) Column

Fig.14.3.2 Bundled reinforcement

In this case spaces are determined by the requirements (1) and (2), considering the bundled bars as a bar whose area is equal to the sum of the area of the bundled bars. Horizontal spaces must be provided properly because a rod type concrete vibrator is inserted.

14.4 Shear reinforcement

14.4.1 General

For a linear member subjected to shear force, shear reinforcement must be arranged according to the requirement in this section.

Shear reinforcement is consist of stirrup, bent bar, tie, and spiral reinforcement. Ends of stirrup and bent bar must be embedded in concrete of compressive side.

14.4.2 Minimum reinforcement

For a linear member, shear reinforcement given by Eq.14.4.1 at least shall be arranged all over the member length.

$$A_w = 0.002\, b_w\, s \sin\alpha \qquad (14.4.1)$$

14.4.3 Arrangement

For a linear member shear reinforcement must be arranged within the range consist of the range necessary according to computation and the length of an effective depth at its each outer side when shear reinforcement is necessary according to computation.

14.4.4 Diameter and space of reinforcement

Diameter of shear reinforcement shall be equal to or greater than 6 mm. For a linear member shear reinforcement must be arranged with space equal to or less than 3/4 times of an effective depth of a section.

14.5 Torsion reinforcement

14.5.1 General

For a linear member with a great effect of torsional moment, torsion reinforcement must be arranged according to the requirements in this Section. Torsion reinforcement is consist of longitudinal reinforcement and transverse reinforcement arranged in normal to longitudinal direction.

14.5.2 Minimum reinforcement

Even in a case without consideration of torsion reinforcement, reinforcement with a diameter of 6 mm or more shall be arranged to satisfy the following requirement.

Rectangular section : $A_t = 0.002\, b_0\, d_0\, s$

Circular section : $A_t = 0.002\, (\pi\, d_0^2/4)s$

where, A_t is total amount of longitudinal and transverse reinforcement.

14.5.3 Arrangement

(1) A longitudinal bar at least shall be arranged for each corner of a section as shown in Fig.14.5.1. For a circular section six longitudinal bars at least shall be arranged at the same interval as shown in Fig.14.5.1.

(2) In a case without consideration of torsion reinforcement, the minimum torsion reinforcement shall be arranged all over the length of a member. In a case with consideration of torsion reinforcement, computed amount of reinforcement shall be arranged in the range where the torsion reinforcement is necessary and in a length equal to a height or diameter of a section at both the sides of the range. The minimum reinforcement shall be arranged in the other parts.

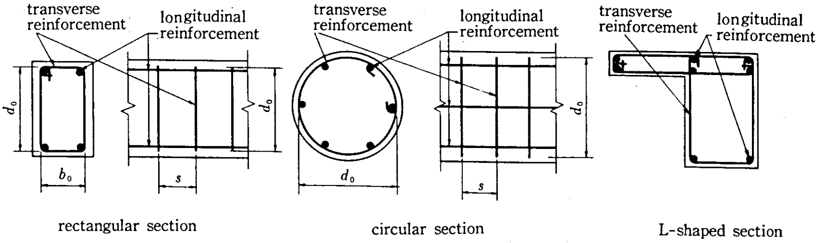


Fig. 14.5.1 Arrangement of longitudinal reinforcement

14.5.4 Shapes and sizes

(1) Torsion reinforcement shall be a composition of closed transverse reinforcement and longitudinal reinforcement normal to the transverse as shown in Fig.14.5.2.

(2) In a case without consideration of torsion reinforcement, spaces of transverse reinforcement, s shall be the smaller of 0.8 times of a member height or diameter and 30 cm.

In a case with consideration of torsion reinforcement, space of transverse and longitudinal reinforcement, s shall be the smaller of 0.8 times ember height or diameter and 30 cm.

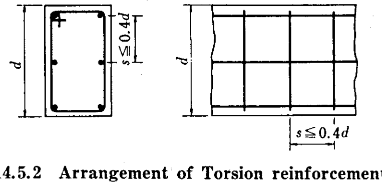


Fig. 14.5.2 Arrangement of Torsion reinforcement

14.6 Bent configurations of reinforcement

14.6.1 Standard hook

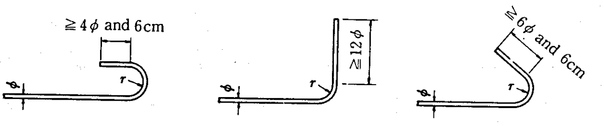
A semicircular, right-angled, or acute-angled hook shall be used for a standard hook.

The semicircular hook is a bar end which is bent at 180 degree to make a semicircle and has a straight extension equal to or greater than four times the bar diameter and 6 cm.

The right-angled hook is a bar end which is bent at 90 degree and has a straight extension equal to or greater than twelve times the bar diameter.

The acute-angled hook is a bar end which is bent at 135 degree and has a straight extension equal to or greater than six times the bar diameter and 6 cm (see Fig.14.6.1).

Reinforcement bent at less than 90 degree is supposed to be bent reinforcement and shall satisfy Section 13.3.



ϕ : bar diameter
r : radius of bend for bar (Table 14.6.1)

semicircular hook (round and deformed bar) | right-angled hook (deformed bar) | acute-angled hook (deformed bar)

Fig.14.6.1 Types of hook at end of bar

14.6.2 Longitudinal reinforcement

When round bars are provided as longitudinal reinforcement, a semicircular hook must be chosen. Radii of bend for longitudinal reinforcement shall be equal to or greater than the values in Table 14.6.1.

Table 14.6.1 Radii of bend of hook

Kinds		Notation	Radii of bend (r)	
			Hook	Stirrup and tie
Hot rolling bar	Type 1	SR24	2 ϕ	1 ϕ
	Type 2	SR30	2.5ϕ	2 ϕ
Hot rolling deformed bar	Type 1	SD24	2 ϕ	1 ϕ
	Type 2	SD30	2.5ϕ	2 ϕ
	Type 3	SD35	2.5ϕ	2 ϕ
	Type 4	SD40	3 ϕ	2.5ϕ

14.6.3 Stirrup and tie

There must be a standard hook at the end of stirrup and tie.

In a case of round bars provided as stirrup and tie, a semicircular hook must be chosen.

In a case of deformed bars provided as stirrup, a semicircular or acute-angled hook shall be chosen as a principle.

Radii of bend for stirrup and tie shall be equal to or greater than the values in Table 14.6.1. For stirrup with $\phi \leqq 10$ mm, however, radius of bend may be 1.5 ϕ.

14.6.4 Other reinforcement

(1) A radius of bend for a bent bar must be equal to or greater than five times the bar diameter (see Fig.14.6.2).

When reinforcement within (2 ϕ + 2 cm) from sides of a concrete member is provided as bent bar, the radius of bend must be equal to or greater than 7.5 times the bar diameter.

(2) A radius of bend for reinforcement along an outer side of a corner in a frame structure must be equal to or greater than ten times the bar diameter (see Fig.14.6.3).

(3) Reinforcement along an inner side of a corner, etc. in a haunch or frame shall not be tensile reinforcement bent in a slab or beam, but other straight reinforcement along the inner side of a haunch (see Fig.14.6.4).

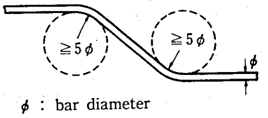


Fig. 14.6.2 **Radius of bend for bent bar**

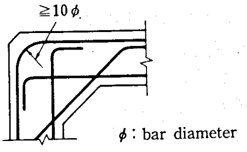


Fig. 14.6.3 **Radius of bend for reinforcement along the outer side of corner in a frame**

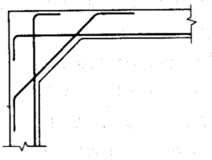


Fig. 14.6.4. **Reinforcement along the inner side of corner, etc. in a haunch or frame**

14.7 Concrete covers

(1) Standard concrete covers must be determined with consideration of quality of concrete, bar diameters, environmental conditions, errors in construction, and importance of a structure.

(2) Concrete covers must be equal to or greater than bar diameters.

(3) The minimum values of concrete covers shall be the values in Table 14.7.1. However, the minimum values for manufactured products are preferable to be determined separately with consideration.

(4) A standard value of a concrete cover shall be obtained by the following equation.

$$c_n = c_m + c_q + c_a + c_e \qquad (14.7.1)$$

where,
- c_n : concrete cover (cm)
- c_m : minimum concrete cover (values in Table 14.7.1)(cm)
- c_q : correction value taking account of quality of concrete (cm)
  -0.5 in case of $f'_{cd} \geqq 400$ kg/cm , 0 in the other cases
- c_a : correction value taking account of environmental condition (cm)
  +1 in case of member subjected to icing and melting, 0 in the other cases
- c_e : correction value taking account of error in construction (cm)
  +0.5 in case of cast-in-place, 0 in the other cases

Table 14.7.1 **Minimum concrete covers** (cm)

Environmental conditions \ Members		Slab	Beam	Column	Wall
a	Important structure with large scale or structure exposed to wind and rain	2.0	2.5	3.0	2.5
b	Part without effective layer for protection which may be subjected to harmful action of smoke, acid, oil, and salts	3.0	3.5	4.0	3.5

Table 14.7.2 Minimum concrete covers for a structure subjected to action of sea water (cm)

a	Part touching directly sea water, part washed by sea water, and part exposed to severe sea wind	7.0
b	Part other than above	5.0

(5) In cases of an important member of a footing and structure, concrete cover is preferable to be equal to or greater than 7.5 cm when concrete is cast directly in the ground, and to be equal to or greater than 5 cm for a bar with a diameter of 16 mm or more or 4 cm for a bar with a diameter less than 16 mm when the member touches directly ground put back or action of weather is severe.
(6) Concrete covers for a structure subjected to action of sea water must be equal to or greater than the values in Table 14.7.2.
(7) Concrete covers for a structure constructed under water must be equal to or greater than 10 cm.
(8) For a part probably rubbed off by running water, concrete covers must be increased adequately.
(9) Concrete covers for a structure which needs especially fireproof must be determined taking account of heat temperature, heating time, characteristics of aggregate provided, etc.
(10) In a case of bundled deformed bars, concrete covers may be determined assuming that the bundled bars is a bar with area equal to their area.

14.8 Development of reinforcement

(1) An end of reinforcement must be embedded sufficiently in concrete, and a force in reinforcement must be developed by bond forces between reinforcement and concrete, a hook, or a mechanical device.
(2) At least one third of positive reinforcement in a slab or beam must be embedded over a support without bend.
(3) At least one third of negative reinforcement in a slab or beam must be extended over an inflection point and be embedded in a compressive side, or must be continued to the next negative reinforcement.
(4) Extension of a bent bar is preferable to be used as positive or negative reinforcement, or an end of a bent bar is preferable to be close to a top or bottom surface of a beam as much as possible with necessary concrete covers, to be extended parallel to the top or bottom surface, and to be embedded in a compressive side of concrete.
(5) Stirrup must enclose positive or negative reinforcement and its end must be embedded in a compressive side of concrete.
(6) At an end of tie there must exist a semicircular or an acute-angled hook enclosing longitudinal reinforcement.
(7) Spiral reinforcement shall be embedded with extra one and a half wind.

14.9 Splices in reinforcement

(1) Proper splices in reinforcement must be chosen according to a kind, a diameter, a stress state, and a location of splices of reinforcement.
(2) For a location of splices, a section with great stresses applied shall be avoided as much as possible.

(3) In order not to gather splices at the same section, a standard distance to slide locations of splices mutually in a longitudinal direction shall be equal to or greater than a length of the splice plus 25 times the bar diameter.
(4) A space between spliced part and the next reinforcement or between spliced parts shall be equal to or greater than the maximum size of aggregate.
(5) In case of splices constructed after arranging reinforcement, a space large enough to insert a machine, etc. for construction of the splice must be taken.
(6) Concrete covers for spliced part shall satisfy the requirements in Section 14.7.

14.10 Chamfers and fillets

A corner of a member must be chamfered or filleted.

Especially, in a case of a cold place or severe weather action, it must be thought carefully how much chamfers or fillets would be made. In such a case the chamfers or fillets must be indicated in a plan.

14.11 Precaution reinforcement for exposed surface

In order to avoid harmful cracks due to shrinkage and change in temperature, precaution reinforcement must be arranged close to an exposed surface in concrete which has a large exposed surface.

14.12 Reinforcing the portion subjected to concentrated reactions

The portion subjected to concentrated reactions, etc., in which excessive stress concentration takes place definitely, must be reinforced taking account of the effect of the stress concentration.

14.13 Reinforcing the periphery of openings

Around openings in a slab, wall, etc. reinforcement must be arranged against cracking due to stress concentration, etc.

14.14 Construction joints

Locations and directions of construction joints must be determined without diminishing strength and good appearance of a structure.

Important construction joints are preferable to be indicated in a plan.

14.15 Expansion joints

Locations and directions of expansion joints must be determined to make the most effective protection against cracking in a structure and to let the structure make free movements due to its shrinkage, etc. as much as possible and must be indicated in a plan.

14.16 Watertight structures

In structures which need to be watertight, arrangement of reinforcement, spacing and arrangement of construction and expansion joints, etc. must be determined to prevent cracks from taking place.

14.17 Drainage and water stop

(1) In structures touching water, if necessary, drainage and water stop must be taken account.
(2) It is a principle that water stop should be made in a surface subjected to direct water pressure.

14.18 Protection of concrete surface

In order to make durable the portion subjected to severe actions like rubbing, deteriorating, impacting, etc., concrete surface must be protected by adequate material.

14.19 Haunches

For connections of members in a frame, supports of a fixed slab and beam, supports of a continuous slab and beam, etc., haunches shall be put as a standard.

15 SEISMIC DESIGN

15.1 Notation

15.2 General requirements for seismic design

Check of safety of structures against earthquakes should be made for ultimate limit state, as a rule. In general check should be made for the ultimate limit state of failure of cross sections and the ultimate limit state of displacements. Check for the ultimate limit state of stability of rigid body motion should also be made, when it is necessary.

