EBCS-2

Ethiopian Building Code Standard

STRUCTURAL USE OF CONCRETE

Ministry of Works & Urban Development
Addis Ababa, Ethiopia
1995

EBCS-2
Structural Use of Concrete

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FOREWORD

The Proclamation to define the powers and duties of the Central and Regional Executive Organs of the Transitional Government of Ethiopia No. 41/1993 empowers the Ministry of Works and Urban Development to prepare the Country's Building Code, issue Standards for design and construction works, and follow up and supervise the implementation of same.

In exercise of these powers and in discharge of its responsibility, the Ministry is issuing a series of **Building Code Standards** of general application.

The purpose of these standards is to serve as nationally recognized documents, the application of which is deemed to ensure compliance of buildings with the minimum requirements for design, construction and quality of materials set down by the **National Building Code.**

The major benefits to be gained in applying these standards are the harmonization of professional practice and the ensuring of appropriate levels of safety, health and economy with due consideration of the objective conditions and needs of the country.

As these standards are technical documents which, by their very nature, require periodic updating, revised editions will be issued by the Ministry from time to time as appropriate.

The Ministry welcomes comments and suggestions on all aspect of the **Ethiopian Building** Code Standards. All feedback received will be carefully reviewed by professional experts in the field of building construction with a view to possible incorporation of amendments in future editions.

Haile Assegidie
Minister
Ministry of Works and
Urban Development
1995

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CHAPTER 1
GENERAL

1.1 SCOPE

(1) This Code of Practice applies to the design of buildings and civil engineering works in plain, reinforced and prestressed concrete made with normal weight aggregates.

(2) The Code has been published in two parts:

Part 1: Design, Materials and Construction
Part 2: Design Aids

(3) This Code is only concerned with the requirements for resistance, serviceability and durability of concrete structures. Other requirements, such as those concerning thermal or sound insulation, are not covered.

(4) Construction is covered to the extent that is necessary to indicate the quality of the construction materials and products which should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules. Construction and workmanship are covered in Chapters 8 and 9, and are to be considered as minimum requirements which may have to be further developed for particular types of buildings or civil engineering works and methods of construction.

(5) This Code does not cover the special requirements of seismic design. Provisions related to such requirements are given in EBCS 8 "Design of Structures for Earthquake Resistance" which complements, and is consistent with, EBCS 2.

(6) Numerical values of the actions on buildings and civil engineering works to be taken into account in the design are not given in this Code. They are provided in EBCS 1 "Basis of Design and Actions on Structures" applicable to the various types of construction.

(7) The design aids in Part 2 have been prepared in accordance with the assumptions laid down in Part 1, with the intention that they may be used as standard design aids and so avoid duplication of efforts by individual design offices.

(8) It has been assumed in the drafting of this Code that the design of concrete structures is entrusted to registered structural or civil engineers, appropriately qualified, for whose guidance it has been prepared and that the execution of the work is carried out under the direction of appropriately qualified supervisors.

1.2 CLASSIFICATION OF CONCRETE WORKS

(1) Concrete works are classified as either Class I or II depending on the quality of workmanship and the competence of the supervisors directing the works.

(2) Works carried out under the direction of appropriately qualified supervisors ensuring the attainment of level of quality control envisaged in Chapter 9 are classified as Class I works.

(3) Works with a lower level of quality control are classified as Class II works.

(4) Class II works are permissible only for single story structures.

1.3 UNITS

The units used in this Code are those of the International System of units known as SI, and shall be according to ISO 1000.

1.4 NOTATIONS

The symbols used in this Code are in accordance with ISO Standard 3898. The symbols used in this Code are as follows:

A_c	Area of concrete
$A_{c,ef}$	The section of the zone of the concrete where the reinforcing bars can effectively influence the crack widths
A_{ch}	Area of rectangular core of column measured out-to-out of hoop
A_{ct}	Area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack
A_d	Design value (specified value) of the accidental action
A_e	The cross-sectional of the longitudinal rainforcement
A_{ef}	Enclosed area within a mean polygonal perimeter
A_p	Area of prestressing tendon or tendons
A_s	Area of tension reinforcement
A_{s2}	Area of compression reinforcement
A_{sw}	Cross-sectional area of shear reinforcement
$A_{s,cal}$	Theoretical area of reinforcement required by the design
$A_{s,ef}$	Area of reinforcement actually provided
A_{sf}	Area of transverse reinforcement per unit length perpendicular to the webflange interface
A_{sh}	Area of transverse hoop bar
A_{sw}	The longitudinal steel inside the slab, within the projection of the slab
A_v	The area of shear reinforcement within a distance s
A_1	Loaded area of the restricted zone under local contact pressure
A_2	Distribution area of the local contact pressure
a_l	Distance for displacing the moment diagram (Fig. 10-3)
a_v	Shear span
a_1, a_2	The side length of area A_1 and A_2, respectively
b	Width of wall measured center-to-center of bracing walls, or width measured from the center of a bracing wall to the free edge, or actual flange width in a T or L beam
b_e	The effective width of a T-beam
b_w	Width of the web or rib of a member
b_1	Side length of the rectangle of outline u parallel to the eccentricity
b_2	Side length of the rectangle of outline u perpendicular to the eccentricity
b_w	Width of the web on T, I, or L beams
c	Concrete cover
d	The distance from extreme compression to centroid of tension rein-forcemtne
d_{ef}	The diameter of the largest circle which can be inscribed within u_{ef}

E_c	Tangent modulus of elasticity of concrete at a stress $\sigma_c = 0$ and at 28 days
E_{cd}	Design value of the secant modulus of elasticity
E_{ct}	Short term elastic modulus of concrete
$E_{c(t)}$	Tangent modulus of elasticity of concrete at a stress of $\sigma_c = 0$ and at time t
E_{cm}	Secant modulus of elasticity of concrete
E_s	Elastic modulus of reinforcement or prestressing steel
$E_c(t_o)$	Tangent modulus of elasticity at time t_o
E_{c28}	Tangent modulus of elasticity at 28 days
e	Eccentricity
e_a	Additional eccentricity according to Eq 4.1
e_e	Equivalent constant first-order eccentricity of the design axial load
e_{eq}	Equivalent uniaxial eccentricity
e_i	Initial eccentricity
e_o	Equivalent uniform first order eccentricity
e_{01}	Smaller first order eccentricity
e_{02}	Larger first order eccentricity
e_2	Second-order eccentricity (Section 4.4.10.3).
e_{tot}	Total eccentricity in the direction of the larger relative eccentricity
F_d	Design load
F_{gk}	Characteristic axial load of long duration causing creep
$F_{g+q,k}$	Characteristic total axial load
F_k	Characteristic load
F_{pu}	Ultimate tendon force
F_{ser}	Service value
	Frequent values
F_t	Tensile force developed by anchorage
f_{bd}	Design bond strength
f_c	Compressive strength of concrete
f_{cd}	Design compreseive strength of concrete
f_{ck}	Characteristic compressive strength of concrete
f_{cm}	Mean value of concrete compressive strength
$f_{ct,ef}$	Tensile strength of the concrete effective at the time when the cracks may first be expected to occur
f_{ctd}	Design tensile strength of concrete
f_{ctk}	Characteristic tensile strength of concrete
f_{ctm}	Mean value of axial tensile strength of concrete
f_d	Design strength
f_k	Characteristic strength
f_p	Tensile strength of prestressing steel
f_{pk}	Charactersistic tensile strengrh of prestressing steel
$f_{p0.1}$	0.1% proof-stress of prestressing steel
$f_{p0.1k}$	Charactersistic 0.1% proof-stress of prestressing steel
f_t	Tensile strength of reinforcement
f_{tk}	Characteristic tensile strength of reinforcement
f_y	Yield strength of reinforcement
f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reiforcement
f_{ywd}	Design yield strength of stirrups
$f_{0.2}$	Yield strength of reinforcement at 0.2% offset

G_{ind}	Indirect action
G_k	Characteristic permanent load
$G_{k,inf}$	Lower characteristic value of the permanent action
$G_{k,sup}$	Upper characteristic value of the permanent action
$G_{k,j}$	Characteristic value of permanent actions
G_1	Permanent stabilizing action
G_2	Permanent non-stabilizing action
g_d	is the uniformly distributed design permanent load
h	Overall depth of section in the plane of bending. Cross-sectional dimension in the direction of buckling
h_{ef}	Thickness of equivalent hollow section
h_f	Thickness of flange
h_1	Height of supported beam
h_2	Height of girder
I_c, I_s	Moments of inertia of the concrete and reinforcement sections, respectively, of the substitute column, with respect to the centroid of the concrete section
I_t	Second moment area of the uncracked transformed concrete section
i	Radius of gyration
i_p	Dispertion length
$J_{(t,to)}$	Creep function at time t
K_c	Flexural stiffness of column
K_{eq}	Flexural stiffness of equivalent column
K_{ij}	Effective beam stiffness coefficient (EI/L)
K_t	Total lateral stiffness of the columns of the story (story rigidity), with modulus of elasticity taken as unity
k	Relative eccentricity ratio Unintentional angular displacement (per unit length) related to the profile of the tendons
k_c	Coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking
k_1,k_2	Margin of strength
L	Clear span or clear height of a member
L_e	Effective buckling length
L_c	Distance between points of zero moments
L_x	Length of the shorter side of a panel
L_y	Length of the longer side of a panel
l	Nominal dimension
l_a	Anchorage length
l_b	Basic anchorage length
$l_{b,min}$	Minimum anchorage length
$l_{b,net}$	Required anchorage length
l_n	Length of clear span in direction that moments are being determined, measured face-to-face of supports
l_o	Length of lap of bars

$l_{o,min}$	Minimum lap length
l_p	Transmission length
l_s	Distance between points of zero shear
l_1	Length of span in direction that moments are being determined, measured center-tocenter of supports
l_2	Length of span transverse to l_1, measured center-to-center of supports
M_{bal}	Balanced moment capacity of the column.
M_{cr}	Theoretical cracking moment
M_d	Design moment at the critical section including second-order effects
M_k	Maximum applied moment at mid-span due to sustained characteristic loads
M_o	Total factored static moment
M_{sd}	Design value of the applied internal bending moment
M_u	Ultimate moment
M_1	Smaller first order moment due to design load
M_2	Larger first order moment due to design load
m_f	Span moment in two-way slab
m_t	Moment per unit width at the point of reference
m_{xf}	Span moment in the shorter direction of a panel
m_{yf}	Span moment in the longer direction of a panel
N_{cr}	Critical value for failure in a sway mode
N_d	Design axial load
N_{de}	Maximum design axial load acting on a column or wall during an earthquake
N_{td}	Transverse tensile force
N_{Sd}	Design axial force (tension or compression)
N_u	Ultimate axial load
N_{ub}	Axial load capacity of simultaneous assumed strain of concrete and yielding of tension steel
n	Number of bars in a bundle
	Modular ratio, E_s/E_{cm}
ΔP_c	Loss due to elastic deformation of the member at transfer
P_d	Design value of prestress at ultimate limit state
ΔP_{ir}	Short-term relaxation loss
$P_{k,sup}$, $P_{k,inf}$	are respectively the upper and lower characteristic values
P_m	Prestress after occurrence of all losses
$P_{m,o}$	Initial prestress at time $t = 0$
$P_{m,t}$	Mean value of the prestressing at time t and at a particular point along the member
P_o	Initial force at the active end of the tendon immediately after stressing
ΔP_{sl}	Loss due to anchorage slip
$\Delta P_t(t)$	Loss due to creep, shrinkage and relaxation at time t
$\Delta P_\mu(x)$	Loss due to friction
Q_k	Characteristic imposed loads
$Q_{k,1}$	Characteristic values of one of the variable actions
$Q_{k,i}$	Characteristic value of the other variable actions
Q_1	Variable non-stabilizing action
q_d	Uniformly distributed design live load
R_t	Coefficient given in Table A-2

S_d	Design situation for combination of action for ultimate limit states for persistent and transient design situation
s	Shear reinforcement spacing in the direction of the longitudinal reinforcement
s_h	Spacing of horizontal stirrups
s_{max}	Maximum spacing between stirrups
s_n	Standard deviation of the set of sample results
s_{rm}	Average distance between cracks
s_r	Spacing of vertical stirrups
T_c	Torque caried by the concrete
T_{ef}	Torsional resistance of the reinforcement
T_{Rd}	Torsional resistance of a section
T_{sd}	Design torsional moment strength provided by torsion reinforcement
t	Time
t_o	Time at initial loading of the concrete
u	Periphery of critical section
u_{ef}	Mean polygonal perimeter
V_c	Shear carried by the concrete
V_{cd}	Shear resistance of the concrete
V_{cn}	Additional shear force of a member subjected to axial force
V_h	Shear resistance of horizontal stirrups
V_{Rd}	Shear resistance of a section
V_{Rd1}	Shear resistance to inclined compression
V_{Rd2}	Shear resistance to diagonal tension
V_s	Shear resistance of vertical stirrups
V_{Sd}	Shear acting along the periphery u of the critical section
v	Punching shear
v_{sd}	Longitudinal unit shear Punching unit shear
$v_{sd,max}$	Maximum punching unit shear
w_k	Characteristic crack width
w_m	Mean crack width
w_1	Limining value of crack width
w_2	Limiting value of crack width
x	Neutral axis depth
x_1, x_2, x_3	Strength of lot
Z	Section Modulus
z	Internal lever arm
α	Coefficient for biaxial bending of columns (Table 5-1) Ratio of flexural stiffness of beam section to flexural stiffness of a width of slab bounded laterally by center lines of adjacent panels (if any) on each side of a beam
α_c	Ratio of flexural stiffness of columns above and below the slab to combined flexural stiffness of the slabs and beams at a joint taken in the direction of the span for which moments are being determined

α_{c1} Ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at one end of the column

α_{c2} Ratio of the sum of the column stiffnesses to the sum of the beam stiffnesses at the other end of the column

α_{cmin} The minimum of α_{c1} and α_{c2}

α_{ec} Ratio of flexural stiffness of equivalent column to combined flexural stiffness of the slabs and beams at a joint taken in the direction of the span for which moments are being determined

α_{i} Coefficient given in Table A-1 as function of aspect ratio L_y/L_x and support conditions

α_{min} Minimum α_c

α_1 α in the direction of l_1

α_2 α in the direction of l_2

β Shear coefficient given by Eq.6-13
Deflection coefficient depending on the loading condition
Ratio of long side to short side of footing

β_a Coefficient for effective depth given in Table 8-1
Ratio of dead load per unit area to live load per unit area (in each case without load factors)

β_a Factor for transmission length of prestressing strand

β_t Reduction factor for torsion due to combined action effects

β_v Reduction factor for shear due to combined action effects

γ_c Partial safety factor for concrete

γ_f Partial safety factor for loads
Fraction of unbalanced moment transferred by flexure at slab-column connection

γ_m Partial safety factor for materials

γ_p Partial safety factor for prestress

γ_s Partial safety factor for steel

γ_F, γ_G, γ_Q, γ_A and γ_P Partial safety factors for the action considered taking account of, for example, the the possibility of unfavorable deviation of the actions

$\gamma_{G,inf}$ Lower value of the partial safety factor for the permanent action

$\gamma_{G,sup}$ Upper value of the partial safety factor for the permanent action

$\gamma_{G,j}$ Partial safety factor for permanent action j

$\gamma_{GA,j}$ as $\gamma_{G,j}$, but for accidental design situations

$\Delta\sigma_{p,c+s+r}$ Variation of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t

σ_{pgo} Initial stress in the tendons due to prestress and permanent actions

$\Delta\sigma_{pr}$ Variation of stress in the tendons at section x due to relaxation

$\epsilon_s(t, t_o)$ Estimated shrinkage strain, derived from the values in Table 2.7 for final shrinkage

$\phi(t, t_o)$ Creep coefficient, as defined in Section 2.5.4

δ Reduction coefficient for redistribution of moments

δ_i Deflection due to the theoretical cracking moment M_{cr} acting on the uncracked transformed section

δ_{ii} Deflection due to the balance of the applied moment over and above the cracking value and acting on a section with an equivalent stiffness of 75% of the cracked value.