15.3 Earthquake loads

(1) Earthquake loads should be determined considering the characteristics of earthquake motions at the site of construction and the characteristics of structures, as a rule.
(2) For the earthquake loads horizontal earthquake loads should be considered in general, and when it is necessary vertical earthquake loads should also be considered. The intensity of the vertical earthquake loads may be set equal to 1/2 of the horizontal component.
(3) The earthquake loads used for check for the ultimate limit state of failure of cross sections may be in general considered to be static loads which are 0.2 times the weight and acting in horizontal direction. Corrections to that value should be made for activeness of earthquakes at the site of construction and ground conditions, and when it is necessary natural periods and importance of the structure.
(4) For the earthquake loads used for check for the ultimate limit state of displacement the intensity should be such that the structure is to be subjected one time or less during its service life.

15.4 Combination of loads and safety factors

(1) Check of safety should be made as a rule for combination of earthquake loads and permanent loads.

(2) When among the permanent loads earth pressure or water pressure are affected by earthquakes or when the weight of liquid is affected by vertical earthquake motions, these effects must be cosidered.

(3) For the earthquake loads to be used in seismic design appropriate safety factors should be used. In general following values may be used.

Load factor γ_f = 1.15

Load modification factor ρ_f = 1.0

Structures factor γ_i = 1.0

Structural analysis factor γ_a = 1.0

Load combination factor ψ = 1.0

15.5 Structural analysis

(1) Member forces to be used for check for the ultimate limit state of failure of cross sections may be obtained by static linear analyses. For that computation the moment of inertia and materials constants should be determined by Section 6.7.

(2) The response displacements for check for the ultimate limit state of displacements should be obtained as a rule by nonlinear analyses.

(3) The response displacements may be obtained by Eq.(15.5.1) when static nonlinear analysis is used in lieu of (2).

$$\delta=(1/2)\,(1+\gamma^2)\delta_y \qquad (15.5.1)$$

where, δ : Response displacement

δ_y : Displacement at the earthquake intensity of k_{hd_1}

γ : k_{hd_2}/k_{hd_1}

k_{hd_1} : Earthquake intensity when a failure of a cross section was predicted in the structure

k_{hd_2} : Earthquake intensity two or three times of k_{hd_1}

15.6 Computation of strengths of cross sections and ultimate displacements

(1) Computation of strengths of cross sections should be made by Sections 7.3, 8.3 and 9.3. The ultimate strain in steel should not exceed 2% in general.

(2) For computations of the ultimate displacements the assumptions in Section 7.3 may be used. The ultimate strain in steel should not exceed 10%.

The ultimate displacement is the maximum displacement that the structure can undergo before the resisting moment at any cross section lowers, due to inelastic deformation of the section, below the resisting moment corresponding to the commencement of yield in steel at that cross section.

15.7 Check of safety

(1) Check of safety for the ultimate limit state of failure of cross sections should be made by ensuring that the ratios of the design values of strengths of cross sections obtained by Section 15.6 and the design values of member forces obtained by Section 15.5(1) are greater than structures factor γ_i .

(2) Check of safety for the ultimate limit state of displacement should be made

by ensuring that the response displacements obtained by Sections 15.5(2) or 15.5(3) do not exceed the ultimate displacement obtained by Section 15.6 for the structure or for the member, and when it is necessary by ensuring that the displacement limits determined by the functions of the structures are not exceeded.

Also, when it is necessary, check should be made for the limit state of stability of rigid body motion.

15.8 Structural details

15.8.1 General

To provided the members with required ductility the reinforcement ratios for stirrups, ties and longitudinal reinforcement must be determined as appropriate and taking account of the geometry of the members.

15.8.2 Anchorage of reinforcement in tension zone

When anchoring tension reinforcement in the intermediate tension zone of the members, the following conditions must be satisfied.

(i) The cross sectional area of the bars to be terminated should be less than 1/2 of the total cross sectional area of the bars at that section.

(ii) The end of the bars to be terminated should be located beyond the point at a distance equal to member effective depth or development length of the bars, whichever is greater, apart from the section where the resisting moment provided by continuing bars is greater than 1.5 times the design moment, and the resisting shear between the two locations is greater 1.5 times the design shear.

15.8.3 Stirrups

Stirrups should securely link tension bars and compression bars in the beams and confine the concrete inside the main reinforcing bars by bending the ends of the stirrups more than 135 degrees and anchoring the ends by hooking to the compression bars, or by employing closed shape stirrups enclosing the main reinforcing bars (Fig.15.8.1). Also, the maximum spaces of the stirrups should be the smallest of the following requirements in the region between the junction of the members and a section 1.5 times the beam effective depth apart from it.

(i) Less than 1/4 of the member effective depth.

(ii) Less than 8 times the diameter of the main bar.

(iii) Less than 24 times the diameter of the stirrup.

(iv) Less than 30 cm.

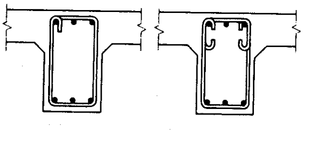


Fig. 15.8.1 Shape of stirrups

15.8.4 Ties

(1) The ends of ties should be bent more than 135 degrees and anchored in the interior concrete, or welded together to develop the yield strength of the bars, or the ties should be continuous spirals.

(2) To increase strength and ductility of cross sections it is preferable to arrange several ties to form a mesh (Fig.15.8.2) or to use intermediate ties (Fig.15.8.3).

(3) The maximum spaces of the ties should be less than 1/4 of the minimum dimension of the member in the region between the junction of the members and a location two times the column width apart from it.

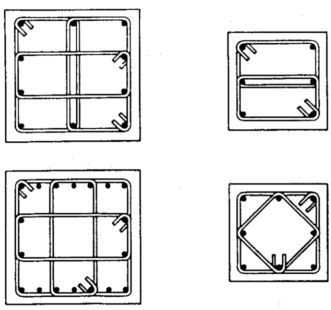


Fig. 15.8.2 Arrangement of ties

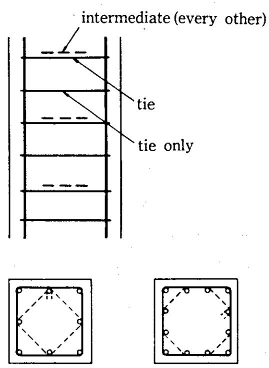


Fig. 15.8.3 Arrangement of intermediate ties

15.8.5 Lap splices of the bars

(1) Lap splices in the main bars should not be used in the joints of the members and in the regions between the junction of the members and a section two times the member effective depth apart from it. When splices are used in other regions, they should be sufficiently reinforced either by providing stirrups at spacings less than 1/4 of the member effective depth and less than 10 cm, or by other appropriate methods.

(2) Lap splices in the column main bars are allowed only near the middle of the members. The requirements for the lap splices in tension bars may be used for the case.

16 STEEL AND REINFORCED CONCRETE

16.1 Notation

16.2 General

This chapter shall apply for design of concrete structures or members which consist of embedded structural steel shapes, steel plates or steel fabrication with accompaning reinforcement and prestressing steel. This chapter shall not apply for such structural systems that the concrete-filled steel pipes or rectangular columns and that where steel members, not embedded in concrete, are combined with concrete through studs on the surfaces of concrete.

16.3 General requirements for the performance of steel and reinforced concrete

The following requiements must be satisfied so that members or concrete may be treated as steel and reinforced concrete.

(1) Steel shall be embedded in concrete and check for buckling is omitted.
(2) The collaboration of concrete with steel in a body shall be maintained to some degree.
(3) Concrete is not teared due to the separation at the interface between concrete and steel.
(4) Adequate concrete protection for steel shall be provided.

16.4 General requirements

16.4.1 Selection of steel

It is a principle that reinforcement and steel, whose yield points are almost the equal as each other, shall be used together.

16.4.2 Stress in construction stage

For check for applied loads, the steel stress introduced in construction stage should be also checked.

16.4.3 Computation method for design

The cross section and members shall have required performances in the ultimate limit state, and in the serviceability limit state, the requirements shall be satisfied for the applied loads. On this occasion, check shall be made with assumption that steel and concrete work in a body.

16.5 Classification of structural systems

The steel and reinforced concrete structure treated in this chapter is one of the following structural systems.

(1) structure with integrated strengths
(2) structure with reinforcement and steel in combination
(3) structure during erection

16.6 Check for serviceability limit state

16.6.1 Check for crack

(1) When the check for the limit state of crack is necessary, check is made as a standard by controlling the steel stress calculated assuming part of steel as the equivalent reinforcement area to stay below the value stipulated in Table 16.6.1.

Table 16.6.1 Upper limit of tensile steel stress

A_{ss}/A_s (%)	Upper limit value of tensile stress (kg/cm²)
30	1 800
50	1 600
70	1 400

where, A_{ss} : area of tensile steel frames
A_s : area of tensile steel

(2) When cracking is especially harmful for the durability of structures etc., appropriate measures shall be taken according to circumstances, or the values in Table 16.6.1 shall be reduced.

16.6.2 Check for collaboration of concrete with steel in a body

For check for the serviceability limit state, the collaboration of concrete with steel shall be ensured by controlling the bond stress between both materials not to exceed 20% of the value shown in Table 3.3.1.

16.6.3 Check for fatigue

Where check for the fatigue limit state is nesessary, check may be made in accordance with the provisions in Chapter 12 based on the computed reinforcement stress assuming part of steel as the equivalent reinforcement area.

16.7 Check for ultimate limit state

16.7.1 Flexure and axial loads

(1) The strength of the structure with integrated strengths shall be taken as the sum of the strength assigned independently to the reinforced concrete alone and that of the steel part.

(2) The strength of the structure with reinforcement and steel in combination shall be computed converting steel into reinforcement in accordance with Chapter 7.

16.7.2 Shear

(1) The shear strength of the structure with integrated strengths shall be determined as the steel shear strength plus that of the reinforced concrete.

$$V_{sry}=V_{ry}+V_{sy} \qquad (16.7.1)$$

where, V_{sry} : shear strength for SRC member classified into the structure with integrated strengths
V_{ry} : shear strength for part of reinforced concrete in accordance with Eq.(8.3.3)
V_{sy} : shear strength for steel part

(2) The shear strength of the structure with reinforcement and steel in combination shall be computed in which steel is taken as the equivalent reinforcement in accordance with Chapter 8.

16.7.3 Torsion

(1) The torsional moment strength of the structure with integrated strengths shall be the torsional moment strength for steel part plus that for reinforced concrete part.
(2) The torsional moment strength of the structure with reinforcment and steel in combination shall be computed in accordance with chapter 9 by converting steel into equivalent reinforcement.

16.8 Check for construction stage and structures during erection

(1) Check for buckling of steel shall be made before composition with concrete is performed.
(2) For steel used only during erection, the stiffness required and the influences on the failure mode of members in employed stage shall be checked in addition to the check mentioned above in (1).

16.9 Design of connections

(1) Corners of frames shall be the structures with adequate performances to transfer flexure moment, shear force, torsional moment and axial loads acting on beams to columns.
(2) For footings of columns, transmission of column loads through floor systems shall be provided well.

16.10 Structural details

16.10.1 Maximum and minimum steel

(1) The area of longitudinal steel shall not exceed 8% of concrete area and minimum steel area should be in accordance with 14.3.2. Especially the area of the longitudinal reinforcement shall be arranged greater than one-half of the minimum value required in 14.3.2.
(2) The minimum steel for shear reinforcement shall be according to 14.4, and stirrups and ties must be arranged so that they may enclose all longitudinal steel. Stirrups and ties shall be arranged greater than one half of the minimum value required in 14.4.

16.10.2 Concrete cover

(1) Concrete cover between concrete surface and surface of steel shall be equal to or greater than 10cm.
(2) Concrete cover between concrete surface and surface reinforcement shall be determined in accordance with 14.7.

16.10.3 Connection of steel and handling of end

(1) Connection of steel must be placed at the portion where the strength of cross section is enough for failure and must be designed to possess at least more than 75% of strength of material itself. The drop of strength due to placing connections shall not let the cross section be the critical one.
(2) The end of the steel in the tensile and compressive parts in the vicinity of

the free end of a cantilever and etc., must be adequately developed by connecting both ends mutually with steel and etc.

16.10.4 Splices and developement of reinforcement
(1) Theof splices of reinforcement shall be avoided to locate the same section of steel connections.
(2) Requirements for splices and developement of reinforcement is in acordance with 14.8 and 14.9.

16.10.5 Spaces of reinforcement and steel
(1) Spaces of reinforcement should be accordance with 14.3.3.
(2) Spaces between reinforcement and steel shall be greater than 4cm or 4/3 times of the maximum size of aggregate.

17 PRESTRESSED CONCRETE

17.1 Notation

17.2 General

The level of prestress shall be selected to optimize the structural performance in all limit states to be considered, and thus for the structure to serve its purposes safely and economically.

17.3 Check for serviceability limit state

17.3.1 Check for flexure

(1) Check for crack

For the limit states concerning cracking check shall be made by ensuring that one of the following requirements (i)-(iii) is satisfied. The selection of the requirement shall be most appropriate for the purposes of service of the structures, the importance of the structures, the environmental conditions and others.

(i) No tensile stresses shall occur under design loads.

(ii) The increase in the stresses in reinforcing bars and prestressing steel in cracked sections due to member forces S_e shall not exceed 1/2 of the values given in Table 10.5.1 under design loads. Also, (i) shall be satisfied for the member forces S_p due to permanent loads multiplied by k_p, where k_p is a reduction factor which depends on the ratio of the permanent load to the total load and environmental conditions.

(iii) The crack widths shall be limited according to Section 10.5.

(2) Check for stresses in concrete and steel

(i) Compressive stresses in concrete shall not exceed the following values.

Flexural compressive stress ----- $0.35\, f'_{ck}$

Biaxial flexural compressive stress
in the corner of cross section ----- $0.40 f'_{ck}$
Uniaxial compressive stress ----- $0.30 f'_{ck}$

(ii) Tensile stresses in prestressing steel shall not exceed $0.60 f_{pu}$ and $0.75 f_{py}$.

(3) Check for deflection

The requirements of Chapter 11 shall be satisfied.