δ_{max} Deflection of fully cracked section

δ_{max} Deflection of fully cracked section

ϵ_c	Strain in concrete fiber
ϵ_s	Strain of reinforcement
ϵ_{sm}	Mean strain of reinforcement considering the contribution of concrete in tension
ϵ_1	The larger concrete strain below the neutral axis of the cracked section
ϵ_2	The smaller concrete strain below the neutral axis of the cracked section
$\epsilon_{cs\infty}$	Final shirinkage strain
ϵ_{uk}	Characteristic value of the elongation at maximum load
θ	Sum of the angular displacements over a distance x (irrespective of direction or sign)
κ_w	Correction coefficient to take account of the effect of the slope of stirrups on the spacing of cracks
κ_1	Coefficient which characterises the bond properties of bars
κ_2	Coefficient representing the influence of the form of the strees diagram
λ	Slenderness of a column
	Coefficient for standard deviation of the set of sample results
μ	Coefficient of friction between the tendons and their ducts
ν	Relative design axial load
ρ	Geometrical ratio of reinforcement
ρ_e	Effective geometrical ratio of reinforcement
ρ_{ex}	Geometrical ratio of reinforcement in the x direction
ρ_{ey}	Geometrical ratio of reinforcement in the y direction
ρ_{max}	Maximum geometrical ratio of reinforcement
ρ_{min}	Minimum geometrical ratio of reinforcement
ρ_w	Geometrical ratio of web reinforcement
$\rho_{w,min}$	Minimum geometrical ratio of web reinforcement
σ_{cpo}	Initial stress in the concrete adjacent to the tendons, due to prestress
σ_{cg}	Stress in the concrete adjacent to the tendons, due to self-weight and any other permanent actions
σ_{ct}	Maximum tensile stress in the concrete appropriate to a serviceability limit state
$\sigma_{o,max}$	Maximum stress applied to the tendon
σ_{pmo}	Stress in the tendon immediately after tensioning or transfer
σ_s	Maximum stress permitted in the reinforcement immediately after formation of the crack
σ_{sr}	Steel stress at rupture of concrete section
ϕ	Diameter of reinforcement bar
$\phi_{(t,to)}$	Creep coefficient
$\phi_{(\infty,to)}$	Final creep coefficient
ϕ_b	Diameter of bars forming the bundle
ϕ_e	Effective diameter of the bundle
$\phi_{(t,to)}$	Creep coefficient related to the elastic deformation at 28 days
ω	Mechanical reinforcement ratio
$\psi_o F_k$	Combination values
$\psi_2 F_k$	Quasi-permanent values
ψ_0	Combination value
ψ_1	Frequent value
ψ_2	Quasi-permanent value
ψ_0, ψ_1, ψ_2	Partial safety factors defined in Section 3.4.4 (3).

CHAPTER 2
DATA ON CONCRETE AND STEEL

2.1 GENERAL

(1) The strength and other data for the concrete are defined on the basis of standard tests.

2.2 GRADES OF CONCRETE

(1) Concrete is graded in terms of its characteristic compressive cube strength. The grade of concrete to be used in design depends on the classification of the concrete work and the intended use.

(2) Table 2.1 gives the permissible grades of concrete for the two classes of concrete works.

(3) The numbers in the grade designation denote the specified characteristic compressive strength in MPa.

Table 2.1 Grades of Concrete

Class	Permissible Grades of Concrete							
I	C5	C15	C20	C25	C30	C40	C50	C60
II	C5	C15	C20					

Grade C5 shall be used only as lean concrete

2.3 CHARACTERISTIC COMPRESSIVE STRENGTH OF CONCRETE

(1) For the purpose of this Code, compressive strength of concrete is determined from tests on 150 mm cubes at the age of 28 days in accordance with Ethiopian Standards.

(2) The characteristic compressive strength is defined as that strength below which 5% of all possible strength measurements may be expected to fall. In practice, the concrete may be regarded as complying with the grade specified for the design if the results of the tests comply with the acceptance criteria laid down in Chapter 9.

(3) Cylindrical or cubical specimens of other sizes may also be used with conversion factors determined from a comprehensive series of tests. In the absence of such tests, the conversion factors given in Table 2.2 may be applied to obtain the equivalent characteristic strength on the basis of 150 mm cubes.

(4) In Table 2.3 the characteristic cylinder compressive strength f_{ck} are given for the different grades of concrete.

Table 2.2 Conversion Factors for Strength

Size and Type of Test Specimen	Conversion Factor
Cube (200 mm)	1.05
Cylinder (150 mm diameter 300 mm height)	1.25

Table 2.3 Grades of Concrete and Characteristic Cylinder Compressive Strength f_{ck}

Grades of Concrete	C15	C20	C25	C30	C40	C50	C60
f_{ck}	12	16	20	24	32	40	48

2.4 CHARACTERISTIC TENSILE STRENGTH

(1) In this Code, the characteristic tensile strength refers to the axial tensile strength as determined by tests in accordance with standards issued or approved by Ethiopian Standards.

(2) In the absence of more accurate data, the characteristic tensile strength may also be determined from the characteristic cylinder compressive strength according to Eq. 2.1.

$$f_{ctk} = 0.7 f_{ctm} \tag{2.1}$$

where f_{ctm} is the mean value given by Eq. 2.2.

$$f_{ctm} = 0.3 f_{ck}^{2/3} \tag{2.2}$$

(3) The corresponding values of f_{ctk} and f_{ctm} for the different grades of concrete are given in Table 2.4.

Table 2.4 Grades of Concrete and Values of f_{ctk} and f_{ctm}

Grades of Concrete	C15	C20	C25	C30	C40	C50	C60
f_{ctm}	1.6	1.9	2.2	2.5	3.0	3.5	4.0
f_{ctk}	1.1	1.3	1.5	1.7	2.1	2.5	2.8

2.5 DEFORMATION PROPERTIES OF CONCRETE

(1) The values of the material properties required for the calculation of instantaneous and time dependent deformations of concrete depend not only upon the grades of concrete but also upon the properties of the aggregates and other parameters related to the mix design and the environment. For this reason, where an accurate calculation is considered necessary, the values shall be established from known data appropriate to the particular materials and conditions of use. For many calculations an approximate estimate will usually be sufficient.

2.5.1 Stress-Strain Diagrams

(1) Any idealized stress-strain diagram which results in prediction of strength in substantial agreement with the results of comprehensive tests may be used (see Section 4.4)

2.5.2 Modulus of Elasticity

(1) The modulus of elasticity depends not only on the concrete grade but also on the actual properties of the aggregates used (see Section 2.5(1) above).

(2) In the absence of more accurate data, or in cases where great accuracy is not required, an estimate of the mean value of the secant modulus E_{cm} can be obtained from Table 2.5 for a given concrete grade.

Table 2.5 Values of the Secant Modulus of Elasticity E_{cm} in GPa

Grades of Concrete	C15	C20	C25	C30	C40	C50	C60
E_{cm}	26	27	29	32	35	37	39

(3) The values in Table 2.5 are based on the following equation:

$$E_{cm} = 9.5(f_{ck} + 8)^{1/3} \tag{2.3}$$

Where E_{cm} is in GPa and f_{ck} is in MPa. They relate to concrete cured under normal conditions and made with aggregates predominantly consisting of quartzite gravel. When deflections are of great importance, tests shall be carried out on concrete made with the aggregate to be used in the structure. In other cases experience with a particular aggregate, backed by general test data, will often provide a reliable value for E_{cm}, but with unknown aggregates, it would be advisable to consider a range of values.

(4) As a rule, since the grade of concrete corresponds to a strength at an age of 28 days, the values of E_{cm} in Table 2.5 also relate to that same age. Where great accuracy is not required, E_{cm} can also be determined from Eq.2.3 for a concrete age t other than 28 days. In this case, f_{ck} is replaced by the actual cylinder concrete strength at time t.

2.5.3 Poisson's Ratio

(1) Any value between 0 and 0.2 can be adopted for Poisson's ratio.

2.5.4 Creep and Shrinkage

(1) Creep and shrinkage of the concrete depend mainly on the ambient humidity, the dimensions of the element and the composition of the concrete. Creep is also influenced by the maturity of the concrete when the load is first applied and on the duration and magnitude of the loading. Any estimation of the creep coefficient $\phi_{(t,to)}$, and of the basic shrinkage strain, ϵ_{cs}, shall take these parameters into account.

(2) In cases where great accuracy is not required, the values given in Tables 2.6 and 2.7 respectively can be considered as the final creep coefficient $\phi_{(\infty,to)}$ and the final shrinkage strain ϵ_{cs} of a normal weight concrete subjected to a compressive stress not exceeding $0.45f_{ck}$ at the time t_o at first loading.

(3) The data given in Tables 2.6 and 2.7 apply for a range of the mean temperature of the concrete between 10 °C and 20 °C. Maximum seasonal temperature up to 40 °C can be accepted. In the same way, variations in relative humidity around the mean values given in Tables 2.6 and 2.7 between RH = 20% and RH = 100% are acceptable.

(4) Linear interpolation between the values in Tables 2.6 and 2.7 is permitted.

Table 2.6 Final Creep Coefficient $\phi_{(\infty,to)}$ of Normal Weight Concrete

Age at Loading t_o (days)	Notional size $2A_c/u$ (in mm)					
	50	150	600	50	150	600
	Dry atmospheric conditions (inside) (RH = 50%)			Humid atmospheric conditions (outside) (RH = 80%)		
1	5.5	4.6	3.7	3.6	3.2	2.9
7	3.9	3.1	2.6	2.6	2.3	2.0
28	3.0	2.5	2.0	1.9	1.7	1.5
90	2.4	2.0	1.6	1.5	1.4	1.2
365	1.8	1.5	1.2	1.1	1.0	1.0

Table 2.7 Final Shrinkage Strains $\epsilon_{cs\infty}$ (in ‰) of Normal Weight Concrete

Location of the number	Relative humidity (%)	Notional size $2A_c/u$ (mm) ≤ 150	600
Inside	50	- 0.60	- 0.50
Outside	80	- 0.33	- 0.28

where: A_c = cross-sectional area of concrete
u = perimeter of that area

(5) The values of Tables 2.6 and 2.7 apply to concrete having plastic consistency when fresh. For concrete of other consistency the values have to be multiplied by 0.70 (stiff consistency) or 1.20 (soft consistency)

(6) For concrete with superplasticizers, the consistency before adding the superplasticizers is used for the evaluation of the creep and shrinkage coefficients as given in Tables 2.6 and 2.7.

2.5.5 Coefficient of Thermal Expansion

(1) The coefficient of thermal expansion may be taken as 10×10^{-6} per °C.

2.6 CHARACTERISTIC STRENGTH OF REINFORCING STEEL

(1) The mechanical and technological properties of steel used for reinforced concrete shall be defined by standard and/or agrément documents or by certificates of compliance.

(2) The characteristic strength f_{yk} is defined as the 5% fractile of the proof stress f_y or 0.2% offset strength, denoted as $f_{0.2}$

(3) If the steel supplier guarantees a minimum value for f_y or $f_{0.2}$, that value may be taken as the characteristic strength.

2.7 CLASSIFICATION AND GEOMETRY OF REINFORCING STEEL

(1) Reinforcing steel shall be classified according to:

(a) Grade, denoting the value of the specified characteristic yield stress (f_{yk}) in MPa.
(b) Class, indicating the ductility characteristics
(c) Size
(d) Surface characteristics
(e) Weldability

(2) Each consignment shall be accompanied by a certificate containing all the information necessary for its identification with regard to (a) to (e) above, and additional information where necessary.

(3) The actual cross sectional area of the products shall not differ from their nominal cross sectional area by more than the limits specified in relevant Standards.

(4) In this Code, two classes of ductility are defined (see Section 2.9.2):
(a) high (Class A)
(b) normal (Class B)

(5) In this Code two shapes of surface characteristics are defined:
(a) Ribbed bars, resulting in high bond action
(b) Plain, smooth bars, resulting in low bond action.

(6) For other types of bar, with other surface characteristics (ribs or indentations), reference should be made to relevant documents, based on test data.

(7) Welded fabric, used as reinforcing steel, shall comply with the dimensional requirements in relevant Standards.

2.8 PHYSICAL PROPERTIES OF REINFORCING STEEL

(1) The following mean values may be assumed:

(a) Density	7 850 kg/m^3
(b) Coefficient of thermal expansion	10×10^{-6} per °C

2.9 MECHANICAL PROPERTIES OF REINFORCING STEEL

2.9.1 Strength

(1) The yield stress f_{yk} and the tensile strength f_{tk} are defined respectively as the characteristic value of the yield load, and the characteristic maximum load in direct axial tension, each divided by the nominal cross sectional area.

(2) For products without a pronounced yield stress f_{yk}, the 0.2% proof stress $f_{0.2k}$ may be substituted.

2.9.2 Ductility

(1) The products shall have adequate ductility in elongation, as specified in relevant Standards.

(2) Adequate ductility in elongation may be assumed, for design purposes, if the products satisfy the following ductility requirements:

(a) High ductility: $\epsilon_{uk} > 5\%$; value of $(f_t / f_y)_k > 1.08$
(b) Normal ductility: $\epsilon_{uk} > 2.5\%$; value of $(f_t / f_y)_k > 1.05$

In which ϵ_{uk} denotes the characteristic value of the elongation at maximum load.