17.3.2 Check for shear and torsion

(1) For the members in which flexural cracking is not allowed under design loads check shall be made by ensuring that the ratios of the tensile principal stress due to shear and torsion to the limiting value for tensile stress are smaller than the inverse of structure factor γ_i . The limiting value for tensile stress is the characteristic value of tensile strength multiplied by a factor β_{cr} which is a reduction factor determined depending on the danger of diagonal cracking.

(2) When flexural cracking is permissible under design loads the requirements of Chapters 8 and 9 shall be satisfied.

17.4 Check for ultimate limit state

Check for the ultimate limit state shall be made according to Chapter 7 for flexure, Chapter 8 for shear and Chapter 9 for torsion.

17.5 Check for fatigue limit state

Check for fatigue limit state shall be made according to Chapter 12. For the members in which cracking is not permissible for serviceability limit state under design loads check by this section is not required.

17.6 Check for prestress transfer

(1) The tensile stress in prestressing steel shall be the smallest of the values given in Table 17.6.1.

Table 17.6.1 Limiting values for tensile stress in prestressing steel at prestress transfer

	Permissible values of tensile stress in prestressing steel
During prestressing	$0.80 f_{pu}$ or $0.90 f_{py}$
Immediately after prestress transfer	$0.70 f_{pu}$ or $0.85 f_{py}$

(2) Check for the stresses in concrete shall be made according to (i) and (ii) for the most unfavorable combination of the characteristic values of prestress immediately after prestress transfer and accompanying loads.

(i) It shall be ensured that tensile stress in concrete does not exceed 1/1.7 of the characteristic value for the tensile strength of concrete at the age of prestress transfer.

(ii) It shall be ensured that compressive stress in concrete does not exceed 1/1.7 of the characteristic value for the compressive strength of concrete at the age of prestress transfer.

17.7 Structural details

17.7.1 Spaces between tendons

(1) For pretensioned members spaces between tendons at the member ends shall be greater than 3 times the diameter of tendons in both horizontal and vertical directions, and the horizontal spaces shall be greater than 4/3 times the maximum size of coarse aggregate.

(2) In post-tensioned members the spaces between the sheaths shall be greater than 4/3 times the maximum size of coarse aggregate. Two of the smaller size sheaths may be bundled in the horizontal plane. In that case the horizontal spaces between the bundles shall be greater than 6 cm and 1.6 times the diameter of the single sheath, and the vertical spaces shall be greater than the vertical dimension of the sheath.

When the internal vibrators are to be used the sheaths and reinforcing bars shall be so arranged that the internal vibrators can be inserted at the required intervals.

(3) In the structures near the seashores the tendons should not be bundled as a rule.

17.7.2 Concrete cover for ordinary reinforcing bars

The concrete cover for ordinary reinforcing bars shall satisfy the requirements of Section 14.7.

17.7.3 Concrete cover for prestressing steel

(1) For pretensioned members concrete cover for the tendons shall be greater than the values given in Section 14.7.

(2) For post-tensioned members concrete cover for the sheaths or bundles of sheaths shall be greater than the values given in Section 14.7, 4 cm in vertical and horizontal directions, and the horizontal dimension of the sheath or the bundle of the sheaths.

17.7.4 Arrangement of tendon anchorages and couplers

(1) The tendon anchorages shall be positioned in a manner such that the required prestress is effected at every design section and such that the tendons are securely anchored.

(2) When several anchorages are to be placed in a single cross section the geometry and dimensions of the concrete portion housing the anchorages shall be designed to possess adequate safety taking account of the number of anchorages, the force to be anchored, the minimum distances required between the anchorages and others.

(3) Near the end supports of the beams some of the tendons shall be placed as close to the bottom edge of the member, and the anchorages as close to the bottom of the end of the member as practical. When this arrangement is difficult, the region shall be reinforced with additional ordinary reinforcing bars in the longitudinal direction.

17.7.5 Protection of anchorage zone

The structure of the anchorage zones shall be such that is safe against failure and deterioration.

17.7.6 Reinforcing the concrete in the vicinity of anchorages

(1) The concrete in the vicinity of anchorages shall be reinforced against tensile stresses developing in transverse directions to the tendons by the shaped, grid or spiral reinforcement.

(2) When anchorages are placed in the intermediate regions of the members reinforcement shall be provided for tensile stresses developing in the concrete near the anchorages.

17.7.7 Precaution reinforcement

(1) In post-tensioned members the cross sectional area of reinforcement greater than 0.1% of the gross cross sectional area of the member shall be provided.

In tensile zone of the members longitudinal reinforcing bars shall be placed at the spaces smaller than 30 cm, and the bar diameters shall be greater than 9 mm.

(2) Precaution reinforcement shall be provided to enclose the portions subject to high compressive stresses.

(3) Precaution reinforcement shall be provided in the portions where cracking is likely due to temperature differences.

(4) When there is broad area of exposed surface of concrete precaution reinforcement shall be provided near the surface to prevent detrimental cracks due to drying shrinkage and temperature changes.

(5) When detrimental tensile stresses are suspected to occur in concrete due to deformation of scaffolding during construction, precaution reinforcement shall be provided.

(6) Where excessive stress concentration is obvious due to concentrated reactions, reinforcement shall be provided taking account of that effect.

(7) In the periphery of the openings in slabs and walls reinforcement shall be provided against cracking due to stress concentration and other effects.

17.7.8 Consideration for durability

The portions of marine structures washed by sea water, or the portions of structures exposed to sea water sprashes or severe ocean winds shall be designed to be of the geometric forms such that the surface exposed to the exterior is as small as practicable. The shape of the cross sections and arrangement of reinforcement shall be such that allow snug packing of concrete without substantial difficulties.

18 DESIGN OF MEMBERS

18.1 Notation

18.2 General

(1) Structures shall be designed with the consideration of their geometry, support and load conditions and so on.
(2) For design of structures, simplified structural models, such as beams, columns, walls, frames, arches, shells, flat slabs and their combinations may be assumed corresponding to their geometry in general.
(3) Member forces shall be computed by linear analyses in general.

18.3 Slab

18.3.1 General

It is a principle that bending moment, shear force, torsional moment and support reactions of slabs should be computed by thin plate theories taking account of their support conditions, geometry and load conditions.

For ultimate limit states of slabs, however, check may be made based on plastic analyses which certainly provide safe estimations.

18.3.2 Span length

The span length for analyses shall be taken as center-to-center of supports. However, for long slab supports, the span length shall be considered the clear span plus the slab thickness. For slabs monolithically built with rigid walls or beams, the clear span may be adopted in computation.

18.3.3 Distributed width of concentrated load

For structural analyses of slabs, load acting on the slab surface shall be modeled to be distributed on the area located at a distance 1/2 of the slab thickness from the periphery of loaded slab face with the similar shape as that of the actual load area.

When concrete or asphalt-concrete is used for slab overlayers, full thickness of the overlayer shall be added to the distance mentioned above. In case of soft materials used for overlayers, 3/4 times the slab overlayer thickness is used in computation.

18.3.4 Check for flexure

Check for flexure of slabs may be made in accordance with beams. Where the direction of the primary bending moment does not coincide with reinforcement direction, strengths for flexure shall be provided sufficiently in all directions.

18.3.5 Check for shear

Check for shear of slabs shall be made for the following two provisions.

(1) Check shall be made in accordance with beams by assuming slabs to act as wide beams and to resist shear in the cross section of a single direction in the vicinity of supports.

(2) Around the concentrated load or near the support, the check for punching shear failure shall be made by assuming the resisting section against shear to be along a conical cone or a pyramid in two directions.

18.3.6 Cantilever slabs

(1) For wide cantilever slabs subjected to a single concentrated load, the maximum bending moment per unit slab width may be obtained with the assumption that the effective width is two times the distance between the support face and the loading portion (See Fig.18.3.1).

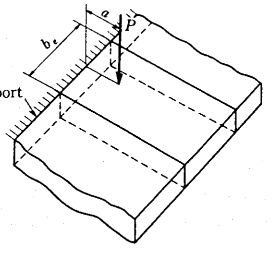


Fig. 18.3.1 Effective width of cantilever slab

(2) In the case of uniform load, the bending moment in the span direction may be computed in accordance with beam by assuming cantilever slabs to act as cantilever beams.

(3) For the primary reinforcement of cantilever slab, hook shall be provided. When cantilever slab is greatly loaded at the end, primary reinforcement shall be bent at the slab end and developed along lower face of slab (See Fig.18.3.2).

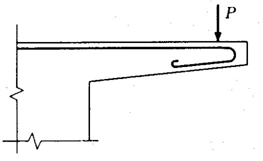


Fig. 18.3.2 Development of primary rainforcement of cantilever slab subject to greater load at the edge

(4) When the bending moment of cantilever slabs is computed by (1) and (2), the distribution reinforcement per 1 m in slab length shall be, in general, equal to or greater than 1/6 of the area of the primary tensile reinforcement per 1 m in slab width. Along the lower face of the cantilever slab, reinforcing bars which diameter is greater than 6 mm shall be arranged normal to the span direction, and the spaces between reinforcements shall not exceed three times the slab thickness.

18.3.7 One-way slabs

(1) For the simply supported one-way slab subjected to a single concentrated load, the maximum bending moment per unit slab width may be computed based on the effective width defined as the load distribution width plus $2.4x(1-x/l)$.

where, x : distance between the section considered and the adjacent support

l : span length

the effective width $b_e=v+2.4x(1-x/l)$, the effective width $b_e=c+v+1.2x(1-x/l)$

the maximum bending moment per unit width$=Px(1-x/l)(1-u/2l)/b_e$

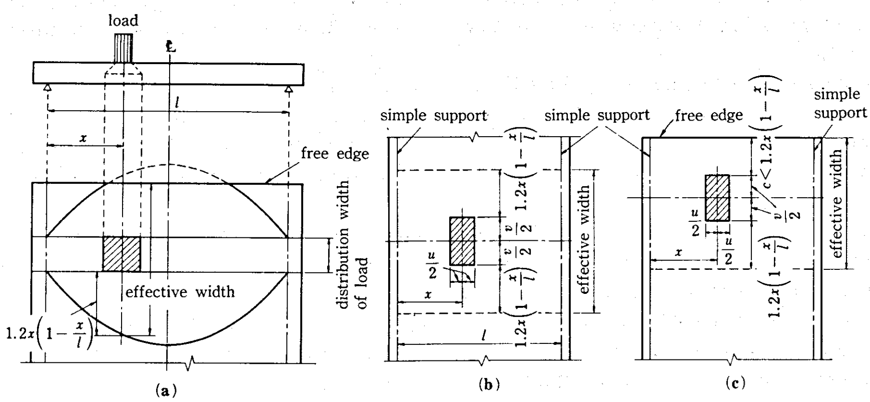


Fig. 18.3.3 Effective width of one-way slab

(2) When a concentrated load is located in the vicinity of the free edge of slabs, the effective width shall not exceed the value given in (1), and the value of the distance from the free edge to the center of the distribution width plus one half the effective width stipulated in (1).

(3) For simply supported one-way slabs subjected to uniform loads, the distribution reinforcement per 1 m in slab length shall be arranged, in general, greater than 1/6 of the area of the primary tensile reinforcement per 1 m in slab width.

For the concentrated load, distribution reinforcement of α times the area of the primary tensile reinforcement necessary for the concentrated load per in slab length shall be arranged in addition to the distribution reinforcement stipulated above. α shall be defined as follows.

(i) In the case of loading in the vicinity of the slab center

lower side distribution reinforcement $\alpha=(1-0.25\cdot l/b)(1-0.8\cdot v/b)$

where, for $l/b>2.5$, the value of α corresponding to $l/b=2.5$ is used.

(ii) In the case of loading near the single side edge of slabs

upper side distribution reinforcement $\alpha=(1-2\cdot v/b)/8$

where, l : span length of slab

b : slab width

v : load distribution width

18.3.8 Two-way slabs

(1) When two-way slabs whose short to long span ratios do not exceed 0.4 are loaded uniformly, member forces may be computed assuming that slabs are equivalent to one-way slabs which loads are supported only along the short

span.
(2) Other than for (1), thin plate theories or approximation methods shall be used for computing member forces of slabs.
(3) In the case of slabs discussed in (1), distribution reinforcement not less than 1/4 the main reinforcement area along the short span per 1 m in slab width shall be arranged per 1 m in length along the short side of slabs.

For continuous or fixed end slabs, the reinforcement not less than 1/4 the main reinforcement per 1 m in slab width shall be placed normal to the short span direction per 1 m in length along the short side at the top side of the slab. The length of this reinforcement shall be equal to or greater than that of the reinforcement located at the top of the slab in the direction normal to the long side.
(4) Precausion reinforcement shall be placed on both top and bottom sides of slab corners when a two-way slab is not monolithically produced with walls and beams sideways, or does not extend continuously beyond slab supports. This reinforcement discussed here shall be located over the range whose side is 1/5 times the long span length from corners.

The precaution reinforcement may be located at the top of slabs parallel to the diagonal line from each corner and arranged perpendicular to the diagonal along the bottom face of slabs. The reinforcement stipulated here may be placed parallel to both sides of slabs respectively. The area of reinforcement per 1 m width in each direction shall be equal to the area of positive moment reinforcement along the short span (See Fig.18.3.4).

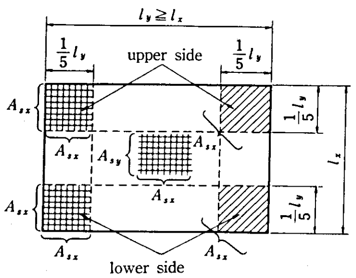


Fig. 18.3.4 Minimum reinforcement at the corners of two-way slab

18.3.9 Skewed slabs
(1) For skewed slabs, member forces shall be determined taking account of the degree of the slant angle and the slab width.
(2) For skewed slabs on two simple supports, inclined with the angle greater than 45 deg from support edges, the following simplified methods may be used for computing member forces.

(i) Positive bending moment shall be obtained based on the slab width and the ratio of the slab width to the diagonal span length, b/l_i, as follows (See Fig.18.3.5).

(a) When $b/l_i \leqq 0.75$,
the positive bending moment shall be evaluated based on a one-way slab by assuming the diagonal span length in computation.

(b) When $b/l_i > 0.75$,
for computing the positive bending moment, structural system shall be one-way slab whose span shall be specified as the straight one between

supports.

(ii) Negative bending moment, which is the product of the positive bending moment at the slab center and the coefficient α corresponding to the angle in Table 18.3.1, may be assumed to act at obtuse corners along both directions parallel to the diagonal and the support sides.

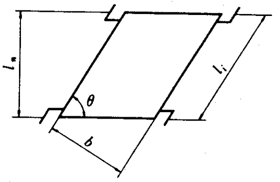


Fig. 18.3.5 Skewed slab

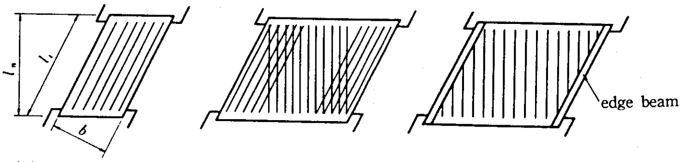
Table 18.3.1 Coefficient α for bending moment

slant angle	coefficient α
$90^\circ > \theta \geqq 75^\circ$	-0.6
$75^\circ > \theta \geqq 60^\circ$	-0.8
$60^\circ > \theta \geqq 45^\circ$	-1.0

(3) In the cases of (1) and (2), the primary distribution and precaution reinforcement shall be provided as follows.

(i) When design is in accordance with (2)(i)(a), the positive moment reinforcement shall be arranged in the diagonal span direction.

When the provision (2)(i)(b) is used, the positive moment reinforcement shall be located at the center of the slab along the direction of the normal span, and at the both sides of the slab, shall be arranged along the diagonal span according to the degree of the slant angle and the slab width, or in the normal span direction in providing edge beams (cf Fig.18.3.6).



(a) $b/l_s \leqq 0.75$ (b) $b/l_s > 0.75$

Fig. 18.3.6 Placing of primary reinforcement for skewed slabs

(ii) In the case of (2)(i)(a), distribution reinforcement not less than 1/4 the positive reinforcement shall be placed in the direction normal to the positive moment reinforcement or parallel to the support.

For the design due to (2)(i)(b), the distribution reinforcement not less than 1/3 the positive reinforcement shall be provided.

(iii) At the top of the obtuse corners in the support area of skewed slabs, the precaution reinforcement required by (2)(ii) shall be provided. In this case, the precaution reinforcement in each direction parallel to the diagonal span and the support shall be arranged in the parallelogram with the side length 1/5 the diagonal span length (See Fig.18.3.7).

The reinforcement located at the top of the obtuse corners may be considered as a part of the distribution reinforcement computed due to (2)(i)(a).

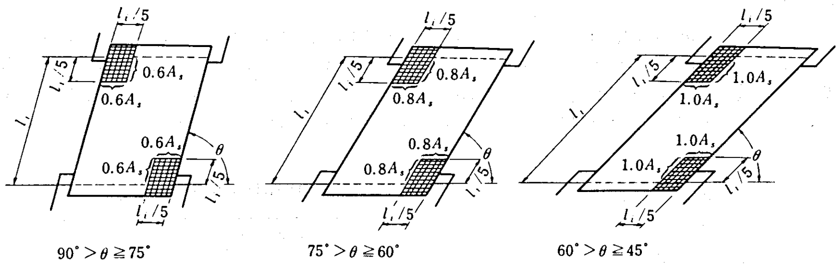


Fig. 18.3.7 Placing of minimum reinforcement for skewed slabs

18.3.10 Circular slabs

It is a principle that the reinforcement in the radial and hoop directions of primary bending moment shall be placed.

18.3.11 Structural details

(1) The minimum slab thickness shall be 8cm.
(2) The spacing between the centers of positive and negative reinforcements of slabs shall not exceed 30cm or not be greater than two times the slab thickness at the section where the maximum bending moment acts.
(3) When slabs have openings, it shall be checked that the slabs possess the sufficient strength and satisfy serviceability taking account of effects of openings.
(4) Where the negative bending moment may act on the simple supprt of slabs, the reinforcement shall be provided for it.

18.4 Beams

18.4.1 General

(1) Member forces of beams are as a rule computed by linear analyses taking accout of structural systems, load conditions and so on.
(2) The flexural, shear and torsional stiffnesses of member cross section, in which linear analyses are used for determining member forces, may be computed based on the gross concrete cross section in general.
(3) For check of ultimate limit states, strengths of member cross section, in which the design values of axial loads do not exceed 0.1 $f'_{cd}bh$, may be determined in accordance with flexural members neglecting the effects of axial loads.
(4) The tensile reinforcement ratio of the member cross section designed as a flexural member p or the difference between the tensile and compressive reinforcement ratios ($p-p'$) shall not exceed 75% of the balanced strain reinforcement ratio.

18.4.2 Span length

(1) The span length used in computation of a simplly supported beam shall be the distance between the centers of supports. In the case of wide supports, the sum

of the clear span and the overall beam depth at the center of the span shall be utilized in computaion.

(2) The clear span may be used for computation where the beam is considered to act monolithically with adjacent rigid walls and beams.

(3) The span for a continuous beam shall be the distance between the centers of supports.

18.4.3 Effective flange widths of T beams

(1) The effective frange widths of T beams used in computation of member forces may be in accordance with the following equations in general.

(a) For a beam with slabs on both sides, (See Fig.18.4.1(a))

$$b_e = b_w + 2(b_s + l/8)$$

where, b_e shall not exceed the distance between the center lines of slabs located on both sides of the beam.

(b) For a beam with a slab on a single side, (See Fig.18.4.1(b))

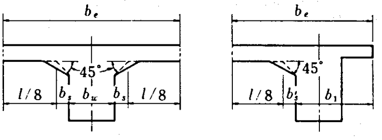
$$b_e = b_1 + b_s + l/8)$$

where, b_e shall not exceed the sum of b_1 and the 1/2 the clear slab span length.

In these cases, l shall be defined as the span length for a simply supported beam, distance between moment inflection points for a continuous beam and two times the clear span length for a cantilever beam. The value shall not be greater than the depth of haunchs.

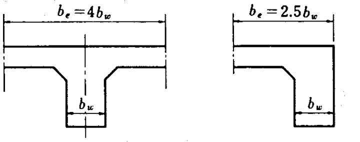
(2) For computaion of redundunt forces, the effective compressive and tensile frange widths of T beams may be the value in accordance with the provision (1) in general.

(3) The effective tensile flange width may be four times the web width for the beam with slabs on both sides (See Fig.18.4.2(a)), and may be 2.5 times in the case of a single slab connected (See Fig.18.4.2(b)). The tensile reinforcement located in slabs shall be enveloped by the ties equivalent to web reinforcement (in preparation).



(a) with slabs on both sides (b) with a slab on a single side

Fig. 18.4.1 Effective flange widths of T beams



(a) with slabs on both sides (b) with a slab on a single side

Fig. 18.4.2 Effective tensile flange widths

18.4.4 Isolated beams

Isolated beams, in which special analyses for stability in the lateral direction are omitted, shall satisfy the following requirements.

(1) An isolated rectangular beam shall be supported in the lateral direction with spacing not greater than 15 times the beam width.

(2) The support whose spacing dose not exceed 25 times the width of an isolated T beam shall be provided in the lateral direction.

(3) The flange thickness of an isolated T beam shall not be less than one half the web width.

(4) The effective flange width of an isolated T beam shall be determined not to be greater than 4 times the web width.

18.4.5 Continuous beams

(1) On check for ultimate limit states, the bending moment acting on supports of a continuous beam due to the linear analysis may be adjusted in accordance with the provision 6.3(3). The tensile reinforcement ratio p or the differecne between the tensile and compressive reinforcement ratios ($p-p'$) shall not exceed 50% of the balanced strain reinforcement ratio.

(2) Regardless of moment redistributions, the negative bending moment at the support, in which reaction act widely, may be reduced in accordance with the following equation. (See Fig.18.4.3)

$$M_d=M_{od}-rv^2/8$$

but, $M_d>0.9\,M_{od}$

where, M_d : design value of reduced bending moment on the middle supports

M_{od} : design value of bending moment on the middle supports due to loads considerd

$r=Q_d/v$

R_{od} : design value of reaction on the middle supports due to loads considerd

v : assumed longitudinal width of load distribution area in the centroid of cross section.

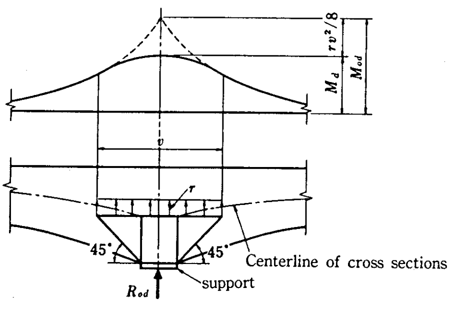


Fig. 18.4.3 Design value for bending moment on the middle support

18.4.6 Deep beams

(1) Scope

Beams resisting in-plane loads with span to overall deapth ratios not greater than the following value shall be designed as deep beams in accordance with this section.

(a) simply supported beams $l/h<2.0$

(b) continuous beams with two spans $l/h<2.5$

(c) continuous beams with three or more spans $l/h<3.0$

where l : span length of beam

h : overall deapth of beam

(2) Check for flexure

For the maximum moment analyzed by an ordinary beam theory, the main tensile reinforcement may be computed in accrdance with Section 7.3 or Section 7.4. The longitudinal reinforcement located from the extreme tension fiber of cross section to 0.2h shall be considered as the main tensile reinforcement. But, this specified reinforcement shall be developed to meet the requirement of this section (4)(iii).

(3) Check for shear

Check for shear may be made in ultimate limit states in general by ensuring that the ratio of the design value for the shear strength according to Section 8.3.3(2) to the design value for the shear force factored by Eq.(18.4.1) is greater than the structure factor γ_i .

$$V_d = V_{od}/\beta \qquad (18.4.1)$$

where V_d : design value for shear force of deep beam

V_{od} : design value for shear strength of deep beam due to load considered

β : modification factor for shear force

$\beta = 4(1+1/\sqrt{100\,p_w})/[1+(0.5\,l/d)^2]$

p_w : ratio of the longitudinal tensile reinforcement area to the web cross sectional one

(4) Structural details

(i) Widths of deep beams shall be equal to or greater than 10cm.

(ii) In the side faces of deep beams, minimum horizontal and vertical reinforcement not less than 0.08% of the concrete cross sectional area in each side shall be provided not less than two times of the beam width and not less than 30cm.

(iii) The main tensile reinforcement of simply supported beams shall extend from support to support cntinuously and tension forces greater than the maximum value in reinforcement shall be developed at the support portions.

Positive and negative moment reinforcement of continuous beams shall be placed all over the beam length. Lap splices of reinforing bars placed in the bottom layer may be located in the middle supports, but the lap splices on the simple supports shall be designed in accordance with the design for simply supported beams. Minimum horizontal reinforcement shall be spaced one half the required value in (ii) over the range 0.4 times of the span length from the middle support to both sides.

(iv) For indirectly loaded deep beams, or loaded from the flexural tension zone in cross section, the sufficient reinforcement for suspension enough to transfer the loads to overall deep beams shall be arranged.

(v) Details for support portions shall be designed in accordance with Section 18.4.7(5)(iii).

18.4.7 Corbels

(1) Scope

The provisions of this section for corbels shall apply for design of cantilever beams with the ratios of the distance between column face and loading portion to the overall beam depth and subjected to in-plane loads (See Fig.18.4.4).

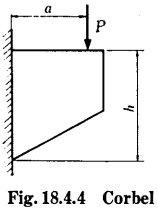


Fig. 18.4.4 Corbel

(2) Analysis

Frame system consisting of the horizontal tension member and the diagonal compression strut shown in Fig.18.4.5 may apply for analysis of corbels.

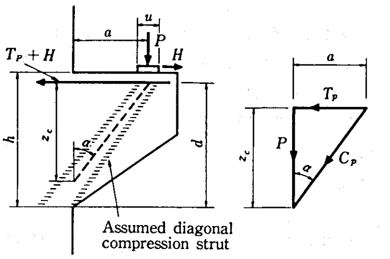


Fig. 18.4.5 Analytical model for corbels

(3) Check for flexure

The main tensile reinforcement shall be provided in accordance with the tension force computed in (2).

(4) Check for shear

Check for shear shall be made by ensuring the ratio of V_{ccd} due to Eq.(18.4.2) to the concentrated load P to stand above the structure factor γ_i . Generaly γ_b may be 1.15.

$$V_{ccd}=f_{vd}(1+\sqrt{100p_w})b_w d/[1+(a/z_c)^2]/\gamma_b \qquad (18.4.2)$$

where $f_{vd}=f_{vk}/\gamma_c$

$f_{vk}=0.6f'_{ck}$

- f'_{ck} : characteristic value for compressive strength of concrete
- γ_c : material factor of concrete
- γ_b : member factor
- p_w : ratio of longitudinal tensile reinforcement area to web cross sectional one
- a : distance between column face and loading point
- b_w : web width, where $b_w \leqq$ (lateral width of bearing plate)
- d : effective depth of corbel at the column face
- z_c : height of main reinforcement measured from the junction of compressive resultant line and column face (See Fig.18.4.5) $=\beta_d \cdot d$

  $\beta_d=0.6+0.5a/d\leqq 0.95$

(5) Structural details

(i) The effective depth of the corbel just below the loaded portion shall

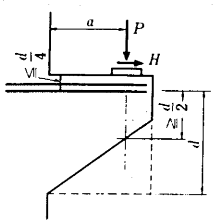


Fig. 18.4.6 Effective depth and primary tensile reinforcement below the loading position of corbel

not be less than one half of the effective depth at the support edge.
(ii) Where the main tensile reinforcement is arranged in two or more layers, bars shall be placed within d/4 from the upper surface of corbels.
(iii) The main reinforcement shall be anchored in support members where bars shall be horizontally bent. The rudius of bend measured on the inside of the bar shall be than 10ϕ . The distance from the center of the loading point to the loop inside of the main tensile reinforcement shall be required greater than 13ϕ . The clear distance from the bearing plate edge to the loop inside shall be required greater than 2ϕ and concrete cover.