(3) High bond bars with diameters less than 6 mm shall not be treated as having high ductility.

(4) The products shall have adequate bendability for the anticipated use.

2.9.3 Stress-Strain Diagram

(1) In the absence of more accurate information, an elasto-plastic diagram can be used for hot rolled steel or steel cold worked by drawing or rolling.

(2) For other types of production, the actual stress-strain diagrams can be replaced by bilinear, trilinear or other diagrams chosen so that the approximations are on the safe side.

2.9.4 Modulus of Elasticity

(1) The mean value of modulus of elasticity E_s may be assumed as 200 GPa.

2.9.5 Fatigue

(1) Where required, the products shall have adequate fatigue strength.

2.10 TECHNOLOGICAL PROPERTIES

2.10.1 Bond and Anchorage

(1) The surface characteristics of ribbed bars shall be such that adequate bond is obtained with the concrete, permitting the full force that is assumed in design, to be developed in the reinforcement.

(2) Ribbed bars, having projected ribs not satisfying the requirements for high bond bars given in relevant standards shall be treated as plain bars with respect to bond.

(3) The behavior in bond of reinforcing steels with other surface shapes shall be defined in relevant Standards or technical approved documents.

(4) The strength of the welded joints along the anchorage length of welded fabric shall be adequate.

(5) The strength of the welded joint can withstand a shearing force not less than 30% of a force equivalent to the specified characteristic yield stress times the nominal cross sectional area of the anchored wire.

2.10.2 Weldability

(1) The products shall have weldability properties adequate for the anticipated use

(2) Where required, and where the weldability is unknown, tests should be requested.

(3) Ductility characteristics; as specified in Section 2.9.2, shall be maintained, when necessary, at sections near to weld.

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CHAPTER 3
BASIS OF DESIGN

3.1 FUNDAMENTAL REQUIREMENTS

(1) A structure shall be designed and constructed in such a way that:

(a) With acceptable probability, it will remain fit for the use for which it is required, having due regard to its intended life, and
(b) with appropriate degrees of reliability, it will sustain all actions and influences likely to occur during normal execution and use and have adequate durability.

(2) A structure shall also be designed in such a way that it will not be damaged by events like explosions, impact or consequences of human errors, to an extent disproportionate to the original cause.

(3) The potential damage due to the events in (2) above shall be minimized or avoided by appropriate choice of one or more of the following:

(a) Avoiding, eliminating or reducing the hazards which the structure is to sustain.
(b) Selecting a structural form which has low sensitivity to the hazards considered.
(c) Selecting a structural form and design that can survive adequately the accidental removal of an individual element.
(d) Tying the structure together.

(4) The above requirements shall be met by the choice of suitable materials, by appropriate design and detailing and by compliance with control procedures for production, construction and use envisaged in this Code.

3.2 LIMIT STATES

3.2.1 General

(1) A structure, or part of a structure, is considered unfit for use when it exceeds a particular state, called a limit state, beyond which it infringes one of the criteria governing its performance or use.

(2) All relevant limit states shall be considered in the design so as to ensure an adequate degree of safety and serviceability. The usual approach will be to design on the basis of the most critical limit state and then to check that the remaining limit states will not be reached.

(3) The limit states can be placed in two categories:

(a) The *Ultimate Limit States* are those associated with collapse, or with other forms of structural failure which may endanger the safety of people. States prior to structural collapse which, for simplicity, are considered in place of the collapse itself are also treated as ultimate limit states.
(b) The *Serviceability Limit States* correspond to states beyond which specified service requirements are no longer met.

3.2.2 Ultimate Limit States

(1) The ultimate limit states which may require consideration include:

(a) Loss of equilibrium of a part or the whole of the structure considered as a rigid body.
(b) Failure by excessive deformation, rupture or loss of stability of the structure or any part of it, including supports and foundations.

3.2.3 Serviceability Limit State

(1) Serviceability limit states which may require consideration include;

(a) Deformations or deflections which affect the appearance or effective use of the structure (including the malfunction of machines or services) or cause damage to finishes of non-structural elements.
(b) Vibration which causes discomfort to people, damage to the building or its contents, or which limits its functional effectiveness.
(c) Cracking of the concrete which is likely to affect appearance, durability or water tightness adversely.

3.3 DESIGN SITUATIONS

(1) Design situations are classified as:

(a) Persistent situations corresponding to normal conditions of use of the structure.
(b) Transient situations, such as those, for example during construction or repair.
(c) Accidental situations.

3.4 ACTIONS

3.4.1 Definitions and Principal Classification

(1) An action F is:

(a) A force (load) applied to the structure (direct action), or
(b) an imposed deformation (indirect action); for example, temperature effects or settlement.

(2) Actions are classified:

(a) By their variation in time:
  (i) Permanent actions (G), e.g. self-weight of structures, fittings ancillaries and fixed equipment.
  (ii) Variable actions (Q), e.g. imposed loads or wind loads.
  (iii) Accidental actions (A), e.g. explosions or impact from vehicles.

(b) By their spatial variation:
  (i) Fixed actions, e.g. self-weight.
  (ii) Free actions, which result in different arrangements of actions, e.g. movable imposed loads and wind loads.

(3) Prestressing (P) is a permanent action but, for practical reasons, it is treated separately.

(4) Indirect actions are either permanent G_{ind} (e.g. settlement of support) or variable Q_{ind} (e.g. temperature) and are treated accordingly.

(5) Supplementary classifications relating to the response of the structure are given in the relevant sections of this Code.

3.4.2 Representative Values of Actions

(1) For verification in the partial safety factor method, actions are introduced into the calculations by representative values, i.e. by values corresponding to certain levels of intensity. For different calculations, one may have to distinguish different representative values of an action according to its variation in time. The complete set of representative values is as follows:

(a) Characteristic values, F_k
(b) Combination values, $\psi_o F_k$
(c) Frequent values, $\psi_1 F_k$
(d) Quasi-permanent values, $\psi_2 F_k$

The above values are evaluated mainly on a statistical basis.

(2) Maximum values and minimum values, which may be zero, are defined when appropriate.

(3) Depending on the variation with time of certain actions, their representative values are sometimes subclassified as actions of long duration (or sustained actions) or of short duration (or transient actions). In special cases, certain actions have their representative values divided into sustained and transient components.

3.4.3 Representative Values of Permanent Actions

(1) The representative values of permanent actions are specified as:

(a) The characteristic values F_k specified in EBCS1 - "Basis of Design and Actions on Structures", or
(b) by the client, or the designer in consultation with the client, provided that minimum provisions, specified in the relevant codes or by the competent authority, are observed.

(2) The other representative values are assumed to be equal to those in Section 3.4.2(1).

(3) For permanent actions where the coefficient of variation is large or where the actions are likely to vary during the life of the structure (e.g. for some superimposed permanent loads), two characteristic values are distinguished, an upper ($G_{k,sup}$) and a lower ($G_{k,inf}$). Elsewhere, a single characteristic value (G_k) is sufficient.

(4) The self-weight of the structure may, in most cases, be calculated on the basis of the nominal dimensions and mean unit masses.

3.4.4 Representative Values of Variable Actions

(1) The main representative value is the characteristic value Q_k.

(2) For variable actions, the characteristic value (Q_k) corresponds to either:

(a) The upper value with an intended probability of not being exceeded or the lower value with an intended probability of not being reached, during some reference period, having regard to the intended life of the structure or the assumed duration of the design situation, or
(b) the specified value.

(3) Other representative values are expressed in terms of the characteristic value Q_k by means of a factor ψ_i. These values are defined as:

(a) Combination value: $\psi_0 Q_k$
(b) Frequent value: $\psi_1 Q_k$
(c) Quasi-permanent value: $\psi_2 Q_k$

(4) Supplementary representative values are used for fatigue verification and dynamic analysis.

(5) The factors ψ_i are specified:

(a) in EBCS1 - "Basis of Design and Actions on Structures", or
(b) by the client or the designer in conjunction with the client, provided that minimum provisions, specified in the relevant codes or by the competent public authority, are observed.

3.4.5 Representative Values of Accidental Actions

(1) The representative value of accidental actions is the characteristic value A_k (when relevant) and generally correspond to a specified unique nominal value beyond which there is no longer an assurance of a probability of survival of the structure.

(2) Their service, combination and frequent values are considered negligible or zero.

3.4.6 Design Values of Actions

(1) The design value F_d of an action is expressed in general terms as

$$F_d = \gamma_F F_k \tag{3.1}$$

Specific examples are:

$$\begin{aligned} G_d &= \gamma_G G_k \\ Q_d &= \gamma_Q Q_k \text{ or } \gamma_Q \psi_i Q_k \\ A_d &= \gamma_A A_k \text{ (if } A_d \text{ is not directly specified)} \\ P_d &= \gamma_P P_k \end{aligned} \tag{3.2}$$

where γ_F, γ_G, γ_Q, γ_A and γ_P are the partial safety factors for the action considered taking account of, for example, the possibility of unfavorable deviations of the actions, the possibility of inaccurate modelling of the actions, uncertainties in the assessment of effects of actions, and uncertainties in the assessment of the limit state considered.

(2) The upper and lower design values of permanent actions are expressed as follows:

(a) Where only a single characteristic value G_k is used, then

$$G_{d,sup} = \gamma_{G,sup}\, G_k$$
$$G_{d,inf} = \gamma_{G,inf}\, G_k$$

(b) Where upper and lower characteristic values of permanent actions are used, then

$$G_{d,sup} = \gamma_{G,sup}\, G_{k,sup}$$
$$G_{d,inf} = \gamma_{G,inf}\, G_{k,inf}$$

where $G_{k,\,inf}$ is the lower characteristic value of the permanent action
$G_{k,\,sup}$ is the upper characteristic value of the permanent action
$\gamma_{G,\,inf}$ is the lower value of the partial safety factor for the permanent action
$\gamma_{G,\,sup}$ is the upper value of the partial safety factor for the permanent action

3.4.7 Design values of the Effects of Actions

(1) The effects of actions are responses (e.g. internal forces and moments, stresses, strains) of the structure to the actions. Design values of the effects of actions are determined from the design values of the actions, geometrical data and material properties when relevant.

(2) In some cases, in particular for nonlinear analysis, the effect of the randomness of the intensity of the actions and the uncertainty associated with the analytical procedures, e.g. the models used in the calculations, shall be considered separately. This may be achieved by the application of a coefficient of model uncertainty, either applied to the actions or to the internal forces and moments.

3.5 MATERIALS

3.5.1 Characteristic Strength

(1) A material property is represented by a characteristic value which in general corresponds to a fractile in the assumed statistical distribution of the particular property of the material, specified by relevant standards and tested under specified conditions.

(2) In certain cases, a nominal value is used as the characteristic value.

(3) The characteristic strength of concrete and steel is defined in Sections 2.3 and 2.6, respectively.

3.5.2 Design Strength

(1) The design strength for a given material property and limit state is obtained, in principle, by dividing the characteristic strength f_k by the appropriate partial safety factor for the material property γ_m, i.e.,

$$f_d = \frac{f_k}{\gamma_m} \tag{3.3}$$

(2) However, in the case of concrete under compression, a further correction factor is introduced in this Code for convenience (see Eq. 3.4).

3.5.3 Partial Safety Factors for Materials

3.5.3.1 *Ultimate Limit State*

(1) Partial safety factor for materials appropriate to various design situations, ordinary and accidental, are given in Tables 3.1 and 3.2 for the two classes of concrete works, Class I and Class II, respectively.

Table 3.1 Partial Safety Factor for Materials - Class I Works

Design Situations	Concrete, γ_c	Reinforcing Steel, γ_s
Persistent and Transient	1.50	1.15
Accidental	1.30	1.00

Table 3.2 Partial Safety Factor for Materials - Class II Works

Design Situations	Concrete, γ_c	Reinforcing Steel, γ_s
Persistent and Transient	1.65	1.20
Accidental	1.45	1.10

3.5.3.2 *Serviceability Limit States*

(1) The value of γ_m in the serviceability limit states may be taken as 1.0 for both steel and concrete.

3.5.4 Design Strength for Concrete

(1) The design strength of concrete is defined by:

(a) In compression

$$f_{cd} = \frac{0.85 f_{ck}}{\gamma_c} \tag{3.4}$$

(b) In tension

$$f_{ctd} = \frac{f_{ctk}}{\gamma_c} \tag{3.5}$$

3.5.5 Design Strength for Steel

(1) The design strength of steel in tension and compression is defined by:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \tag{3.6}$$

3.6 COMBINATION OF ACTIONS

3.6.1 Ultimate Limit States

(1) The combination of actions for the ultimate limit states for persistent and transient design situation shall be in accordance with Eq. 3.7:

$$S_d = S\left\{\Sigma\gamma_{G,j}G_{k,j} + \gamma_{Q,1}Q_{k,1} + \sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i}\right\} \quad (3.7)$$

(2) The combination for accidental design situation shall be

$$S_d = \left\{\Sigma\gamma_{GA,j} + A_d + \psi_{1,1}Q_{k,1} + \sum_{i>1}\psi_{2,1}Q_{k,1}\right\} \quad (3.8)$$

where
- $G_{k,j}$ is the characteristic value of permanent actions
- $Q_{k,1}$ is the characteristic value of one of the variable actions
- $Q_{k,i}$ is the characteristic value of the other variable actions
- A_d is the design value (specified value) of the accidental action
- $\gamma_{G,j}$ is the partial safety factor for permanent action j
- $\gamma_{GA,j}$ is as $\gamma_{G,j}$, but for accidental design situations
- $\gamma_{Q,i}$ is the partial safety factor for variable action i
- ψ_0, ψ_1, ψ_2 are partial safety factors defined in Section 3.4.4 (3).

(3) Combinations for accidental design situations either involve an explicit accidental action A (e.g. shock) or refer to a situation after an accidental event ($A = 0$). Unless specified otherwise, $\gamma_{GA} = 1$ may be used.

(4) Partial safety factors for various design situations are given in Table 3.3.