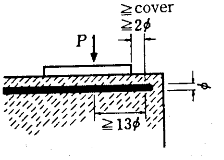


Fig. 18.4.7 Development of primary tensile reinforcement

(iv) In the side faces of corbels, precaution reinforcement shall be arranged equal to or greater than 40% of main tensile reinforcement and spaces between precaution reinforcement shall be less than or equal to 30cm.

18.4.8 Structural details

(1) The cross sectional area of the main longitudinal reinforcement shall be more than or equal to $0.002 b_w d$. The area may be reduced to $0.0015 b_w d$ when the longitudinal tensile reinforcement is provided greater than 4/3 of longitudinal reinforcement necessary in computation.

The longitudinal steel cross sectional area for prestressed concrete beams shall be greater than or equal to $0.0015 b_w d$.

(2) For beams, stirrups greater than or equal to 6mm in diameter shall be arranged within the spaces of one half of the effective depth and web width. The area of stirrups shall be greater than or equal to $0.0015 b_w s \sin\theta$.

Where compression reinforcement is provided, the stirrup spacing shall not exceed 15 times the diameter of the compression reinforcing bar and 48 times the diameter of the stirrup.

(3) The flange thickness of T-beams and top and bottom slab thicknesses of box beams shall be greater than or equal to 8cm.

(4) Web widths of T-beams and box-beams shall be greater than or equal to 10cm.

(5) Horizontal precaution reinforcement shall be arranged when beams are deep.

(6) Precaution reinforcement shall be arranged taking account of web cracking in the vicinity of supports.

18.5 Columns

18.5.1 General

(1) Design for columns shall be based on axial loads, moments and shear forces determined from structural analyses taking account of shapes and stiffness of

members, stiffness ratio of columns to members connecting with columns, structures of connections and load conditions, etc.
(2) Columns whose ratios of the effective length to the radius of gyration (slenderness ratio) do not exceed 35, may be designed in negrecting lateral displacements of columns.
(3) Structural analyses for columns whose slenderness ratios are greater than orequal to 35 shall be executed with the consideration of the effects of lateral displacements. The secondary moment due to lateral displacements shall be analyzed taking account of the effects of slenderness displacement ratio, geometry of cross section, kinds of loads, boundary conditions, material properties, arrangemnt and quantity of reinforcement, creep and drying shrinkage.

18.5.2 Slenderness ratio
(1) The effective length of columns should be determined taking account of degree of fixations in both ends of columns.
For columns braced at the end sideways, the effective length of column should be taken the length of center line of structures.
For fixed end columns with a free end in the other side, the effective length of columns shall be twice the length of center line of structures.
(2) The radius of gyration may be computed using gross cross section of concrete.

18.5.3 Tied reinforced columns
(1) The minimum lateral dimension of tied reinforced concrete columns shall be greater than or equal to 20cm.
(2) The diameter of longitudinal reinforcement shall be greater than or equal to 13 mm. The number of longitudinal reinforcement shall be more than or equal to 4. The cross sectional area of longitudinal reinforcement shall be greater than or equal to 0.8 % and smaller than or equal to 6 % of required concrete cross sectional area.
(3) The diameter of tie reinforcing bars shall be greater than or equal to 6 mm and the spaces between tie bars shall not exceed the minimum lateral dimension of columns, 12 times of the diameter of the longitudinal reinforcement and 48 times of the diameter of tie bars. Sufficient ties shall be provided especially in the connections with beams or other members (See Fig.18.5.1).

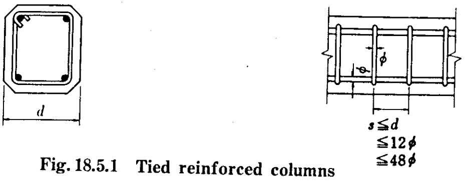


Fig. 18.5.1 Tied reinforced columns

18.5.4 Spirally reinforced columns
(1) The design value for strength of spirally reinforced columns, N_{spd} which are subjected to only axial forces may be determined in accordance with Eq.(18.5.1). γ_b may be 1.15 in general.

$$N_{spd}=(0.85 f'_{cd} A_c + f'_{yd} A_{st} + 2.5 f_{pyd} A_{spe})/\gamma_b \qquad (18.5.1)$$

where

- A_c : effective concrete area of spirally reinforced column
- A_{st} : gross area of longitudinal reinforcement
- A_{spe} : equivalent area of spirals ($=\pi d_{sp} A_{sp}/s$)
- d_{sp} . cross sectional area of spiral reinforcement
- A_{sp} : diameter of effective cross section of spirally reinforced column
- s : spaces between spirals
- f'_{cd} : design value for compressive strength of concrete
- f'_{yd} : design value for compressive yield strength of longitudinal reinforcement
- f_{pyd} : design value for tensile yield strength of spiral reinforcement
- γ_b : member factor

(2) The characteristic value for compressive strength of concrete for spiral columns shall be greater than or equal to 200 kg/cm^2

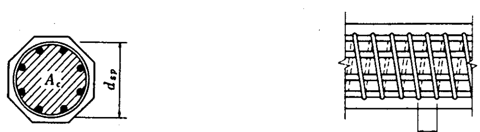
(3) The diameter of effective cross section for spiral columns shall be greater than or equal to 20 cm, where the diameter of effective section is such that of the center line of spiral.

(4) The diameter of longitudinal reinforcement shall be greater than or equal to 13 mm. The number of longitudinal reinforcement shall be more than or equal to 6. The cross sectional area of longitudinal reinforcement shall be greater than or equal to 1 % and smaller than or equal to 6 % of the effective area of column and shall be greater than or equal to 1/3 of equivalent cross sectional area of spiral reinforcement.

(5) The diameter of spiral reinforcement shall be greater than or equal to 6mm and the pich of spirals shall not exceed 1/5 of the diameter of the effective cross section and not exceed 8 cm (See Fig.18.5.2).

Equivalent area of spirals shall not exceed 3% of the effective cross sectional area of column.

For connection part of column with beams or other members, spiral reinforcement shall be provided sufficiently. Spiral reinforcement shall be anchored by winding with excessive one and a half turns of spirals.



A_c = effective cross sectional area of spirally reinforced column

$s \leq \frac{1}{5} d_{sp}$
≤ 8cm

Fig. 18.5.2 Spiral columns

18.5.5 Splices of reinforcement

(1) Longitudinal reinforcement shall be spliced by mechanical connections, sleeve joints, pressure welding and welded splices, or lap splices where the number of splices at any section shall not exceed one half of the longitudinal reinforcing bars.

(2) For lap splices of spirals, the lap splice length shall be required one and a half turns or further.

18.6 Walls

18.6.1 General

(1) Walls may be designed in accordance with columns, slabs and beams taking account of the geometry and directions of loads.
(2) Walls subjected to vertical loads may be designed in accordance with the assumption that walls are equivalent to columns whose cross sections are consisted of the one side equal to length of the wall thickness and the other equal to length of the load distribution width plus four times of the wall thickness.

18.6.2 Shear walls

Shear walls subjected to in-plane horizontal loads may be design in accordance with Chapter 15 and Section 8.5.

18.6.3 Structural details

(1) Wall thickness shall be greater than or equal to 10cm in general, and greater than or equal to 1/25 of the length of unsupported side of walls. When displacement of all over sides of walls is fixed, wall thickness shall be greater than or equal to 1/30 of shorter side.
(2) The area of vertical reinforcement of walls subjected to vertical loads shall not be less than 0.4% and not exceed 4% of the concrete sectional area.
(3) The vertical and horizontal reinforcement of shear walls shall be provided greater than or equal to 0.15% of the concrete areas normal to the reinforcing directions, respectively.
(4) The diameter of vertical reinforcing bars shall be greater than or equal to 13mm and the spaces between reinforcements shall be smaller than or equal to 2 times of wall thickness and 30 cm. The vertical reinforcement located in both sides of walls shall be mutually tied by reinforcing bars.
(5) Horizontal reinforcement shall be designed in accordance with beams, columns and slabs.

18.7 Frames

18.7.1 Structural analyses

(1) The structures with beams and slabs monolithically built with columns and walls respectively shall be analyzed as frames.
(2) Member forces should be in general computed taking account of the flexural deformations only.

However, when the ratios of member thickness to member length are equal to or greater than 0.3, frame analyses shall be made in taking account of the rigid areas at corners and flexural and shear deformations of members.

(3) The rigid areas may be in general determined as follows.

a) When the beam end (or column) is fixedly connected with the column (or beam), the interior portion of the column (or beam) apart from 1/4 times the beam (or column) height from the column face shall be defined as rigid area (See Fig.18.7.1(a)).

b) When members have haunches inclining greater than or equal to 25 degrees with the member center line, the rigid areas shall be the portion between the intersection of member center lines and the cross section where the haunch depth is equal to 1.5 times of member depth. However, in case of the haunch with a slant angle greater than or equal to 60 degrees, the interior section apart from 1/4 times of member depth from the begining portion of the haunch shall be the edge of the rigid area (See Fig.18.7.1(b)).
c) When two or more edge sections corresponding to the rigid area mentioned above are determined because of the difference of right and left side haunches, the larger rigid area shall be selected.

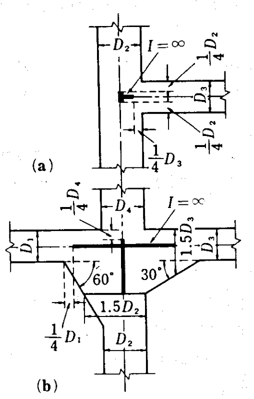


Fig.18.7.1 Rigid areas

18.7.2 Center lines

Center lines shall be such that joining centroids of cross sections. The effects of haunches may be in general neglected other than extremely larger haunches.

Lengths of beam and column center lines may be respectively the distances between column center lines and between beam ones in general. For fixed end frames, the length of column center line may be the distance from the beam center line to the face of footings, and in the case of higed end frame, to the center of the hinge.

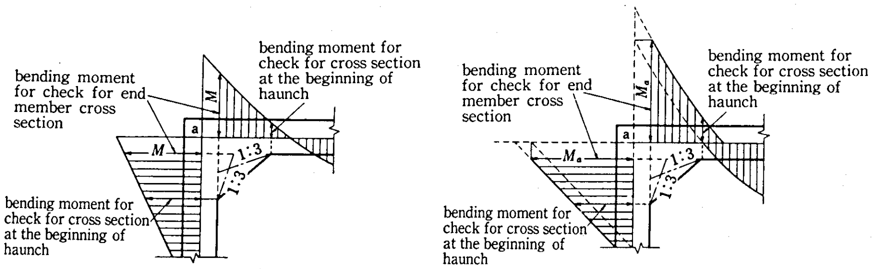
18.7.3 Check for cross sections located at the end of members

(1) The value of bending moment for check for cross section located at the end of members shall be the moment at the column face for beams and the moment at the top or bottom face of the beam for column (See Fig.18.7.2(a)).

However, when the structural analyses are executed neglecting the effects of haunches, nodal bending moment shall be adopted (See Fig.18.7.2(b)). The value of bending moment at the biginning of the haunch shall be determined by shifting moment diagrams as shown in Fig.18.7.2(b).

(2) The value of shear force for check for the end cross section shall be shear force at the top or bottom face of beams for columns. For beams, shear force may be reduced in accordance with Section 8.3.5. In this case, V_{hd} shall be equal to 0 and x shall be the distance from the column center line and d shall be taken as the effective depth of the cross section located at the beginning of

the haunch (in preparation).



(a) in the case of analysis taking account of haunches

(b) in the case of analysis neglecting the effective of haunches

Fig. 18.7.2 Bending moment for check for end member cross sections

18.7.4 Structural details

(1) It is a principle that haunches should be provided at the corners of frames.

(2) The reinforcement of corners shall be arranged taking account of construction joints.

(3) The precaution reinforcement should be provided for vertical cracking due to deformations at errection in construction stage and the effect of vertical reactions from columns.

(4) The precaution reinforcement shall be provided in the bottom side of haunchs. The area of precaution reinforcement should be greater than or equal to one half of the reinforcement placed in the lower side of beams.

(5) Tie reinforcement shall be densely spaced in the vicinity of beam-column intersections. The tie reinforcement not less than 0.25% of the concrete cross sectional area shall be provided in columns over two times of the column width from the lower side of the beam haunch in the beam-column connection and the spaces between tie bars shall be smaller than or equal to 100mm. The stirrups not less than 0.20% of the concrete area and 1.2 times of the calculated value shall be provided in the range 1.5 times of the beam overall depth from the connections with beams (See Fig.18.7.3).

(6) At the corners of beam-column connections, diagonal reinforcing bars and stirrups should be placed as shown in Fig.18.7.4.

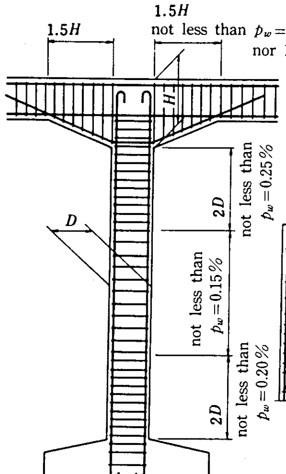


Fig. 18.7.3 Placing of reinforcement at the middle node of frame system

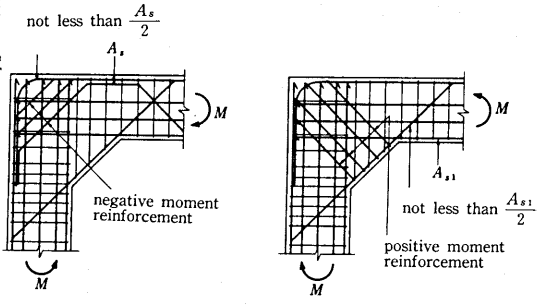


Fig. 18.7.4 Placing of reinforcement at the corners

18.8 Arches

18.8.1 General

(1) It is recommended that the arch center line should coincide with the line of thrust.
(2) Cross sectional shapes of arch ribs shall be selected taking account of the rise-to-span ratio, arch center line, strength for buckling, compressive strength of concrete and construction systems.
(3) Abutments of arch ribs shall be solid enough to resist the reactions at the end of arch ribs.