Table 3.3 Partial Safety Factors for Actions in Building Structures

Design Situation	Action	Factor γ	Favorable	Unfavorable
Persistent and Transient	Permanent	γ_G	1.00	1.30
	Variable	γ_Q	0	1.60
Accidental	Permanent	γ_G	1.00	1.00

(5) For building structures, Eq. 3.9 may be used in lieu of Eqs. 3.7 and 3.8:

$$S_d = S(1.30G + 1.60Q_{vk}) \quad (3.9a)$$

$$S_d = S(1.0G + 1.60Q_{hk}) \quad (3.9b)$$

$$S_d = S(1.20(G + Q_{vk} + Q_{hk})) \quad (3.9c)$$

$$S_d = S(A_d + G + Q_{vk}) \quad (3.9d)$$

$0.75(1.3G_1 + 1.6Q_{hk}) + \Delta sd$

(6) The combination for static equilibrium may be taken as

$$S_d = S(0.9G_1 - 1.1G_2 - 1.6Q_1) \quad (3.10)$$

where G_1 is the permanent stabilizing action
G_2 is the permanent non-stabilizing action
Q_1 is the variable non-stabilizing action

3.7 ANALYSIS OF LINE ELEMENTS

3.7.1 Methods of Analysis

(1) For analysis in the Ultimate Limit State, plastic, non-linear and linear elastic theory may be applied.

(2) Elastic methods of analysis may be applied for analysis in the Serviceability Limit State and for the Alternate Design method.

3.7.2 Load Arrangements and Load Cases

(1) A load arrangement identifies the position, magnitude and direction of a free action.

(2) A load case identifies compatible load arrangements, sets of deformations and imperfections considered for a particular verification.

(3) Detailed rules on load arrangements and load cases are given in EBCS1 - "Basis of Design and Actions on Structures"

(4) The following simplifying assumptions may be made for computing load-effects in frames due to gravity loading:

(a) The live load may be considered to be applied only to the floor or roof under consideration, and the far ends of the columns may be assumed as fixed.
(b) Consideration may be limited to the following load cases:

(i) Design dead load on all spans with full design live load on two adjacent spans.
(ii) Design dead load on all spans with full design live load on alternate spans.

3.7.3 Imperfections

(1) In the Ultimate Limit State, consideration shall be given to the effects of possible imperfections in the geometry of the unloaded structure. Where significant, any possible unfavorable effect of such imperfections shall be taken into account.

(2) Individual sections shall be designed for the internal forces and moments arising from global analysis, combining effects of actions and imperfections of the structure as a whole.

(3) In the absence of other provisions, the effects of imperfections may be assessed by assuming that the structure is inclined to the vertical at an angle ϕ defined by:

(a) For single storey frames or for structures loaded mainly at the top

$$\tan\phi = \frac{1}{150} \tag{3.11}$$

(b) For other types of frames

$$\tan\phi = \frac{1}{200} \tag{3.12}$$

(4) Where the effects of imperfections are smaller than the effects of design horizontal actions, their influence may be ignored. Imperfections need not be considered in accidental combinations of actions.

3.7.4 Time Dependent Effects

(1) Time dependent effects shall be taken into account where significant.

(2) Creep and shrinkage normally need only be considered for the Serviceability Limit State except where their influence on second-order effects are likely to be significant.

3.7.5 Idealization of the Structure

(1) The elements of a structure are normally classified, by consideration of their nature and function, as beams, columns, slabs, walls, plates, arches, shells, etc. Rules are provided for the analysis of the commoner of these elements and of structures consisting of combinations of these elements.

(2) To be considered as a beam or a column, the span or length of the member shall not be less than twice the overall section depth. A beam whose span is less than twice its depth is considered as a deep beam.

(3) To be considered as a slab, the minimum span shall not be less than four times the overall slab thickness.

(4) A slab subjected to predominantly uniformly distributed loads may be considered to be one-way spanning if either:

(a) it possess two free (unsupported) and sensibly parallel edges, or
(b) if it is the central part of a sensibly rectangular slab supported on four edges with a ratio of the longer to shorter span greater than 2.

(5) Ribbed or waffle slabs may be treated as solid slabs for the purposes of analysis, provided that the flange of structural topping and transverse ribs have sufficient torsional stiffness. This may be assumed provided:

(a) The rib spacing does not exceed 1.5 m.
(b) The depth of the rib below the flange does not exceed four times its width.
(c) The depth of the flange is at least 1/10 of the clear distance between ribs or 50 mm, whichever is greater.
(d) Transverse ribs are provided at a clear spacing not exceeding 10 times the overall depth of the slab.

The minimum flange thickness of 50 mm may be reduced to 40 mm where permanent blocks are incorporated between the ribs.

(6) A wall shall have a horizontal length of at least four times its thickness. Otherwise it shall be treated as a column.

3.7.6 Stiffness

(1) Any reasonable assumptions may be adopted for computing the relative flexural and torsional stiffness of members. The assumptions made shall be consistent throughout the analysis.

3.7.7 Effective Span Length

(1) The effective span of a simply supported member shall be taken as the lower of the following two values:

(a) The distance between the center lines of the supports.
(b) The clear distance between the faces of the supports plus the effective depth.

(2) The effective span of a continuous element shall normally be taken as the distance between the center lines of the supports.

(3) For a cantilever, the effective span is taken to be its length, measured from:

(a) The face of the supports, for an isolated, fixed-ended cantilever.
(b) The center line of the support for a cantilever which forms the end of a continuous beam.

3.7.8 Effective Flange Width for T- Beams and L-Beams

(1) In the absence of a more accurate determination, the effective width to be used to obtain the load-effects for a given span of a symmetrical T-beam shall not exceed the lesser of:

(a) The thickness of the web plus one-fifth of the effective span, or
(b) The actual width of the top slab (extending between the centers of the adjacent spans).

(2) The effective width shall be taken as constant over the entire span, including the parts near intermediate supports for continuous beams.

(3) For edge beams (L-beams), the effective width shall not exceed the lesser of:

(a) The thickness of the web plus one-tenth of the effective span
(b) The thickness of the web plus half the clear distance to the adjacent beam.

3.7.9 Redistribution of Moments

(1) Moments obtained from a linear analysis may be reduced by multiplying by the following reduction coefficient δ provided that the moments are increased in other sections in order to maintain equilibrium.

(2) For continuous beams and for beams in rigid jointed braced frames with span/effective-depth ratio not greater than 20,

$$\delta = 0.44 + 1.25(\frac{x}{d}) \qquad (3.13)$$

The neutral axis height, x, is calculated at the ultimate limit state and the term x/d refers to the section where the moment is reduced.

(3) For other continuous beams and rigid jointed braced frames

$$\delta \geq 0.75 \qquad (3.14)$$

(4) For sway frames with slenderness ratio λ of columns less than 25

$$\delta \geq 0.90 \qquad (3.15)$$

3.7.10 Second-Order Effects

(1) Second-order effects shall be taken into account where they may significantly affect the overall stability of a structure or the attainment of the ultimate limit state at critical sections.

(2) For normal buildings, second-order effects may be neglected where they increase the moments, calculated ignoring displacements, by not more than 10%.

3.8 ANALYSIS AND DESIGN OF PLANE ELEMENTS

3.8.1 Slabs

3.8.1.1 *Methods of Analysis*

(1) Moments and internal shear forces may be determined on the basis of the following types of analysis:

(a) Linear analysis, optionally followed by redistribution
(b) Plastic analysis
(c) Non-linear analysis

3.8.1.2 *Linear Analysis, with or without Redistribution*

(1) The linear analysis shall be based generally on the gross cross-sections by adopting for Poisson's ratio a value between 0 and 0.2.

(2) Linear analysis is valid for the Serviceability Limit States and for the Ultimate Limit States.

(3) If required, the support moments in continuous slabs. resulting from a linear analysis may be reduced by not more than 25%, for an appropriate width, provided that the corresponding average moments for the same width at midspan, are adjusted to satisfy equilibrium, provided further that the provisions of Section 3.7.9 (1) and (2) are complied with.

(4) Appendix A, which is based on linear analysis with redistribution, may be used for the analysis of two-way slabs. No further redistribution is, however, allowed.

3.8.1.3 *Plastic Analysis*

(1) In general, plastic analysis is only applicable to the ultimate limit states. Both static (e.g., the strip method) and dynamic (e.g., yield line theory) methods may be used.

(2) The following conditions shall be satisfied:

(a) When using plastic analysis, the area of tensile reinforcement shall not exceed, at any point or in any direction a value corresponding to $x/d = 0.25$.
(b) If a static method is used, the moment distribution selected shall not differ substantially from the elastic moment distribution.
(c) If a dynamic method is used, the ratio of the support moments to the mid-span moments shall normally be not less than 0.5, nor more than 2.

3.8.2 Flat Slabs

3.8.2.1 *Definition*

(1) The term flat slabs or plate means a reinforced concrete slab with or without drops and supported, generally without beams, by columns with or without flared column heads.

3.8.2.2 *Method of Analysis*

(1) The forces acting in the middle plane of a plate can be determined on the basis of any of the following types of analysis:

(a) Linear Analysis
(b) Plastic Analysis
(c) Non-linear Analysis

(2) The empirical method or the equivalent frame method given in Appendix A may be used for the analysis of flat slabs and two-way slab systems.

CHAPTER 4
ULTIMATE LIMIT STATES

4.1 SCOPE

(1) This chapter gives methods of analysis and design of linear elements that in general ensure that the objectives set out in Chapter 3 for the Ultimate Limit State are met.

(2) Other methods may be used provided they can be shown to be satisfactory for the type of structure or member considered.

(3) It is assumed that the ultimate limit state is the critical limit state.

4.2 BASIS OF DESIGN

4.2.1 Analysis of Sections

(1) The calculation of the ultimate resistance of members for flexure and axial loads shall be based on the following assumptions, in addition to those given in Sections 3.7 and 3.8.

(a) Plane sections remain plane
(b) The reinforcement is subjected to the same variations in strain as the adjacent concrete
(c) The tensile strength of the concrete is neglected
(d) The maximum compressive strain in the concrete is taken to be:
    0.0035 in bending (simple or compound)
    0.002 in axial compression
(e) The maximum tensile strain in the reinforcement is taken to be 0.01.

4.2.2 Strain Distribution

(1) Referring to Fig. 4.1, the strain diagram shall be assumed to pass through one of the three points *A*, *B* or *C*.

4.2.3 Idealized Stress-Strain Diagram for Concrete

4.2.3.1 *Parabolic-Rectangular Diagram*

(1) The parabolic-rectanglar stress distribution shown in Fig. 4.2 may be used for calculation of section capacity.

4.2.3.2 *Rectanglar Diagram*

(1) For sections which are partly in tension (beams or columns with large eccentricity), the simplified rectangular stress block shown in Fig. 4.3 may be used.

4.2.4 Stress-Strain Diagram for Steel

(1) The elasto-plastic diagram shown in Fig. 4.4 may be used for ordinary steel.

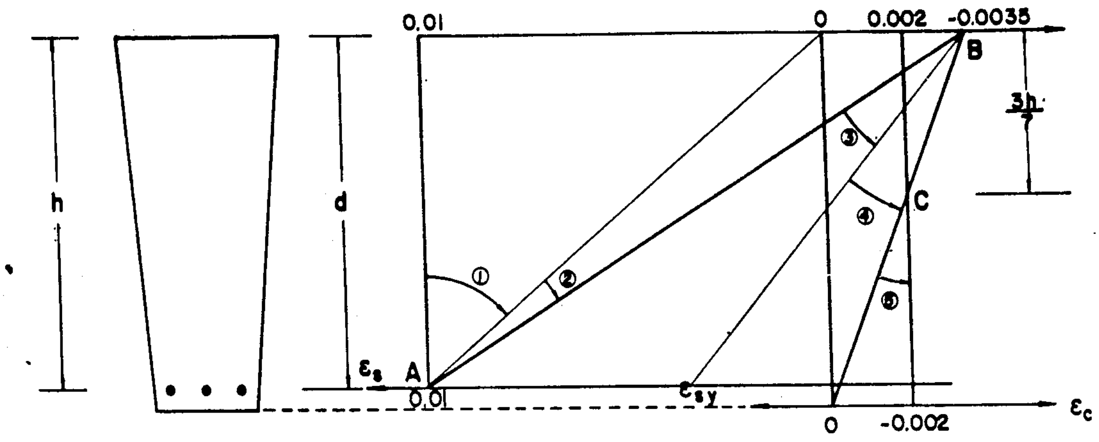


Figure 4.1 Strain Diagram in the Ultimate Limit State

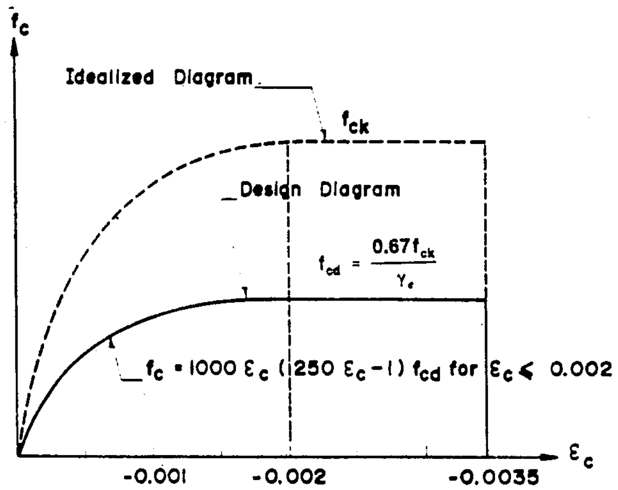


Figure 4.2 Parabolic-Rectanglar Stress-Strain Diagram for Concrete in Compression

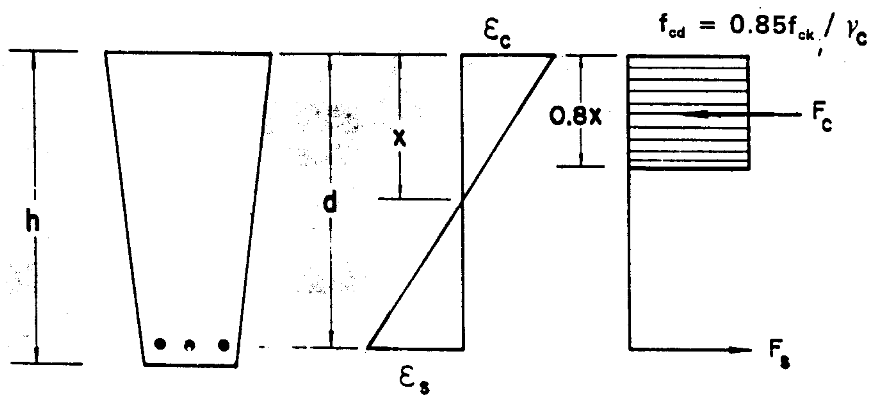


Figure 4.3 Rectangular Stress Diagram

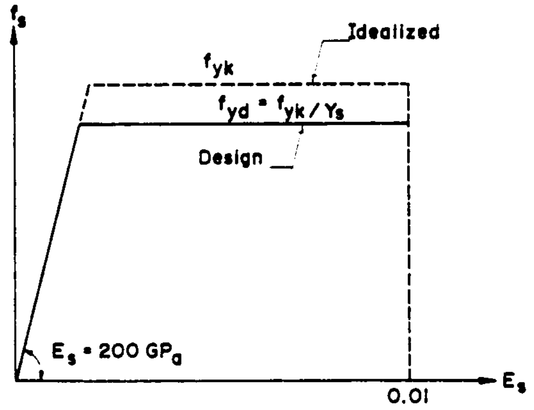


Figure 4.4 Stress-Strain Diagram for Reinforcing Steel

4.3 FLEXURAL MEMBERS

4.3.1 General

(1) Compression reinforcement in conjunction with additional tension reinforcement may be used to increase the strength of flexural members.

(2) In the analysis of a cross-section of a beam which has to resist a small axial load, the effect of the ultimate axial load may be ignored if the axial load does not exceed $0.1 f_{ck}$ times the cross-sectional area.

(3) Design of deep beams shall be in accordance with Section 6.3.

4.3.2 Distance Between Lateral Supports of Flexural Members

(1) The spacing of lateral supports for a beam shall not exceed 50 times the least width of the compression flange or face. Effects of lateral eccentricity of load shall be taken in determining the spacing of lateral supports.

4.4 COMPRESSION MEMBERS

4.4.1 Scope and Definition

(1) This section refers to slender structures or slender members mainly subjected to compression whose load carrying capacity is significantly influenced by their deformations (second-order effects).

(2) The principles given in this section apply to linear reinforced concrete members subjected to axial compression, with or without bending, for which the effects of torsion can be neglected.

(3) These principles may also be applied to other types of structural member, such as walls, shells, slender beams in which lateral buckling of the compression zone may occur, deep beams or other exceptional structures or members in which significant local deformations may arise.

(4) In compression members, the influence of second-order effects shall be considered if the increase above the first-order bending moments due to deflections exceeds 10%.

4.4.2 Analysis and Design Procedures

(1) The internal forces and moments may generally be determined by elastic global analysis using either:

(a) First-order theory, using the initial geometry of the structure, or
(b) Second-order theory, taking into account the influence of the deformation of the structure.

(2) First-order theory may be used for the global analysis in the following cases:

(a) Non-sway frames (Section 4.4.4.2)
(b) Braced frames (Section 4.4.4.3)
(c) Design methods which make indirect allowances for second-order effects.

(3) Second-order theory may be used for the global analysis in all cases.

(4) Design for structural stability taking account of second-order effects shall ensure that, for the most unfavorable combinations of actions at the ultimate limit state, loss of static equilibrium (locally or for the structure as a whole) does not occur or the resistance of individual cross-sections subjected to bending and longitudinal force is not exceeded.

(5) The structural behavior shall be considered in any direction in which failure due to second-order effects may occur.

4.4.3 Allowance for Imperfections

(1) Allowance shall be made for the uncertainties associated with the prediction of second-order effects and, in particular, dimensional inaccuracies and uncertainties in the position and line of action of the axial loads.

(2) Suitable equivalent geometric imperfections may be used, with values which reflect the possible effects of all types of imperfection.

(3) For frame structures the effects of imperfections may be allowed for in frame analysis by means of an equivalent geometric imperfection in the form of an initial sway imperfection ϕ determined in accordance with Section 3.7.3.

(4) For isolated elements, the equivalent geometric imperfections may be introduced by increasing the eccentricity of the longitudinal force by an additional eccentricity e_a, acting in the most unfavorable direction:

$$e_a = \frac{L_e}{300} \geq 20 \text{ mm} \qquad (4.1)$$

where L_e denotes the effective length of the isolated element (see Section 4.4.7).

4.4.4 Classification of Structures and Structural Elements

4.4.4.1 *General*

(1) For the purpose of design calculations, structures or structural members may be classified as braced or unbraced depending on the provision or not of bracing elements and as sway or non-sway depending on their sensitivity to second-order effects due to lateral displacements.

(2) Similarly, isolated columns are classified as slender or non-slender.

4.4.4.2 *Sway or Non-Sway Structures*

(1) A frame may be classified as non-sway if its response to in-plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal forces or moments arising from horizontal displacements of its nodes.

(2) Any other frame shall be classified as a sway frame and the effects of the horizontal displacements of its nodes shall be taken into account in its design (see Section 4.4.2).

(3) A frame may be classified as non-sway for a given load case if the critical load ratio N_{Sd}/N_{cr} for that load case satisfies the criterion:

$$N_{Sd}/N_{cr} \leq 0.1 \tag{4.2}$$

where N_{sd} is the design value of the total vertical load
N_{cr} is its critical value for failure in a sway mode (see Section 4.4.12).

(4) Beam-and-column type plane frames in building structures with beams connecting each column at each story level may be classified as non-sway for a given load case, when first-order theory is used, the horizontal displacements in each story due to the design loads (both horizontal and vertical), plus the initial sway imperfection (see Section 4.4.3) satisfy the criterion of Eq. 4.3.

$$\frac{N\delta}{HL} \leq 0.1 \tag{4.3}$$

where
- δ is the horizontal displacement at the top of the story, relative to the bottom of the story (see (5) below)
- L is the story height
- H is the total horizontal reaction at the bottom of the story
- N is the total vertical reaction at the bottom of the story.

(5) The displacement δ in (4) above shall be determined using stiffness values for beams and columns corresponding to the ultimate limit state. As an approximation, displacements calculated using moment of inertia of the gross section may be multiplied by the ratio of the gross column stiffness to the effective column stiffness in Section 4.4.12 to obtain δ.

(6) For sway frames, the requirements for frame stability given in Section 4.4.8 shall also be satisfied.

4.4.4.3 *Braced or Unbraced Structures*

(1) A frame may be classified as braced if its sway resistance is supplied by a bracing system with a response to in-plane horizontal loads which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system. This may be assumed to be the case if the frame attracts not more than 10% of the horizontal loads.

(2) A braced frame may be treated as fully supported laterally.

(3) The effects of the initial sway imperfections (see Section 4.4.3(3)) in the braced frame shall be taken into account in the design of the bracing system.

(4) The initial sway imperfections (or the equivalent horizontal forces), plus any horizontal loads applied to a braced frame, may be treated as affecting only the bracing system.

(5) The bracing system shall be designed to resist:

- (a) Any horizontal loads applied to the frames which it braces.
- (b) Any horizontal or vertical loads applied directly to the bracing system.
- (c) The effects of the initial sway imperfections (or the equivalent horizontal forces) from the bracing system itself and from all the frames which it braces.

(6) Where the bracing system is a frame or sub-frame, it may itself be either sway or non-sway (see Section 4.4.4.2.)

(7) When applying the criterion given in Section 4.4.4.2(3) to a frame or sub-frame acting as a bracing system, the total vertical load acting an all the frames which it braces shall also be included.

(8) When applying the criterion given in Section 4.4.4.2(4) to a frame or sub-frame acting as a bracing system, the total horizontal and vertical load acting on all the frames which it braces shall also be included, plus the initial sway imperfection applied in the form of the equivalent horizontal forces from the bracing system itself and from all the frames which it braces.

4.4.4.4 *Isolated Columns*

(1) Columns may be considered as isolated columns when they are isolated compression members (such as individual isolated columns and columns with articulations in a non-sway structure), or compression members which are integral parts of a structure but which are considered to be isolated for design purposes (such as slender bracing elements considered as isolated columns, and columns with restrained ends in a non-sway structure).

4.4.5 Definition of Slenderness Ratio

(1) For isolated columns, the slenderness ratio is defined by:

$$\lambda = \frac{L_e}{i} \tag{4.4}$$

where L_e is the effective buckling length
i is the minimum radius of gyration of the concrete section only.

(2) For multistory sway frames comprising rectangular subframes, the following expression may be used to calculate the slenderness ratio of the columns in the same story:

$$\lambda = \sqrt{\frac{12A}{K_l L}} \tag{4.5}$$

where A is the sum of the cross-sectional areas of all the columns of the story
K_l is the total lateral stiffness of the columns of the story (story rigidity), with modulus of elasticity taken as unity
L is the story height

4.4.6 Limits of Slenderness

(1) The slenderness ratio of concrete columns shall not exceed 140

(2) Second-order effects in compressive members need not be taken into account in the following cases:
(a) For sway frames, the greater of Eq. 4.6a or 4.6b

$$\lambda \leq 25 \tag{4.6a}$$

$$\lambda \leq \frac{15}{\sqrt{\nu_d}} \tag{4.6b}$$

(b) For non-sway frames

$$\lambda \leq 50 - 25\frac{(M_1)}{(M_2)} \tag{4.7}$$

where M_1 and M_2 are the first-order (calculated) moments at the ends, M_2 being always positive and greater in magnitude than M_1, and M_1 being positive if member is bent in single curvature and negative if bent in double curvature
ν_d $= N_{sd}/f_{cd}A_c$

4.4.7 *Effective Buckling Length of Compression Members*

(1) The effective buckling length L_e of a column in a given plane may be obtained from the following approximate equations provided the restriction in (3) below is complied with:

(a) Non-sway mode

$$\frac{L_e}{L} = \frac{\alpha_m + 0.4}{\alpha_m + 0.8} \geq 0.7 \tag{4.8}$$

(b) Sway mode

$$\frac{L_e}{L} = \sqrt{\frac{7.5 + 4(\alpha_1 + \alpha_2) + 1.6\alpha_1\alpha_2}{7.5 + \alpha_1 + \alpha_2}} \geq 1.15 \tag{4.9}$$

or conservatively

$$\frac{L_e}{L} = \sqrt{1 + 0.8\alpha_m} \geq 1.15 \tag{4.10}$$

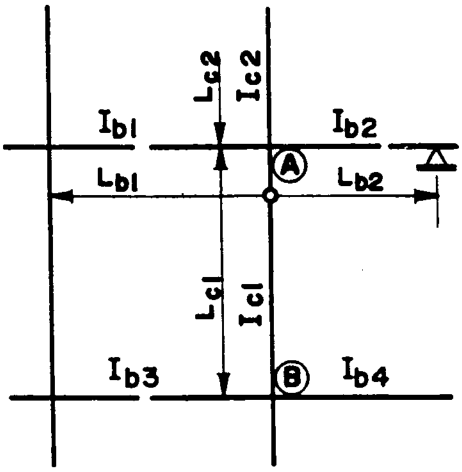
(2) For the theoretical model shown in Fig. 4.5, the stiffness coefficients α_1 and α_2 are obtained from

$$\alpha_1 = \frac{K_1 + K_c}{K_{11} + K_{12}} \tag{4.11}$$

$$\alpha_2 = \frac{K_2 + K_c}{K_{21} + K_{22}} \tag{4.12}$$

$$\alpha_m = \frac{\alpha_1 + \alpha_2}{2} \tag{4.13}$$

where K_1 and K_2 are column stiffness coefficients (EI/L)
K_c is the stiffness coefficient (EI/L) of the column being designed
K_{ij} is the effective beam stiffness coefficient (EI/L)
= 1.0 opposite end elastically or rigidly restrained
= 0.5 opposite end free to rotate
= 0 for a cantilever beam



EXAMPLE

Calculation of α_A in A

$$\alpha_A = \frac{I_{c1}/L_{c1} + I_{c2}/L_{c2}}{I_{b1}/L_{b1} + 0.5I_{b2}/L_{b2}}$$

for E_{cm} = constant

Figure 4.5 Model for Computation of Stiffness Coefficient

(3) The above approximate equations for effective length calculation are applicable for values of α_1 or α_2 not exceeding 10. For higher values more accurate methods must be used.

(4) When calculating α, only members properly framed into the end of the column in the appropriate plane of bending shall be considered. The stiffness of each member shall be obtained by dividing the second moment of area of its concrete section by its actual length.

(5) When the connection between a column and its base is not designed to resist other than nominal moment α at such positions shall be taken as 10. If a base is designed to resist the column moment, α may be taken as 1.0.

(6) For flats slab construction, an equivalent beam shall be taken as having the width and thickness of the slab forming the column strip.

4.4.8 Frame stability

4.4.8.1 *General*

(1) All frames shall have adequate resistance to failure in a sway mode, (see Section 4.4.11). However, where the frame is shown to be a non-sway frame (see Section 4.4.4.2), no further sway mode verification is required.

(2) All frames including sway frames shall also be checked for adequate resistance to failure in non-sway modes (see Section 4.4.9).

4.4.8.2 *Analysis of Sway Frames*

(1) When global analysis is used, the second-order effects in the sway mode shall be included, either directly by using second-order elastic analysis, or indirectly by using first-order analysis with amplified sway moments (see Section 4.4.11).

(2) When second-order elastic global analysis is used, the resulting forces and moments may directly be used for member design.

(3) When first-order elastic analysis, with second-order moments is used for column design, the sway moments in the beams and the beam-to-column connections shall be amplified by at least 1.2 unless a smaller value is shown to be adequate by analysis.

4.4.9 Design of Non-Sway Frames

(1) Individual non-sway compression members shall be considered to be isolated elements and be designed accordingly.

(2) Bracing elements, or in non-sway frames without bracing elements, the individual compression members shall be designed for the relevant horizontal forces and vertical loads taking account of the equivalent geometric imperfections defined in Section 3.7.3 and 4.4.3, respectively.

(3) For individual compression members, the design rules for isolated columns (Section 4.4.10) apply. The effective buckling length L_e may generally be determined according to Section 4.4.7.

4.4.10 Design of Isolated Columns

4.4.10.1 *General*

(1) For buildings, a design method may be used which assumes the compression members to be isolated and adopts a simplified shape for the deformed axis of the column. The additional eccentricity induced in the column by its deflection is then calculated as a function of slenderness ratio.

4.4.10.2 *Total eccentricity*

(1) The total eccentricity to be used for the design of columns of constant cross-section at the critical section is given by:

$$e_{tot} = e_e + e_a + e_2 \tag{4.14}$$

where e_e is equivalent constant first-order eccentricity of the design axial load, see (2) and (3) below
e_a is the additional eccentricity according to Eq 4.1
e_2 is the second-order eccentricity (Section 4.4.10.3).

(2) For first-order eccentricity e_o equal at both ends of a column,

$$e_e = e_0 \tag{4.15}$$

(3) For first-order moments varying linearly along the length, the equivalent eccentricity is the higher of the following two values:

$$e_e = 0.6e_{02} + 0.4e_{01} \tag{4.16a}$$

$$e_e = 0.4e_{02} \tag{4.16b}$$

where e_{01} and e_{02} are the first-order eccentricities at the ends, e_{02} being positive and greater in magnitude than e_{01}.