18.8.2 Structural analyses

(1) Arch center line shall be the line joining the centroids of arch rib cross sections.
(2) In computing member forces, the effects of drying shrinkage of concrete and variations of temperature shall be taken into account.
(3) The variations of arch rib sections shall be considered in calculating redundant forces.
(4) In the case of evaluations of member forces for long-spanned structures, account shall be taken of the effect of the shifted arch center line as a rule.
(5) When there is the possibility of the base to move, its effects shall be taken into account.

18.8.3 Check for buckling

(1) For design of arches, it shall be ensured that arches are safe against the buckling in the plane including the arch rib and in the lateral direction.
(2) Check for in-plane buckling may be made based on the equivalent buckling length of arches in accordance with Section 18.5.1. (in preparation)
(3) Check for lateral buckling may be executed in accordance with Section 18.5.1 assuming arch ribs as straight columns subject to the axial forces equal to the horizontal reactions at the end of the arch ribs. This column length shall be equal to the arch span as a general rule.

18.8.4 Structural details

(1) The longitudinal reinforcement shall be symmetrically arranged along the top and bottom faces of arch ribs. The area of reinforcement shall not be less than 6cm per 1m in arch width and the sum of the reinforcement located at the top and bottom faces shall be 0.15% of the concrete cross sectional area or greater at the same time.
(2) The transverse reinforcement shall be placed so as to encase the longitudinal reinforcement over the arch. The transverse reinforcement not less than 13mm in diameter nor 1/3 the diameter of longitudinal reinforcing bars shall be spaced at the center-to-center distance not greater than 15 times the diameter of longitudinal reinforcing bars nor the minimum dimension of arch rib cross sections.

For filled spandrel arch bridges, the transverse reinforcement shall be arranged in order to resist the bending moment due to the side walls.
(3) Expansion joints shall be provided for the side walls of filled spandrel

arch bridges.

The expansion joints shall be the structures to prevent water leakage, so that they shall be suitably located on the springing in the case of short span or on the arch crown in the case of relatively long span.

(4) In the case of box cross sectional arch ribs diaphragms shall be provided at the portions joining columns (See Fig.18.8.1).

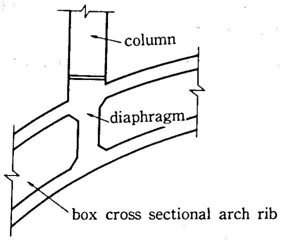


Fig. 18.8.1 Diaphragm of box cross sectional arch rib

18.9 Thin shells

18.9.1 Structural analyses

(1) Member forces shall be computed based on the force equilibrium condition for shell elements and the compatibility condition for deformations in which the displacement between shell elements shall be continuous.

However, within the range where the behavior of thin shells is assumed elastic in three dimensional conditions, analytical models may be simplified in mathematical and structural point of views.

(2) Poisson's ratio may be set equal to 0.2 in computation.

18.9.2 Check for buckling

Thin shells shall have sufficient safety against buckling as a rule. (Checking system is in preparation.)

18.9.3 Structural details

(1) Thin shell thickness shall be equal to or greater than 8cm.

(2) It is a principle that edge members more rigid than shell members shall be provided around shells, and that supporting members shall be located in the middle part of shells subjected to concentrated loads.

(3) The effective width of shells which act with supporting members or edge members shall be in accordance with Section 18.4.3.

(4) The spacing of longitudinal reinforcement shall not exceed 30cm. The area of longitudinal reinforcement shall not be less than 0.3% nor exceed 5% of the concrete area. The area of the reinforcement placed normal to the longitudinal reinforcement shall not be less than 0.3% nor exceed 5% of the concrete area, and shall be 1/4 times the longitudinal reinforcenent at the same time.

(5) In the range on which great tensile forces act, the direction of the longitudinal reinforcement shall coincide with that of the principal stress introduced by permanent loads as much as possible.

(6) The longitudinal reinforcement of shell members located at the connections with supporting and edge members shall extend to the supporting and edge members and anchored.
(7) Splices of reinforcement shall not be placed in the portion where shell members come into contact with supporting or edge members.
(8) Sufficient reinforcement shall be provided around openings.

18.10 Flat slabs

18.10.1 Definitions and notations

(1) Definitions

column line : line to join the centers of columns
column strip : strip with width of 0.25 l_x or 0.25 l_y width from column line
middle strip : strip with width of 0.5 l_x or 0.5 l_y in the middle of column strips
torsional member : slab enveloped by two parallel lines in the l_y direction which are tangents of column faces

(2) Notations

l_x : span length in the direction of the stress due to momemt
l_y : span length in the direction normal to l_x

18.10.2 Scope

This section's provisions shall apply to designs of slabs in flexure, columns and slab-column connections of flat slabs supported directly by columns or through drop panels.

18.10.3 Design for slabs

Design for flat slabs must be made by ensuring the strength required in all sections against the member force distributions which satisfy the equilibrium and compatibility conditions and the sufficient functions in service stage including deflections, or by design methods in which the safety is guaranteed by actual results.

18.10.4 Equivalent rigid frame method for design

(1) For flat slab structures, member forces may be computed assuming the interior potions of structures, bounded by the adjacent column lines or center lines of middle strips, as the equivalent rigid frames which represent the behaviors of flat slabs.
(2) When the equivalent rigid frame method is used for design of flat slabs, check shall be made in each orthogonal direction in the slab plane.
(3) For the equivalent rigid frame method, the stiffnesses of columns and slabs shall be determined based on the actual cross sectional stiffnesses. In determining the stiffness of connections, the conditions that columns are connected with slabs through torsional members shall be taken into consideration.
(4) The connection between columns and slabs through torsional members may be idealized as the condition that columns and beams with the same stiffnesses as slabs' are connected by the springs representing the torsional stiffness (See

Fig.18.10.1).

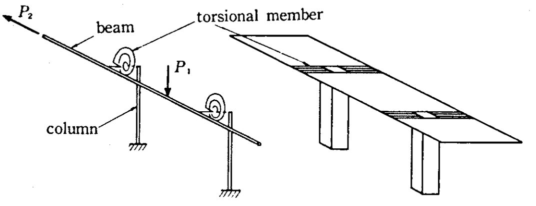


Fig. 18.10.1 Equivalent frame analytical model

(5) The design value for slab moment may be obtained by proportioning beam moment computed in the equivalent rigid frames to column and middle strips by an adequate method.
(6) The following approximation methods may be utilized in analyzing two-way reinforced flat slabs. In this case, structural details in accordance with Section 18.10.6 shall be required.
1) For vertical loads,
(a) Slabs shall be considered as two sets of beams bounded by orthogonal column lines of x and y directions, and in each direction, total loads shall be provided assumed rigid frames, which consist of consist of columns and these beams, so as to cause the most severe condition. Span length of these frames in x direction and beam width are defined as l_x and l_y . The beam depth shall be the slab thickness, t (See Fig.18.10.2). In y direction the same things shall be defined in the similar ways.

For multi-layer rigid frames, only flexural resistance of columns attaching the slabs considered may be taken into account.

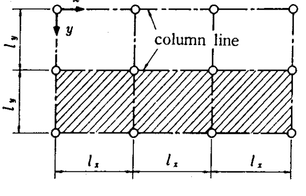


Fig. 18.10.2 Frames of flat slab structures

(b) Positive or negative span bending moments, which are computed by assuming that a slab consists of the middle strip ABDC with l /2 in width and column strips ABFE and CDHG with l /4 in width, shall be uniformly distributed in the proportion of 45% and 55% on the middle strip and column strips on both sides respectively.

Twenty five percent of the negative bending moment shall be uniformly distributed on the middle strip and 75% on the column strips on both sides respectively (See Fig.18.10.3).

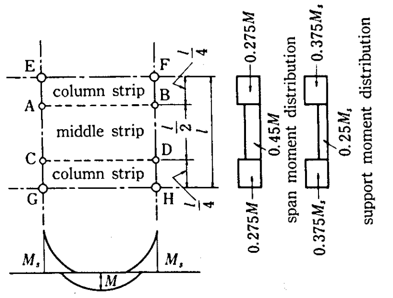


Fig. 18.10.3 Bending moment distribution

When the overall edges of flat slabs are supported, 3/4 times the bending moment of the column strip may be used in general for the strips with 3/4 l in width from the supported edges.

(c) Columns shall be treated as the vertical members of rigid frames.

2) For horizontal loads, the equivalent frames discussed above shall be assumed. However,

(a) the beam width of the rigid frames shall be in accordance with Eq.(18.10.1).

$$b=\frac{1}{2}(l+c) \qquad (18.10.1)$$

where l : span length in each direction

c : side length of column capital (diameter in the case of circular one)

(b) The computed bending moment of beams shall be distributed in proportion of 0.7 and 0.3 on the column strip and on the middle strip respectively.

18.10.5 Design for slab-column connections

(1) Check for punching shear failure shall be made in the slab-column connections of flat slabs.

(2) In case of the slab-column connections where moment is transferred between slabs and columns besides punching shear force, the proportion of bending moment, shear force and torsional moment, which are components of the transferred moment, shall be correctly evaluated at the face of connections and proper design shall be made for this moment transfer.

18.10.6 Structural details

(1) The slab thickness shall not be less than 15cm in general. However, this provision shall not apply to special slabs such as roof slabs.

The ratio of the effective depth of slab to the larger span shall not be less than 1/32, and 1/40 for roof slabs.

(2) Column width shall be equal to or greater than 1/20 of the span length l in the same direction as that of the width, 1/15 of the height of each story h_s and 30cm, where l is the distance between centers of columns (See Fig.18.10.4).

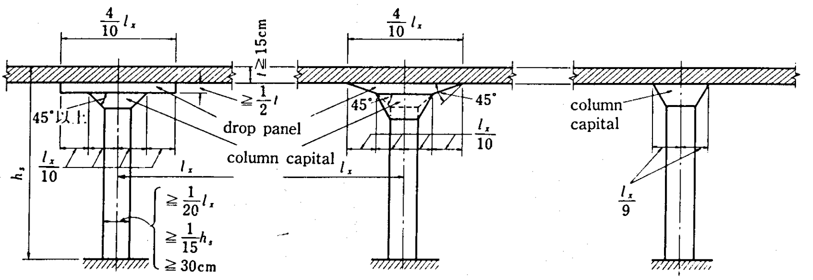


Fig. 18.10.4 Flat slabs

(3) Dimensions of a column head shall be in accordance with Fig.18.10.4. Stress analyses shall be made by assuming that the upper portion of the column capital from the line making 45 degrees with the horizon is effective.
(4) The reinforcement for the frexural component of moment which are transferred to columns shall be placed within a range equal to the column width plus lengths of 1.5 times of the slab thickness on its both sides.
(5) When the moment is transferred to columns, reinforcement shall be placed in the bottom of slabs in the vicinity of column heads. If the reinforcement necessary may not be evaluated by appropreate analyses, the reinforcement not less than one-half the area of reinforcement placed in the top of slabs shall be provided. The range placing shall be according to (4).

19 DESIGN OF GROUNDED STRUCTURES

19.1 General

19.1.1 Scope

This section shall be applied to base structures, underground structures, etc. in which mutual actions between the structures and the ground shall be considered mainly.

19.1.2 Bases of design

(1) Design criteria of a structure must be determined properly according to a type of a structure, a location of construction, and a soil condition, and the structure must be designed to satisfy purposes for use and to hold necessary safety.
(2) It is a principle that a soil condition should be determined by in-situ and in-room tests.

19.1.3 Limit states

(1) Check of ultimate limit states for rigid body motion stability and failure

f member cross sections and serviceability limit states for crack widths and deflections shall be made in general.
(2) Check of an ultimate limit state for rigid body motion stability shall be made with respect to sliding, falling down, and bearing force of ground.

For some kinds of lands, geological conditions, and soil conditions of a location of a structure and some sizes of a structure, it must be checked that a total structure including base ground is safe.
(3) Check of an ultimate limit state for sliding shall be made by confirming that the ratio of the design value of sliding force acting along the bottom of a structure to the calculated value is equal to or greater than a structures factor, γ_i .

The design value of capacity against sliding shall be the characteristic value, evaluated with friction and cohesive forces between the bottom of the structure and the base ground and then divided by the product of a material factor, γ_m and a member factor, γ_b .

When the requirements in this section cannot be satisfied because the limitations of land conditions, etc. increase the area of the bottom of the structure, the capacity against sliding may be ensured by the following ways.

1) to consider the capacity of projections put at the bottom of the structure
2) to consider the passive soil pressure at the front face of the structure introduced by setting the footing more deeply
3) to consider the capacity of piles against horizontal forces

In a case with consideration of the passive soil pressure at the front face of the structure, a fictious ground surface in front of the structure with sufficient consideration of soil removed by washing of running water, etc. or by changing plan must be checked. In a case with consideration of the capacity of piles against horizontal forces, the capacity against sliding shall be only the capacity of piles against the horizontal forces as a rule.
(4) Check of an ultimate limit state for falling down shall be made by confirming that the ratio of 1/6 of the bottom width to the distance between the center of the bottom and the point at which the resultant of load acts is equal to or greater than a structures factor, γ_i , which may be 1.0 in general.

When considering the combination of loads including seismic load, 1/6 of the bottom width may be increased to 1/3.
(5) Check of an ultimate limit state for bearing capacity of ground shall be made by confirming that the ratio of the design value of the bearing capacity of ground touching a structure to the design value of the subgrade reaction is equal to or greater than a structures factor, γ_i .

The design value of the bearing capacity of ground in the vertical direction shall be the characteristic value of the bearing capacity of ground obtained by loading tests or formulae of static mechanics and then divided by the product of a material factor for the ground, γ_m and a member factor, γ_b .

The subgrade reaction in general may be calculated assuming that the ground is an elastic material.
(6) Check of a serviceability limit state for deflections in general may be omitted in cases of a depth of a one-way slab or a height of a beam which is equal to or greater than the values in Table 19.1.1.