(4) For different eccentricities at the ends, (3) above, the critical end section shall be checked for first-order moments:

$$e_{tot} = e_{02} + e_a \tag{4.17}$$

4.4.10.3 *Second-Order Eccentricity*

(1) For non-sway frames, the second-order eccentricity e_2 of an isolated column may be obtained as

$$e_2 = \frac{k_1 L_e^2}{10}(1/r) \tag{4.18}$$

where L_e is the effective buckling length of the column
$k_1 = \lambda/20 - 0.75$ for $15 \le \lambda \le 35$
$k_1 = 1.0$ for $\lambda > 35$
$1/r$ is the curvature at the critical section, see (2) below.

(2) The curvature is generally a non-linear function of the axial load and bending moment in the critical section, but the following approximate value may be used in the absence of more accurate methods:

$$\frac{1}{r} = k_2\left(\frac{5}{d}\right)10^{-3} \tag{4.19}$$

where d is the column dimension in the buckling plane less the cover to the center of the longitudinal reinforcement
k_2 $= M_d/M_{bal}$
M_d is the design moment at the critical section including second-order effects
M_{bal} is the balanced moment capacity of the column.

(3) The appropriate value of k_2 may be found iteratively taking an initial value corresponding to first-order actions.

4.4.11 *Amplified Sway Moments Method for Sway Frames*

(1) In the amplified sway moments method, the sway moments found by a first-order analysis shall be increased by multiplying them by the moment magnification factor:

$$\delta_s = \frac{1}{1 - N_{Sd}/N_{cr}} \tag{4.20}$$

where N_{Sd} is the design value of the total vertical load
N_{cr} is its critical value for failure in a sway mode.

(2) The amplified sway moments method shall not be used when the critical load ratio N_{Sd}/N_{cr} is more than 0.25.

(3) Sway moments are those associated with the horizontal translation of the top of a story relative to the bottom of that story. They arise from horizontal loading and may also arise from vertical loading if either the structure or the loading is asymmetrical.

(4) As an alternative to determining N_{Sd}/N_{cr} direct, the following approximation may be used in beam-and-column type frames as described in 4.4.4.2(4):

$$\frac{N_{Sd}}{N_{cr}} = \frac{N\delta}{HL} \tag{4.21}$$

where δ, L, H and N are as defined 4.4.4.2(4).

(5) In the presence of torsional eccentricity in any floor of a structure, unless more accurate methods are used, the sway moments due to torsion should be increased by multiplying them by the larger moment magnification factor δ_s obtained for the two orthogonal directions of the lateral loads acting on the structure.

4.4.12 *Determination of Story Buckling Load N_{cr}*

(1) Unless more accurate methods are used, the buckling load of a story may be assumed to be equal to that of the substitute beam-column frame defined in Fig 4.6 and may be determined as:

$$N_{cr} = \frac{\pi^2 EI_e}{L_e^2} \tag{4.22}$$

where EI_e is the effective stiffness of the substitute column designed in accordance with (4) below

L_e is the effective length

(2) In lieu of a more accurate determination, the effective stiffness of a column EI_e in Eq 4.22 may be taken as:

$$EI_e = 0.2E_cI_c + E_sI_s \tag{4.23}$$

where

E_c = $1100f_{cd}$

E_s is the modulus of elasticity of steel

I_c, I_s are the moments of inertia of the concrete and reinforcement sections, respectively, of the substitute column, with respect to the centroid of the concrete section (see Fig 4.6(c)).

or alternatively

$$EI_e = \frac{M_{bal}}{(1/r_{bal})} \geq 0.4E_cI_c \tag{4.24}$$

where M_{bal} is the balanced moment capacity of the substitute column

$(1/r_{bal})$ is the curvature at balanced load and may be taken as

$$\frac{1}{r_{bal}} = (\frac{5}{d})10^{-3} \tag{4.25}$$

(3) In Eq 4.22 L_e may be determined in accordance with Section 4.4.7 using the stiffness properties of the gross concrete section for both beams and columns of the substitute frame (Fig. 4.6(b)).

(4) The equivalent reinforcement areas, $A_{s,tot}$, in the substitute column (see Fig. 4.6(c)) to be used for calculating I_s and M_{bal} in (2) above may be obtained by designing the substitute column at each floor level to carry the story design axial load and amplified sway moment at the critical section (see Section 4.4.11). The equivalent column dimensions of the substitute column may be taken as shown in Fig. 4.6(c), in the case of rectangular columns. Circular columns may be replaced by square columns of the same cross-sectional area. In the above, concrete cover and bar arrangement in the substitute columns shall be taken to be the same as those of the actual columns.

(5) The amplified sway moment, to be used for the design of the substitute column (see (4) above), may be found iteratively taking the first-order design moment in the substitute column as an initial value.

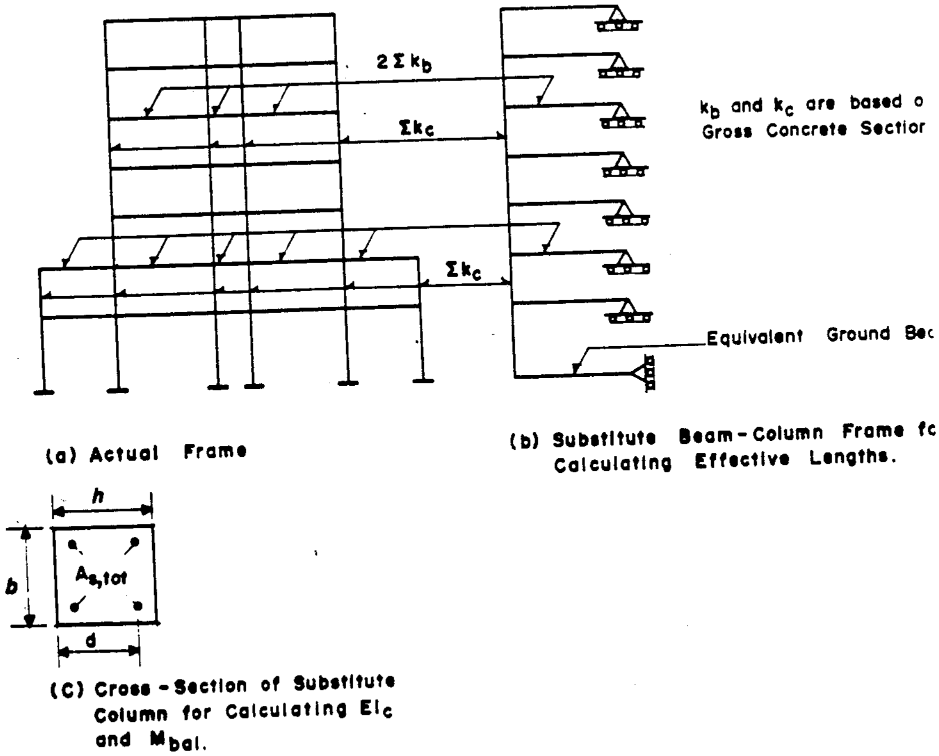


Figure 4.6 Substitute Multi-Story Beam-Column Frame

(6) In lieu of more accurate determination, the first-order design moment, M_{d1}, at the critical section of the substitute column may be determined using Eq. 4.26.

$$M_{dl} = \frac{\alpha_2 + 3}{\alpha_l + \alpha_2 + 6} HL \tag{4.26}$$

where α_1 and α_2 are defined in Section 4.4.7 and shall not exceed 10.

4.4.13 Effect of Creep

(1) Creep effects may be ignored if the increase in the first-order bending moments due to creep deformation and longitudinal force does not exceed 10%.

(2) In non-sway buildings, creep deformation of slender compression members connected monolithically to slabs or beams at their two ends may normally be disregarded because their effects are generally compensated by other influences which are neglected in the design. In interior columns, the restraints at the column ends reduce the creep deformations significantly so that they can be neglected. In edge columns with different eccentricities at each end, creep increases the deformations but it does not decrease the bearing capacity because these deformations are not additional to the critical column deflections in the relevant failure state.

(3) For isolated columns in non-sway structures, creep may be allowed for by multiplying the curvature for short-term loads (Eq. 4.19) by $(1 + \beta_d)$, where β_d is the ratio of dead load design moment to total design moment, always taken as positive.

(4) For sway frames, the effective column stiffness (Eq. 4.23 or 4.24) may be divided by $(1 + \beta_d)$, where β_d is as in (3) above.

4.4.14 Slender Columns Bent About the Major Axis

(1) A slender column bent about its major axis may be treated as biaxially loaded with initial eccentricity e_a acting about the minor axis.

4.4.15 Biaxial Bending of Columns

4.4.15.1 *Small Ratios of Relative Eccentricity*

(1) Columns of rectangular cross-section which are subjected to biaxial bending may be checked separately for uniaxial bending in each respective direction provided the relative eccentricities are such that $k \leq 0.2$; where k denotes the ratio of the smaller relative eccentricity to the larger relative eccentricity.

(2) The relative eccentricity, for a given direction, is defined as the ratio of the total eccentricity, allowing for initial eccentricity and second-order effects in that direction, to the column width in the same direction.

4.4.15.2 *Overlapping Buckling Curves*

(1) Separate checks as in Section 4.4.15.1 is equally applicable to biaxial bending in general, provided the central one-third parts of the effective lengths of the buckled column in the principal directions do not overlap (see Fig. 4.7).

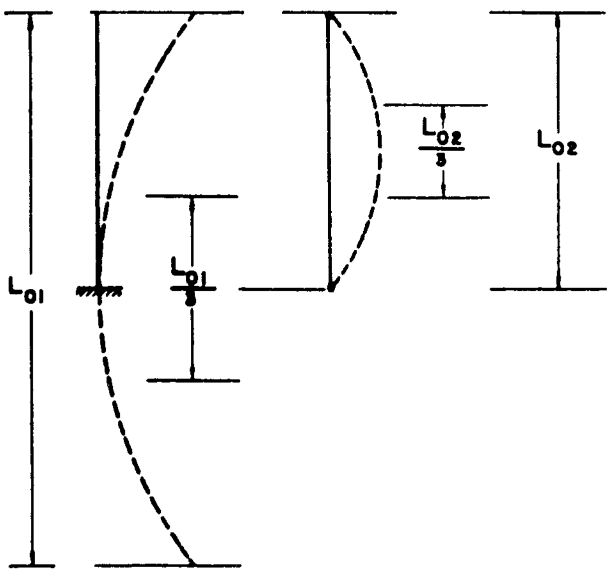


Figure 4.7 Buckling Curves for Bending in each of the Two Principal Directions

4.4.15.3 *Approximate Method*

(1) If neither of the conditions in Sections 4.4.15.1 and 4.4.15.2 is satisfied, then the approximate method of calculation given in this section may be adopted, in the absence of more accurate methods.

(2) For this approximate method, one-fourth of the total reinforcement must either be distributed along each face of the column or at each corner. The column shall be designed for uniaxial bending with the following equivalent uniaxial eccentricity of load, e_{eq}, along the axis parallel to the larger relative eccentricity:

$$e_{eq} = e_{tot}(1 + k\alpha) \quad (4.27)$$

where e_{tot} denotes the total eccentricity in the direction of the larger relative eccentricity
k denotes the relative eccentricity ratio defined in Section 4.4.15.1(2)
α may be obtained from Table 4.1 as a function of the relative normal force $\nu = N_{sd}/(f_{cd}A_c)$

Table 4.1 Values of Factor α

ν	0	0.2	0.4	0.6	0.8	≥ 1.0
α	0.6	0.8	0.9	0.7	0.6	0.5

4.5 SHEAR

4.5.1 General

(1) This Section applies to beams and slabs designed for flexure in accordance with Section 4.3. It also applies to columns subjected to significant shear forces designed in accordance with Sections 4.3 and 4.4.

(2) Provisions for minimum shear reinforcement are given in Chapter 7.

(3) The ultimate limit state in shear is characterised by either diagonal compression failure of the concrete or failure of the web reinforcement due to diagonal tension.

(4) Resistance to diagonal tension is obtained as the sum of the resistances of the web reinforcement and of the concrete section.

(5) Critical section for shear is at a distance d from the face of supports. Sections closer than d shall be designed for the shear at d.

(6) Two-way action (punching) shall be considered according to Section 4.10.

4.5.2 Limiting Value of Ultimate Shear Force

(1) In order to prevent diagonal compression failure in the concrete, the shear resistance V_{Rd} of a section given by Eq. 4.28 shall not be less than the applied shear force V_{sd}.

$$V_{Rd} = 0.25 f_{cd} b_w d \quad (4.28)$$

where b_w is the minimum width of the web.

4.5.3 Shear Resistance of Concrete in Beams and Slabs

4.5.3.1 *Members Without Significant Axial Forces*

(1) The shear force V_c carried by the concrete in members without significant axial forces shall be taken as:

$$V_c = 0.25 f_{ctd}\, k_1 k_2 b_w d \tag{4.29}$$

where

$k_1 = (1 + 50\rho) \leq 2.0$

$k_2 = 1.6 - d \geq 1.0$ (d in meters). For members where more than 50% of the bottom reinforcement is curtailed, $k_2 = 1$

$\rho = A_s/b_w d$

A_s is the area of the tensile reinforcement anchored beyond the intersection of the steel and the line of a possible 45° crack starting from the edge of the section (see Fig. 4.8)

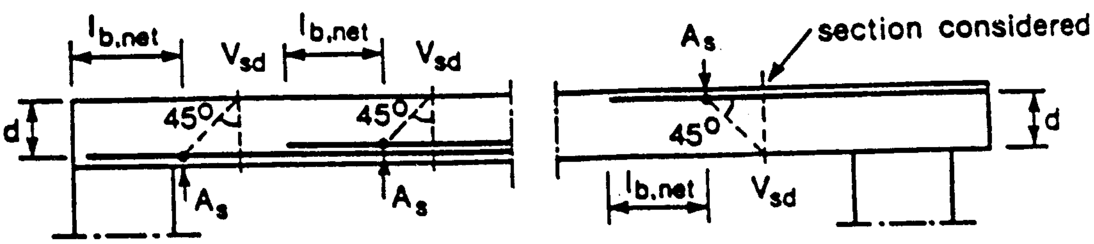


Figure 4.8 A_s to be introduced in Eq. 4.29

4.5.3.2 *Members Subjected to Significant Axial Compression*

(1) For members subjected to axial compression, Eq. 4.30 may be used to compute the additional shear force V_{cn} carried by the concrete.

$$V_{cn} = 0.10 \frac{b_w d}{A_c} N_{Sd} \tag{4.30}$$

where N_{Sd} is the design axial force

4.5.3.3 *Members Subjected to Axial Tension*

(1) For members subjected to axial tension, shear reinforcement shall be designed to carry total shear.