Table 19.1.1 Minimum depths of a beam or a one-way slab when omitting computation of deflections

Members	Minimum depths h			
	Simple supports	Continued at one edge	Continued at both edges	Cantilever
	Members which support partition walls or another structure likely to be damaged by great deflections or do not cling to them			
Plain one-way slab	$l/20$	$l/24$	$l/28$	$l/10$
One-way slab with beams or ribs	$l/16$	$l/18.5$	$l/21$	$l/8$

(7) For check of a serviceability limit state for crack widths the environmental conditions specified in Section 10.3 shall be determined as follows in general.

a structure above the level of underground water : normal environment

a structure below the level of underground water : corrosive environment

a structure in underground water or ground having especially harmful substances : severly corrosive environment

19.1.4 Loads

(1) Design loads during service shall be combined adequately among the followings considering purposes for use of a structure, conditions of the ground at which the structure is constructed, and types of the structure.

For base structures loads from super structures must be considered. For check of a structure itself, subgrade reaction shall be considered.

1) Permanent loads
  * self weights and weights of things in a structure
  * normal soil pressures
  * static hydraulic pressures
  * buoyancy and lift
  * prestressing forces
  * effects of creep and shrinkage of concrete

2) Variable loads
  * live loads acting on ground surface or inside of a structure (including impact)
  * surcharges
  * thermal loads
  * snow loads

3) Accidental loads
  * seismic loads
    inertial forces caused by mass of a structure
    soil pressures during earthquake
    external forces caused by displacements of ground during earthquake
    dynamic liquid pressures
  * collision loads

(2) During construction the following loads other than loads in (1) must be considered.
  * weights of construction machine, etc.

* hitting forces

(3) Normal soil pressures shall be chosen properly among static, active, passive, and loose soil pressures according to kinds of structures, deepness of construction, and soil conditions. For a structure in good rock and so on, a part of them may be neglected.

(4) Seismic loads must be determined considering kinds of structures deepness of construction, and soil conditions.

If the structure is deep enough, seismic loads may be neglected in general. In a case of soft ground, however, they must be considered.

(5) Surcharges may be considered to be equivalent to dead loads or be neglected if a structure is deep enough. For a structure close to ground surface, however, they must be considered as live loads.

19.2 Footings

19.2.1 Scope

This section shall be applied to an isolated footing used for base of a structure, a wall footing, and a combined footing.

The isolated footing is built to sustain a column or a pedestal and is not combined with another.

The wall footing is a footing to distribute the loads from the wall.

The combined footing is built to sustain more than one column or pedestal or is isolated footings combined with each other by members.

19.2.2 Limit states and combinations of loads

(1) Check of ultimate limit states for sliding, falling down, bearing capacity of ground, and failure of cross section of member shall be made for the following combination of loads.

1) normal state : loads for superstructures + self weights
2) seismic state : loads for superstructures + self weights + inertial forces

(2) Check for a serviceability limit state of crack width shall be made for the following combination of loads in general.

1) normal state : loads for superstructures + self weights

(3) Check for a serviceability limit state of displacements which is required by superstructures shall be made for the following combinations of loads in general.

1) normal state : loads for superstructures + self weights
2) seismic state : loads for superstructures + self weights + inertial forces

19.2.3 Structural analysis

(1) A footing may be analyzed assuming that it is composed of cantilevers, simple beams, and continuous beams.

(2) For a footing which is deep enough to be dealt with as a rigid body, subgrade reaction due to all the loads of the footing may be assumed to be distributed linearly except for a case of more precise analysis.

A footing may be dealt with as a rigid body, in the following cases.

1) for an isolated footing and a combined footing : when the average depth of the footing is equal to or greater than about 1/5 of a longer side of the footing

2) for a wall footing : when the average depth of the footing is equal to or greater than about 1/5 of the value of a footing width subtracted by a wall thickness.

For a footing on piles a depth from a pile cap to a top of the footing at the pile shall be equal to or greater than a diameter of the pile.

(3) For a footing which may not be dealt with as a rigid body, the footing may be analyzed by assuming that it is a slab on an elastic foundation, except for a case of more precise analysis.

19.2.4 Isolated footings

(1) Check for flexure

(a) A design section of a footing sustaining a reinforced concrete column or a pedestal shall be a vertical section at a front face of the column or the pedestal.

For a section of a column or a pedestal which is not a square nor a rectangle, a design section may be a front face of a fictious square or rectanglular section which has the same area and the same centroid as the real section.

(b) A design section of a footing sustaining a steel column shall be a vertical section at the center between the front face of the column and the edge of the bottom plate. For a bottom plate with extraordinarily small stiffness, a design section is preferable to get close as much as possible.

(c) An effective width of a design section of a footing for flexural moment may be its full width when the footing is deep enough to be dealt with as a rigid body.

An effective depth of a design section of a footing which may not be dealt with as a rigid body may be obtained assuming a width of a part which may be dealt with as a rigid body.

(d) Flexural moment at a design section shall be one caused by loads acting on all the area of a footing which is in the opposite side of the column with respect to the design section (see Fig.19.2.1).

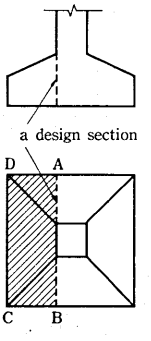


Fig. 19.2.1 A design section for flexural moment

When a design section of a two-way footing is considered, loads acting in the corner area shall be considered in both the ways.

(e) When a slope of a top surface of a footing is more gentle than 0.5, computation for flexural moment may be made neglecting the slope of the top surface.

When a slope of a top surface is steeper than 0.5, the computation must be made taking account of the slope.

(f) For a stepped footing, a section resisting flexural moment may be taken at the step lower by one than the step considered or may be taken in a fictious footing in which a slope of a top straight surface is more gentle than 0.5 and is within the actual footing (see Fig.19.2.2).

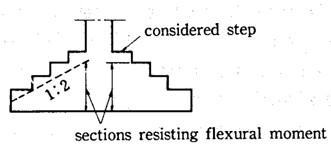


Fig. 19.2.2 Sections resisting flexural moment

(2) Check for shear

A design section for shearing force shall be a vertical section at the distance of $d/2$ from a front face of a column or a pedestal, assuming that its width is a full width of the footing there, where d is an effective depth of the footing at the front face of the column or the pedestal.

(3) Check for punching shear

(a) A design section of a footing for punching shear shall be a vertical section at the distance of $d/2$ from a front face of a column or a pedestal, where d is an effective depth of the footing at the front face of the column or the pedestal.

(b) Shearing force acting on the above design section may be computed from loads acting on area bounded by two lines starting at a corner of the column or the pedestal with the angle of 45 degrees from the main axis of the footing, the design section, and an edge side of the footing (see Fig.19.2.3).

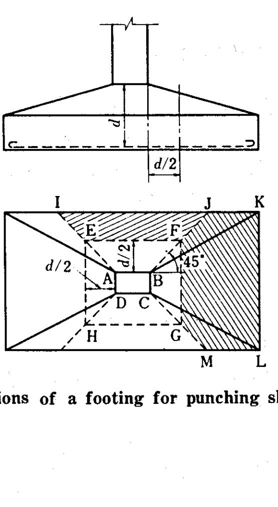


Fig.19.2.3 Design sections of a footing for punching shear and loads

(4) Check of a serviceability limit state for crack widths shall be made according to Section 10.

For check of crack widths due to flexure, an effective width of a footing may be computed by Eq.19.2.1.

$$b_e = b_0 + 2d \leqq B \qquad (19.2.1)$$

where, b_e : an effective width

B : a full width of a footing

b_0 : a width of a column or a pedestal

d : an effective depth of a design section of a footing

(5) Arrangement of reinforcement

(a) For a square footing, reinforcement in each direction shall be arranged at the equal interval over a full width of the footing.

(b) For a rectangular footing, reinforcement in the direction of a long side shall be arranged at the equal interval over a width in the direction of a short side. For reinforcement in the direction of a short side, the amount of reinforcement computed by the following equation shall be arranged at the equal interval within a band which is as long as a short side and the rest amount shall be arranged at the equal interval outside the band.

(amount of reinforcement within the band which is as long as a short side)/(total amount of reinforcement in the direction of a short side) $= 2/(r+1)$

where, r : ratio of a long side to a short side

(c) Main reinforcement in a footing is preferable to be equal to or greater than 0.15% of a cross section of concrete.

19.2.5 Wall footings

Design of a wall footing shall be made in accordance with the design of an isolated footing, assuming that the footing is an isolated footing working only in the direction normal to the wall as a cantilever.

In the direction parallel to the wall, precaution reinforcement must be arranged properly.

19.2.6 Combined footings

Design of a combined footing may be made in the same way as that of an isolated footing.

Transverse reinforcement in a combined footing shall be distributed to each column in proportion to magnitude of loads for each column and shall be arranged uniformly within a column width, b_0 plus effective depths of the footing at its both sides, d. Longitudinal reinforcement shall be arranged all over the width of the footing.

19.2.7 Design of connection between a footing or a pedestal and a column

(1) In order to transfer stresses in longitudinal reinforcement of a column to a footing or a pedestal, longitudinal tensile reinforcement must be extended into the footing or the pedestal at the column base, or reinforcement for connection must be provided there.

A length of extension of the longitudinal reinforcement or the

reinforcement for connection must be long enough to transfer the ultimate stress in reinforcement and to develop fully member forces in the column.
(2) At a cap of a footing or a pedestal, check for bearing must be done.
For a sloped or stepped footing, A_2 in Fig.19.2.4 in stead of A_1 may be taken as a gross area of concrete, A to compute the characteristic value of bearing in Section 3.3. This A_2 is a base area of a pyramid without a peak whose top area is a column area. The slope of the pyramid is 0.5 and the pyramid is included in the footing.

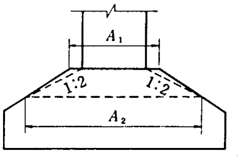


Fig. 19.2.4

19.3 Piles and caissons

19.3.1 Scope
(1) This section shall be applied to design of concrete piles and caissons used for a base of a structure.
Concrete piles are driven piles of reinforced and prestressed precast concrete and cast-in-place piles by reverse circulation drill method, all-casing, and manpower excavation.
Caissons are those in the open caisson and new matic caisson methods.
(2) The product of the stiffness evaluation value, β and the effective setting depth, l for piles is equal to or greater than 1.0 and piles may not be dealt with as a rigid body. The product, βl of caissons is less than 1.0 and caissons may be dealt with as a rigid body.
where, l : an effective depth of embedment of a pile or a caisson (cm)
β : a stiffness evaluation value of a pile or a caisson (cm^{-1})

$$\beta=\sqrt[4]{\frac{kD}{4EI}}$$

EI : flexural stiffness of a pile or a caisson (kg·cm^2)
D : a diameter or width of a pile or a caisson (cm)
k : a design value of a coefficient of horizontal subgrade reaction of a pile or a caisson (kg/cm^3)

19.3.2 Sharing of loads and arrangement of piles
(1) It is a principle that vertical loads acting on a pile foundation should be supported by piles only.
(2) It is a principle that horizontal loads acting on a pile foundation should be supported by piles only.
Piles together with a part of a footing embedded in the ground may share in supporting the horizontal loads. In this case the sharing rate of the loads shall be determined taking account of ground condition around the footing, construction method, etc.
(3) It is a principle that vertical loads acting on a caisson should be supported by only vertical subgrade reaction of ground under the caisson base.

For good ground around the caisson, its resistance may be taken into account.
(4) It is a principle that horizontal loads acting on a caisson should be supported by vertical subgrade reaction under the caisson base, horizontal subgrade reaction around the caisson, and shearing capacity of the base.
(5) It is a principle that piles should be arranged to be subjected equally to permanent loads.

When a footing may not be considered as a rigid body, arrangement of piles must be determined taking account of sharing the loads.
(6) It is a principle that the minimum center-to-center space of piles should be 2.5 times the diameter of the pile.

A distance between a center of the most outer pile and an edge of a footing shall be equal to or greater than 1.25 of a pile diameter for driven piles or precast piles by inside boaring method and 1.0 for cast-in-place piles.
(7) When a center-to-center space of piles is less than 2.5 times the pile diameter, they must be considered as a group of piles.

19.3.3 Limit states and combinations of loads
(1) Check of ultimate limit states for bearing capacity of ground and member failure shall be made for the following combinations of loads in general.

1) normal state : loads for superstructures + self weights + normal soil pressures + static hydraulic pressures
2) seismic state : loads for superstructures + self weights + inertial forces + seismic soil pressures + dynamic hydraulic pressures

(2) Check of a serviceability limit state of crack widths shall be made for the following combination of loads in general.

1) normal state : loads for superstructures + self weights + normal soil pressures + static hydrauric pressures

(3) Check of a serviceability limit state of displacements required by superstructures shall be made for the following combinations of loads in general.

1) normal state : loads for superstructures + self weights + normal soil pressures + static hydraulic pressures
2) seismic state : loads for superstructures + self weights + inertial forces + seismic soil pressures + dynamic hydraulic pressures

(4) For a caisson check of an ultimate limit state of member failure shall be made at the following times during construction other than after completing.

1) immediately after a caisson starts to be being sunk
2) when eccentricity takes place while a caisson to get sunk
3) when water is exchanged after sinking a caisson
4) when a lower part of a caisson is pulled down
5) when a cutting edge is pushed suddenly and strongly into ground

(5) Check for supporting capacity of ground in the direction normal to pile axis may not be made in general.
(6) When condensation is likely to occur, negative friction force on periphery of a pile must be taken into account.

19.3.4 Computation of subgrade reactions and displacements

Subgrade reactions and displacements of a pile or a caisson subjected to vertical and horizontal loads and flexural moment shall be computed in general assuming that ground is an elastic material.

19.3.5 Structural analysis

(1) It is a principle that a pile completely droven in ground should be designed neglecting effects of lateral displacements. However, when a length of a pile is equal to or greater than 100 times the diameter, lateral displacements must be taken into account.
(2) Flexural moments and shearing forces at each part of a pile caused by forces normal to axis and moments at a pile cap shall be obtained assuming that the pile is a beam on an elastic foundation.
(3) Each member of a caisson may be analyzed assuming that it is a slab, a beam, a column, a wall, or a rigid frame structure composed of those.

19.3.6 Structural details

(1) A standard of a precast reinforced concrete pile shall satisfy JIS A 5310.
(2) A standard of a prestressed concrete pile shall satisfy JIS A 5335 or JIS A 5336.
(3) A standard of a concrete cover of a cast-in-place shall be 15 cm.

The minimum diameter and number of longitudinal bars in a cast-in-place pile shall be 16 mm and 6. The maximum and minimum longitudinal reinforcement, and arrangement of ties or spiral reinforcement shall be in accordance with Section 18.5.3 or 18.5.4.
(4) When all the longitudinal reinforcement is lapped at the same section in a cast-in-place pile, Section 13.6 shall be satisfied.
(5) When each member of a caisson is designed as a slab, a beam, a column, or a wall, its structural details must be satisfied. When each member is designed as that in a rigid frame, structural details for a rigid frame must be satisfied.

19.4 Retaining walls

19.4.1 Space

This section shall be applied to design of retaining walls.

19.4.2 Limit states and combinations of loads

(1) Check of ultimate limit states for sliding, falling down, bearing capacity of ground, and failure of member shall be made for the following combinations of loads in general.

1) normal state : self weights + normal soil pressures + surcharges
2) seismic state : self weights + inertial forces + seismic soil pressures

(2) Check of a serviceability limit state for crack width shall be made for the following combination of loads in general.

1) normal state : self weights + normal soil pressures

(3) When effects of hydraulic pressure, buoyancy, lift, snow load, and impact are great because of a setting location, they must be considered.
(4) Self weights for check of an ultimate limit state for stability of a rigid

body movement shall include a weight of soil above a heel plate as well as weight of a retaining wall.

For computation of soil pressures, a fictious back face of a retaining wall may be assumed, and soil between the fictious back face and a vertical wall may be considered as a part of the vertical wall.

(5) For surcharges, live loads shall be taken into account. The surcharges may be computed by assuming that soil behind a retaining wall becomes higher than the actual so as to compensate the surcharges.

(6) Inertial forces of a structure may be computed by the seismic coefficient method in general.

19.4.3 Cantilever-style retaining walls

(1) Cross sections in a vertical wall may be designed in general by assuming that the vertical wall is a cantilever slab whose fixed end is a connection with a base plate.

A vertical component of a soil pressure and a self weight of a vertical wall may be neglected in general.

(2) A base plate of a toe and heel plate may be designed in the same way as a footing, assuming that the base plate is a cantilever slab whose fixed end is a connection with a vertical wall.

(3) For loads acting on a toe plate shall be considered a subgrade reaction upward and a self weight of the toe plate downward in general.

(4) For loads acting on a heel plate shall be considered a soil weight lying on the heel plate, a vertical component of a soil pressure, surcharges, a self weight of a heel plate, and a subgrade reaction.

(5) When a passive soil pressure is considered in a retaining wall with a reversed L-shape, a weight of soil under a fictious ground surface above a toe plate may be considered.

19.4.4 Counterforted retaining walls

(1) Assuming that counterforts are cantilevers with T section which are fixed to a base and whose height varies, design of sections in the counterforts may be made against horizontal loads acting on a vertical wall whose length is equal to a center-to-center space of webs of those assumed cantilevers.

Tensile reinforcement in the counterforts may be computed by assuming that total compressive stresses in concrete are acting at the center of a vertical wall.

When loads acting on both the sides of a counterfort are different, the counterfort must be designed so as to be able to resist in the direction normal to its side.

(2) A vertical wall may be designed considering it as a continuous slab supported by counterforts.

(3) A toe plate may be designed in the same way as a toe plate of a retaining wall with a reversed T-shape.

(4) A heel plate may be designed considering it as a continuous slab supported by counterforts.

(5) A vertical wall and a heel plate at edge may be designed considering them as a cantilever slab fixed to a counterfort.

19.4.5 Buttlessed-type retaining walls

(1) A buttless wall may be designed considering it as a cantilever which is fixed to a base and whose height varies.

(2) A vertical wall and a base may be designed in the same way as a counterforted retaining wall, considering them as a continuous slab supported by buttlesses.

19.4.6 Expansion joints and vertical construction joints

(1) Expansion joints of a vertical wall in general shall be placed at a space equal to or less than 10 m for a gravity retaining wall and at a space between 10 and 20 m for a cantilever-style retaining wall, a counterforted retaining wall, and a buttlessed-type retaining wall. At sections of the expansion joints reinforcement must be cut.

(2) It is a principle that a space of vertical construction joints of a vertical wall should be equal to or less than 10 m. At sections of the joints reinforcement must be continued.

19.4.7 Drainage

For a retaining wall drainage must be made in order to prevent seepage water at back from increasing soil and hydraulic pressures.

19.4.8 Structural details

(1) A slope of a front face of a vertical wall in a retaining wall with a reversed T-shape or a counterforted retaining wall is preferable to be more gentle than 50.

(2) The minimum depth of a member is preferable to be equal to or greater than 30 cm in general.

19.5 Culverts

19.5.1 Scope

This section shall be applied to design of culverts with a rectangular section constructed in the ground.

19.5.2 Limit states and combinations of loads

(1) For a culvert check of limit states may be made for a direction of a span (transverse direction) in general.

However, when a culvert is equal to or longer than 15 m or is set on soft ground, check of limit states must be made for a direction normal to span in the same way as the direction of a span. For this check the culvert may be considered as a beam on an elastic foundation.

(2) Check of an ultimate limit state of failure of member shall be made for the following combination of loads in general.

1) a vertical soil pressure + a horizontal soil pressure + live loads + surcharges

(3) Check of a serviceability limit state of crack widths shall be made for the

following combination of loads in general.

1) a vertical soil pressure + a horizontal soil pressure

(4) When effects of hydraulic pressure, buoyancy, lift, temperature, seismic loads, and loads acting inside a culvert are great because of a setting location, a purpose of use, etc., those effects must be taken into account.

(5) When subjected to partial pressures due to a construction method, etc., check of an ultimate limit state for sliding must be made taking account of loads due to the partial pressures.

(6) A vertical soil pressure acting on a top surface of a culvert, in general, may be computed assuming that it is in proportion to a depth of a soil cover.

(7) A horizontal soil pressure acting on a side of a culvert, in general, may be computed with a static soil pressure, assuming that it is in proportional to a depth from a ground surface.

(8) Vertical loads due to live loads acting on a top surface of a culvert may be transformed to a distributed load in accordance with a depth of a soil cover.

Impact may be neglected when a soil cover is deep enough.

(9) Surcharges may be considered as horizontal soil pressures action on sides of a culvert. In this case the horizontal soil pressure may be kept constant regardless of depth.

(10) Either of live loads including impact and surcharges may be considered. However, when a soil cover is deep enough, both of them shall be taken into account.

19.5.3 Structural analysis

Computation of member forces in a direction of a span may be made considering a culvert as a rigid frame.

19.5.4 Expansion joints

It is a principle that expansion joints should be placed at a space of about 10 to 15 m in a culvert.

19.5.5 Structural details

When each member of a culvert is designed considering the culvert as a frame structure, the design must be in accordance with structural details of a rigid frame generally.

19.6 Underground tanks

19.6.1 Definitions

full state : a state in which a liquid surface in a tank rises up to a design height

empty state : a state in which no liquid is in a tank

19.6.2 Scope

This section shall be applied to design of a side wall and a base of a cylindrical storage vessel constructed mostly in the ground.

19.6.3 Limit states and combinations of loads

(1) Check of an ultimate limit state for failure of members, in general, shall be made for combinations of loads in a normal state, a seismic state, and a construction state shown in Table 19.6.1.

(2) Check of a serviceability limit state for crack widths, in general, shall be made for a combination of loads in a normal state shown in Table 19.6.1.

Table 19.6.1 Limit states and combinations of loads

Limit states	States	Conditions of liquid	Loads							
			Self weight	Roof load	Liquid pressure	Inner pressure	Soil pressure	Underground water pressure and lift	Thermal load	Seismic load
Failure of member	Normal	Empty	○	○			○	○	○	
Crack width		Full	○	○	○	○	○	○	○	
Failure of member	Seismic	Empty	○	○			○	○	○	○
		Full	○	○	○	○	○	○	○	○
Failure of member	Construction		○				○	○		○

(3) Normal soil pressures acting on a tank shall be treated generally as a uniform load in a radial direction. Considering changes in conditions of ground around, a size of a structure and so on, partial soil pressures shall be added if necessary.

(4) For a tank around which ground is frozen by a low-temperature stored thing frozen soil pressures must be considered in empty states of normal and seismic states.

19.6.4 Structural analysis

(1) A structural analysis shall be made by a method suitable to mechanical characteristics and geometrical shapes of a side wall and a base, a method of connecting them, and supporting and loading conditions.

(2) A structural analysis of a side wall may be made in general considering it as a cylindrical shell. Boundary conditions shall be determined taking account of restrictions of ground around and a method for connecting a base.

(3) A structural analysis of a base may be made in general considering it as a thick circular plate. Boundary conditions shall be determined taking account of restrictions of ground under the base and a method for connecting a side wall.

(4) Sections for a structural nalysis may be gross sections of concrete in general. For thermal loads the stiffness may be reduced taking account of cracking. Degree of the reduction shall be limited to 1/2 of the stiffness of the gross section of concrete.

19.6.5 Structural details

(1) In a tension side of a member, reinforcement which is equal to or greater than 0.1 % of concrete section must be arranged at a space equal to or less than

30 cm and 1/2 of a member depth.
(2) For a part locally subjected to concentrated loads or thermal loads, precaution reinforcement as well as reinforcement needed by computation must be arranged in accordance with necessity.

19.7 Shield segments (in preparation)

19.8 Shotcrete (in preparation)

20 DESIGN OF SPECIAL STRUCTURES

20.1 General

This chapter should be applied to large size structures, tower structures, tubular structures and marine structures.

20.2 Large size structures

(1) This provision should be applied to structures whose summation of transverse length, longitudinal length and vertical length is greater than or equal to about 100 meters and size of cross section is greater than or equal to about 3 meters, when the structure is transformed into a rectangular solid.
(2) Structural analysis should be made by methods whose analytical accuracy is suitable.
(3) Structural model should be selected adequately so that object structures may be reflected truly.
(4) Seismic load, wind load and so on should be selected adequately after making research of existing data at construction site.
(5) The effects of hydration heat, drying shrinkage and creep of concrete,in principle, should be analyzed according to the order of stages of construction.

20.3 Tower and tubular structures

(1) This provision should be applied to vertical and comparatively flexible tower structures whose height is greater than or equal to about 25 meters and natural period is greater than or equal to about 0.5 second and height is relatively large in comparison with size of cross section and be applied to tubular structures whose length is relatively long in comparison with size of cross section and which are placed on the ground.
(2) Tower structures, in principle, should be designed for seismic loads according to the modified seismic force factor method. Large-scale tower structures should be designed according to the method of dynamic linear analysis. Tower structures whose height is less than or equal to about 30 meters may be designed according to the seismic force factor method.
(3) Tubular structures which are constructed on soft ground or on places whose soil conditions are extremely different, in principle, should be designed for seismic loads according to the method of dynamic linear analysis taking account

of phase difference.
(4) Tower structures which are largely influenced by axial compressive loads should be checked for buckling.

20.4 Marine structures

20.4.1 Scope

(1) This provision should be applied to general structures which are placed on seashore, large-scale marine floating structures which are placed on offshore and large water depth structures which are placed in ocean.
(2) Large-scale marine structures should be designed according to the provisions of Section 20.2, too.

20.4.2 Structural plans

(1) In making structural plans, purposes of service and functions of structures and actual conditions of natural circumstances, that is, wind, wave, height of sea level, ocean current, tidal current, coastal current, geographical features of bottom of sea and geological features, drifting sand, earthquake, tidal wave, high tide, temperature, water temperature, fog, quality of water, drift ice and so on, should be investigated adequately.
(2) At each stage of plans, investigation, construction and after in use, following considerations should be paid for the preservation of circumstances.

(1) estimation of variation of topography of sea bottom, tidal current and meteorology and prevention undesirable influence on them
(2) prevention of undesirable influence on ecological environment
(3) prevention of sea pollution according to the exhaust of oil or disposed waste
(4) prevention of air pollution
(5) regulation of noises
(6) accommodation with surrounding view

(3) Durable structural systems should be selected adequately taking account of construction methods, maintenance and management cost after in use, external forces and actual circumstances.

20.4.3 Loads

(1) Following loads, in general, should be considered as design loads after completion of construction.

1) Permanent load
   * self weight and mounted dead load
   * static hydraulic pressure and buoyancy
   * soil pressure
   * prestressing force
   * influence of creep and drying shrinkage of concrete
2) Variable load
   * mounted live load
   * load accompanied with trembling or inclination of structures
   * temperature load
   * force by ship at the contact with shore and force by ship in mooring
   * wave force and mooring force

* fluid force due to flow

3) Accidental load

* earthquake force
  force of inertia due to mass of structures
  external force due to displacement of the ground at earthquake
  soil pressure at earthquake
  dynamic hydraulic pressure at earthquake
* wind load
* wave force
* collision with ships
* collision with flying things
* functionally accidental load (explosion, extraordinarily high pressure, etc.)

(2) At construction stage, following loads should be considered besides loads stipulated in the provision (1).

* dynamic load in floating or towing
* pressure of flesh concrete
* pressure of aggregate

20.4.4 Combinations of limit state and load

(1) Check for ultimate limit states of sliding or falling down of structures and supporting capacity of ground and check for failure of cross section, in general, should be made taking account of following combinations of loads.

(1) one of variable loads (characteristic value) + permanent load (characteristic value) + variable load (value for combination of loads)

(2) one of accidental load (characteristic value) + permanent load (characteristic value) + variable load (value for combination of loads)

(2) check for serviceability limit states of crack, in general, should be made taking account of following combination of loads.

(1) permanent load (characteristic value) + variable load (value for combination of loads)

(3) Check for serviceability limit states of displacement, in general, should be made taking account of following combinations of loads.

(1) one of variable loads (characteristic value) + permanent load (characteristic value)

(2) one of accidental loads (characteristic value) + permanent load (characteristic value)

(4) Check for fatigue limit states, in general, should be made taking account of following combination of loads.

(1) one of variable loads (characteristic value) + permanent load (characteristic value)