(2) In the case of fatigue loading, the shear reinforcement shall carry the total shear.

4.5.4 Design of Shear Reinforcement

(1) In beams, bent-up bars shall not be used as shear reinforcements except in combination with stirrups. At least 50% of the design shear force V_{Sd} shall be resisted by vertical stirrups.

(2) Where inclined shear reinforcement is used, the angle between the reinforcement and the longitudinal axis of the beam shall not be less than 45°.

(3) Where the load is not acting at the top of the beam or when the support is not at the bottom of the beam suspension reinforcement shall be provided to transfer the load to the top of the beam.

(4) When shear reinforcement perpendicular to the longitudinal axis is used, its shear resistance V_s may be calculated as:

$$V_s = \frac{A_v d f_{yd}}{s} \tag{4.31}$$

where A_v is the area of shear reinforcement within distance s.

(5) When inclined stirrups are used, the shear resistance of the stirrups may be calculated as:

$$V_s = \frac{A_v d f_{yd}(\sin\alpha + \cos\alpha)}{s} \tag{4.32}$$

where α is the angle of inclination from the horizontal.

(6) When shear reinforcement consists of a single bar or a single group of parallel bars, all bent up at the same distance from the support, the shear resistance of the reinforcement may be calculated as:

$$V_s = A_v f_{yd} \sin\alpha \tag{4.33}$$

4.5.5 Web-Flange Connections

4.5.5.1 *General*

(1) The shear strength of the flange may be calculated considering the flange as a system of compressive struts combined with ties in the form of tensile reinforcement.

(2) The junction of the flanges with the web shall be checked for longitudinal shear.

(3) The ultimate limit state in longitudinal shear is governed either by the effect of inclined flange compression (acting parallel to its middle plane) or by tension in the transverse reinforcement.

(4) The longitudinal shear per unit length v_{sd}, which may be obtained as a function of the applied transverse shear V_{Sd} from Eqs. 4.32 and 4.33, shall not exceed the limits of resistance given by Eqs. 4.34 and 4.35.

(5) Calculation of longitudinal shear per unit length:

(a) For flange in compression

$$v_{sd} = \left(\frac{b_e - b_w}{2b_e}\right)\frac{V_{Sd}}{z} \qquad (4.34)$$

(b) For flange in tension

$$v_{sd} = \left(\frac{A_s - A_{sw}}{2A_s}\right)\frac{V_{Sd}}{z} \qquad (4.35)$$

where b_e is the effective width of a T-section
b_w is the width of the web
z is the internal lever arm
A_s is the area of the longitudinal steel in the effective flanges outside the projection of the web into the slab
A_{sw} is the area of the longitudinal steel inside the slab within the projection of the web into the slab.

4.5.5.2 ***Resistance to Inclined Compression***

(1) The resistance to inclined compression per unit length v_{Rd1} shall be computed as

$$v_{Rd1} = 0.25 f_{cd} h_f \qquad (4.36)$$

where h_f is the total thickness of the flange.

4.5.5.3 ***Resistance to Diagonal Tension***

(1) The resistance to diagonal tension per unit length V_{Rd2} shall be computed as

$$v_{Rd2} = 0.50 f_{cd} h_f + \frac{A_{sf} f_{yd}}{s_f} \qquad (4.37)$$

where A_{sf} is the area of transverse reinforcement per unit length, perpendicular to the web-flange interface (see Fig. 4.9)

(2) If, at the section with $M = M_{max}$ (see Fig 4.9), the flange is subjected to a tensile force, the concrete contribution $0.50 f_{cd} h_f$ in Eq. 4.37 should be neglected.

(3) The cross sectional area of the transverse flexural reinforcement which crosses the interface between web and flange can be taken into account in calculating A_{sf}. If this reinforcement is not sufficient as determined from Eq. 4.37, then additional reinforcement shall be provided.

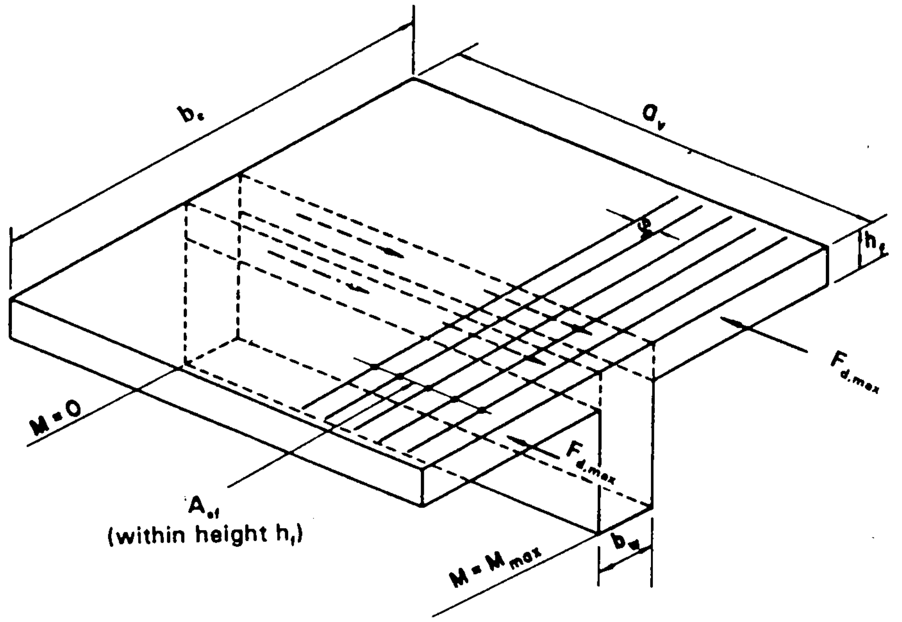


Figure 4.9 Notations for Web-Flange Connections

(4) The reinforcement crossing the plane of the junction shall be:

(a) Placed in the part of the flange subjected to tension by transverse bending if the latter is predominant.
(b) Evenly distributed between the upper and lower parts if the transverse bending is slight.

4.6 TORSION

4.6.1 Definitions

(1) *Compatibility torsion.* Torques which are due solely to the restraint of the angular rotation induced by adjacent members.

(2) *Equilibrium torsion.* Torques which are necessary for equilibrium.

4.6.2 General

(1) Torques due to compatibility torsion are not necessary for equilibrium and may be neglected in ultimate limit state calculations. However, the resulting secondary effects shall be considered in the serviceability limit states and in detailing.

(2) The torsional resistance of any section may be calculated on the basis of an equivalent hollow section with thin walls (see Fig 4.10).

(3) For T-sections and other sections which can be subdivided into rectangles, the torsional resistance may be taken as the sum of the capacities of the individual rectangular sections. The subdivision of the section may be chosen so as to maximize the calculated resistance (see Sections 4.6.3 and 4.6.4).

(4) For hollow sections, the equivalent wall thickness shall not exceed the actual wall thickness. Actual wall thickness for hollow sections that is less than twice the concrete cover to longitudinal bars is not allowed.

(5) The equivalent hollow section has the same outer boundary as the actual section and an equivalent thickness h_{ef} obtained as $h_{ef} \leq A/u \leq$ the actual wall thickness (where u is the outer perimeter and A is the total area of the cross-section enclosed by the outer perimeter, including inner hollow areas).

(6) The critical section for torque is at the face of supports.

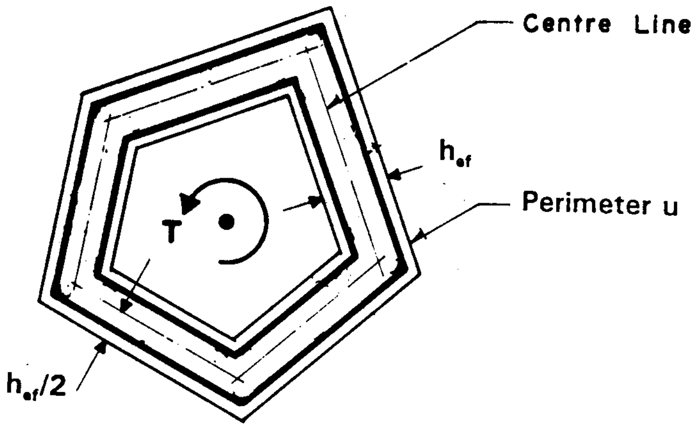


Figure 4.10 Equivalent Hollow Section

4.6.3 Limiting Value of Ultimate Torque

(1) In order to prevent diagonal compression failure in the concrete, the torsional resistance T_{Rd} of a section given by Eq. 4.38 shall not be less than the applied torque T_{Sd}.

$$T_{Rd} = 0.80 f_{cd} A_{ef} h_{ef} \tag{4.38}$$

where A_{ef} is the area enclosed within the centerline of the thin-wall cross-section including inner hollow areas (see Section 4.6.2).

4.6.4 Torsional Resistance of Concrete

(1) The torque T_c carried by the concrete shall be taken as:

$$T_c = 1.2 f_{ctd} A_{ef} h_{ef} \tag{4.39}$$

4.6.5 Design of Torsional Reinforcement

(1) Torsional reinforcement in the form of closed links and longitudinal reinforcement is required to carry the excess torque whenever the applied torque exceeds the concrete resistance given by Eq. 4.39.

(2) The volume of longitudinal torsional reinforcement shall be chosen to be equal to the volume of the links (closed stirrups).

(3) Minimum torsional reinforcement in the form of stirrups shall be provided as required in Chapter 7.

(4) The torsional resistance of the reinforcement T_{ef} is given by Eqs. 4.40 and 4.41.

$$T_{ef} = \frac{2A_{ef} f_{yd} A_s}{s} \tag{4.40}$$

or

$$T_{ef} = \frac{2A_{ef} f_{yd} A_l}{u_{ef}} \tag{4.41}$$

where A_s is the cross-sectional area of the stirrups in the effective wall
A_l is the cross-sectional area of the longitudinal reinforcement
u_{ef} is the mean perimeter enclosing the area A_{ef} (see Fig.4.9).

(2) The longitudinal reinforcement may be distributed evenly around the inside perimeter of the links or concentrated in the corners where there shall always be at least one bar.

(3) Additional requirements are given in Chapter 7.

4.6.6 Combined Action-Effects

4.6.6.1 *Torsion and Bending and/or Longitudinal Stresses.*

(1) The longitudinal reinforcement shall be determined separately for torsion according to Section 4.6.5 and for flexure and axial loads according to Chapter 4.

(2) The area of reinforcement furnished shall be the sum of the areas thus determined.

4.6.6.2 *Torsion and Shear*

(1) The limiting values of torsional and shear resistance shall be taken as the basic values from Eqs. 4.38 and 4.28, respectively multiplied by the following reduction factors β_t and β_v.

(a) torsion

$$\beta_t = \frac{1}{\sqrt{1 + \left(\frac{V_{Sd}/V_{Rd}}{T_{Rd}/T_{Sd}}\right)^2}} \tag{4.42}$$

(b) shear

$$\beta_v = \frac{1}{\sqrt{1 + \left(\frac{T_{Sd}/T_{Rd}}{V_{Sd}/V_{Rd}}\right)^2}} \tag{4.43}$$

(2) The torsional and shear resistance of the concrete shall be taken as the basic values from Eqs.4.39 and 4.29, respectively, multiplied by the reduction factors β_{tc} and β_{vc}.

(a) torsion

$$\beta_{tc} = \frac{1}{\sqrt{1 + \left(\frac{V_{Sd}/V_c}{T_{Sd}/T_c}\right)^2}} \tag{4.44}$$

(b) shear

$$\beta_{vc} = \frac{1}{\sqrt{1 + \left(\frac{T_{Sd}/T_c}{V_{Sd}/V_c}\right)^2}} \tag{4.45}$$

4.7 PUNCHING

4.7.1 General

(1) This section applies to the punching of slabs and footings that are provided with the necessary flexural reinforcement.

(2) The following requirements supplement those of Section 4.5 which must be checked to ensure adequate resistance for one-way action.

(3) The ultimate limit state in punching is characterised by the formation of a truncated punching cone or pyramid around concentrated loads or reactions.

4.7.2 Loaded Area

(1) The provisions of this section are applicable to the following types of loaded area:

(a) Shape (d denotes the average effective depth of the slab or footing):
- rectangular, with perimeter not exceeding 11d and the ratio of length to breadth not exceeding 2
- circular, with diameter not exceeding 3.5d
- any shape, with perimeter not exceeding 11d.

(b) The loaded area is not so close to other concentrated forces that their critical perimeters intersect, nor in a zone subjected to significant shear forces of a different origin.

(2) If the conditions in 1(a) above are not satisfied for wall or rectangular column supports, the critical reduced perimeters according to Fig. 4.11 shall be taken into account, since the shear forces in wall-shaped supports are concentrated in the corners.

4.7.3 Critical Section

(1) The critical section is perpendicular to the middle plane of the slab. It extends along the effective depth d and its outline is defined below.

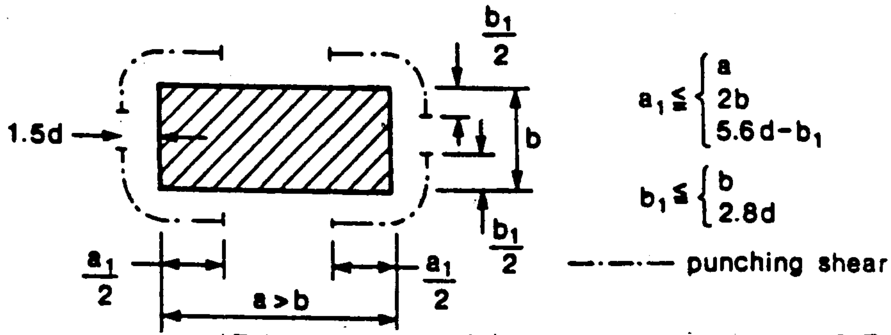


Figure 4.11 Application of Punching Provisions in Non-Standard Cases

4.7.3.1 *Loaded Area Remote from an Opening or a Free Edge*

(1) The outline of the critical section is the closed outline of the minimum perimeter surrounding the loaded area. However, it need not approach closer to the loaded area than lines located at a distance 1.5*d* from that area and parallel to its boundaries (see Fig. 4.12).

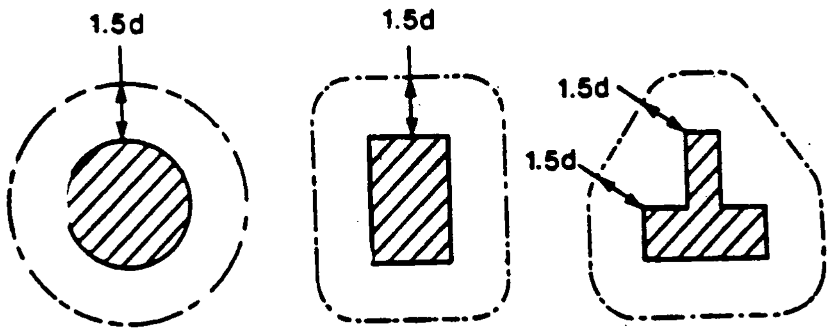


Figure 4.12 Critical Section Remote from a Free Edge

4.7.3.2 *Loaded Area Close to an Opening*

(1) When openings in slabs and footings (see Fig. 4.13) are located at a distance less than 6*d* from the edge of the concentrated load, then that part of the perimeter which is enclosed by radial projections from the centroid of the loaded area to the openings is considered ineffective.

(2) Where a single hole is adjacent to the column and its greatest width is less than one quarter of the column side or one half of the slab depth, whichever is the lesser, its presence may be ignored.

4.7.3.3 *Loaded Area Close to Free Edge*

(1) In the vicinity of a free edge certain parts of the outline defined for the case of remote opening or free edge shall be replaced by perpendicular lines to those edges if the resulting length developed in this way, excluding the free edges, is smaller than the length of the closed outline wholly enclosing the loaded area (see Fig. 4.14).

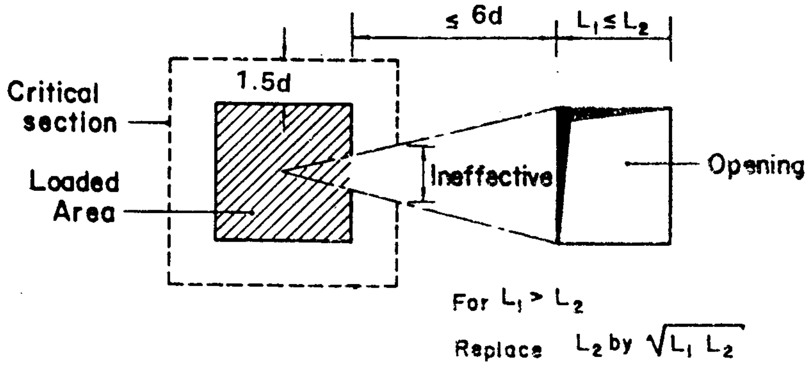


Figure 4.13 Critical Section in the Vicinity of an Opening

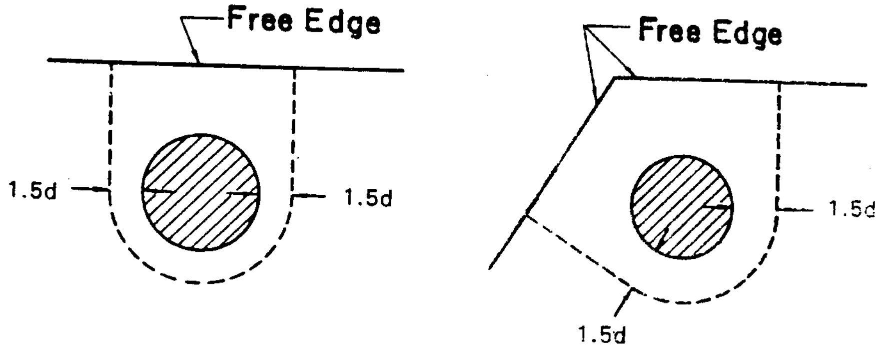


Figure 4.14 Critical Sections Near Free Edges

4.7.4 Applied Load Effect

(1) In the case of a centric load or reaction, the punching shear force V_{Sd} shall not exceed the punching shear resistance V_{Rd1} or V_{Rd2} given by Eqs. 4.36 or 4.37 as appropriate.

(2) In the case of an eccentric load or reaction, the applied load effect of the punching shear force V_{sd} with eccentricity e shall be taken to be equal to that of an equivalent centric load V_{eq} given by Eq. 4.46.

$$V_{eq} = \beta V_{Sd} \tag{4.46}$$

where $\beta = 1 + \eta e u d / Z$

e is the eccentricity of the load or reaction with respect to the centroid of the critical section, always positive

Z is the section modulus of the critical section, corresponding to the direction of the eccentricity

η denotes fraction of moment which is considered transferred by eccentricity of the shear about the centroid of the critical section

$= 1/(1 + \sqrt{(b_2/b_1)}$

b_1 and b_2 are sides of the rectangle of outline u, b_1 being parallel to the direction of the eccentricity e.

(3) Conservatively, the following values of β in (2) above may be used for flat slabs with approximately equal spans and for footings:

(a) Interior column: $\beta = 1.15$
(b) Edge column: $\beta = 1.40$
(c) Corner column: $\beta = 1.50$

4.7.5 Moment Transfer Between Slabs and Columns

(1) A fraction η of the moment is assumed to be transferred by eccentricity of the shear about the centroid of the critical section. The remaining moment shall be considered to be transferred by flexure in accordance with Appendix A.

4.7.6 Resistance of Slabs or Footings Without Punching Shear Reinforcement

(1) The punching resistance V_{Rd1} shall be given by Eq. 4.47.

$$V_{Rd1} = 0.25 f_{ctd} k_1 k_2 u d \tag{4.47}$$

where $k_1 = (1 + 50\rho) \leq 2.0$
$k_2 = 1.6 - d \geq 1.0$ (d in meters). For members where more than 50% of the bottom reinforcement is curtailed, $k_2 = 1$
$d = (d_x + d_y)/2$
$\rho_e = (\rho_{ex} + \rho_{ey})^{1/2} \leq 0.015$
ρ_{ex} and ρ_{ey} correspond to the geometric ratios longitudinal reinforcement parallel to x and y, respectively
d is the average effective height in the x and y directions

4.7.7 Resistance of Slabs or Footings with Punching Shear Reinforcement

(1) The punching resistance with punching shear reinforcement V_{Rd2} shall be given by Eq. 4.48.

$$V_{Rd2} = 1.6 V_{Rd1} \tag{4.48}$$

(2) The shear resistance of the reinforcement may be calculated using Eq.4.33, where A_v is the sum of the areas of web reinforcement within the critical perimeter.

4.7.8 Flat Slabs

(1) Flat slabs containing shear reinforcement shall have a minimum thickness of 200 mm.

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CHAPTER 5
SERVICEABILITY LIMIT STATES

5.1 SCOPE

(1) This Chapter covers the common serviceability limit states. These are deflection control and crack control.

Other limit states (such as stress or vibration) may be of importance in particular structures but these are not covered in this Code.

5.2 LIMIT STATE OF DEFLECTION

5.2.1 General

(1) The deflection of a structure or any part of the structure shall not adversely affect the proper functioning or appearance of the structure.

(2) This may be ensured either by keeping calculated deflections below the limiting values in Section 5.2.2 or by compliance with the requirements for minimum effective depth given in Section 5.2.3.

5.2.2 Limits on Deflection

(1) The final deflection (including the effects of temperature, creep and shrinkage) of all horizontal members shall not, in general, exceed the value.

$$\delta = \frac{L_e}{200} \tag{5.1}$$

where, L_e = the effective span

(2) For roof or floor construction supporting or attached to nonstructural elements (e.g partitions and finishes) likely to be damaged by large deflections, that part of the deflection which occurs after the attachment of the non-structural elements shall not exceed the value.

$$\delta = \frac{L}{350} \leq 20 \text{ mm} \tag{5.2}$$

(3) In any calculation of deflections, the design properties of the materials and the design loads shall be those defined in Sections 3.4 and 3.5 as appropriate for a serviceability limit state.

5.2.3 Requirements for Effective Depth

(1) The minimum effective depth obtained from Eq. 5.3 shall be provided unless computation of deflection indicates that smaller thickness may be used without exceeding the limits stipulated in Section 5.2.2.

$$d = (0.4 + 0.6\frac{f_{yk}}{400})\frac{L_e}{\beta_a} \tag{5.3}$$

where f_{yk} is the characteristic strength of the reinforcement (MPA).
L_e is the effective span; and, for two-way slabs, the shorter span.
β_a is the appropriate constant from Table 5.1, and for slabs carrying partition walls likely to crack, shall be taken as $\beta_a \leq 150/L_o$
L_o is the distance in meter between points of zero moments; and for a cantilever, twice the length to the face of the support.

Table 5.1 Values of β_a (Eq. 5.3)

Member	Simply Supported	End Spans	Interior Spans	Cantilevers
Beams	20	24	28	10
Slabs (a) Span ratio = 2:1 (b) Span ratio = 1:1	 25 35	 30 40	 35 45	 12 10
Flat slabs (based on longer span)	24			-

Note: For slabs with intermediate span ratios interpolate linearly.

5.2.4 Calculation of Deflections

(1) When calculating deflections, the effect of creep and shrinkage strains on the curvature, and thereby on the deflection, shall be considered.

5.2.4.1 *Immediate Deflections*

(1) Unless values are obtained by a more comprehensive analysis, deflections which occur immediately on application of load shall be computed by the usual elastic methods as the sum of the two parts δ_i and δ_{ii} given by Eqs. 5.4 and 5.5, but not more than δ_{max} given by Eq. 5.6.

$$\delta_i = \beta L^2 \frac{M_{cr}}{E_{cm} I_i} \tag{5.4}$$

$$\delta_{ii} = \beta L^2 \frac{M_k - M_{cr}}{0.75 E_s A_s z(d-x)} \tag{5.5}$$

$$\delta_{max} = \beta L^2 \frac{M_k}{E_s A_s z(d-x)} \tag{5.6}$$

(2) Unless the theoretical moment which causes cracking is obtained by a more comprehensive method, it shall be computed by

$$M_{cr} = 1.70 f_{ctk} Z \tag{5.7}$$

where δ_i is the deflection due to the theoretical cracking moment M_{cr} acting on the uncracked transformed section.
δ_{ii} is the deflection due to the balance of the applied moment over and above the cracking value and acting on a section with an equivalent stiffness of 75% of the cracked value.
δ_{max} is the deflection of fully cracked section
A_s is the area of the tension reinforcement
E_{cm} is the short term elastic modulus (tangent modulus) of the concrete (Table 2.5).
E_s is the modulus of elasticity of steel
I_i is the moment of inertia of the uncracked transformed concrete section
M_k is the maximum applied moment at mid-span due to sustained characteristic loads; for cantilevers M_k is the moment at the face of the support
Z is the section modulus
d is the effective depth of the section
x is the neutral axis depth at the section of maximum moment
z is the internal lever arm at the section of maximum moment
β is the deflection coefficient depending on the loading and support conditions (e.g. $\beta = 5/48$ for simply supported span subjected to uniformly distributed load).

Note: The value of x and z may be determined for the service load condition using a modular ratio of 10, or for the ultimate load condition.

5.2.4.2 *Long Term Deflections*

(1) Unless values are obtained by more comprehensive analysis, the additional long-term deflection of flexural members shall be obtained by multiplying the immediate deflection caused by the sustained load considered, computed in accordance with Section 5.2.4.1, by the factor

$$[2 - 1.2A_s'/A_s] \geq 0.6 \tag{5.8}$$

where A_s' is the area of compression reinforcement
A_s is the area of tension reinforcement

5.3 LIMIT STATES OF CRACKING

5.3.1 General

(1) For reinforced concrete, two limit states of cracking: the limit state of crack formation and the limit state of crack widths are of interest.

(2) The particular limit state to be checked is chosen on the basis of the requirements for durability and appearance. The requirements for durability depend on the conditions of exposure and the sensitivity of the reinforcement to corrosion.

5.3.2 Minimum Reinforcement Areas

(1) In assessing the minimum area of reinforcement required to ensure controlled cracking in a member or part of a member which may be subjected to tensile stress due to the restraint of imposed deformations, it is necessary to distinguish between two possible mechanisms by which such stress may arise. The two mechanisms are:

(a) Restraint of intrinsic imposed deformations - where stresses are generated in a member due to dimensional changes of the member considered being restrained (for example stress induced in a member due to restraint to shrinkage of the member).
(b) Restraint of extrinsic imposed deformations - where the stresses are generated in the member considered by its resistance to externally applied deformations (for example where a member is stressed due to settlement of a support).

(2) It is also necessary to distinguish between two basic types of stress distribution within the member at the onset of cracking. These are:

(a) **Bending** - where the tensile stress distribution within the section is triangular (i.e. some part of the section remains in compression).
(b) **Tension** - where the whole of the section is subject to tensile stress.

(3) Unless more rigorous calculation shows a lesser area to be adequate, the required minimum areas of reinforcement may be calculated from the relation given be Eq. 5.9.

$$A_s = k_c k f_{ct.ef} A_{ct} / \sigma_s \qquad (5.9)$$

where A_s is the area of reinforcement

A_{ct} is the area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack.

σ_s is the maximum stress permitted in the reinforcement immediately after formation of the crack. This may be taken as 100% of the yield strength of the reinforcement, f_{yk}. A lower value may, however, be needed to satisfy the crack with limits

$f_{ct.ef}$ is the tensile strength of the concrete effective at the time when the cracks may first be expected to occur. In many cases, such as where the dominant imposed deformation arises from dissipation of the heat of hydration, this may be within 3-5 days from casting depending on the environmental conditions, the shape of the member and the nature of the formwork. When the time of cracking cannot be established with confidence as being less than 28 days, it is suggested that a minimum tensile strength of 3 MPa be adopted.

k_c is a coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking. The stress distribution is that resulting from the combination of effects of loading and restrained imposed deformation.
= 1.0 for pure tension
= 0.4 for bending without normal compressive force

k is a coefficient which allows for the effect of non-uniform self-equilibrating stresses

Values of k for various situations are given below:

(a) tensile stresses due to restraint of intrinsic deformations generally $k = 0.8$
for rectangular sections when $h \leq 300$ mm, $k = 0.8$
$h \geq 800$ mm $k = 0.5$
(b) tensile stresses due to restraint of extrinsic deformations $k = 1.0$.

Parts of sections distant from the main tension reinforcement, such as outstanding parts of a section or the webs of deep sections, may be considered to be subjected to imposed deformations by the tension chord of the member. For such cases, a value in the range $0.5 < k < 1.0$ will be appropriate.

(4) The minimum reinforcement may be reduced or even be dispensed with altogether if the imposed deformation is sufficiently small that it is unlikely to cause cracking. In such cases minimum reinforcement need only be provided to resist the tensions due to the restraint.

5.3.3 Limit State of Crack Formation

(1) The maximum tensile stresses in the concrete are calculated under the action of design loads appropriate to a serviceability limit state and on the basis of the geometrical properties of the transformed uncracked concrete cross section.

(2) The calculated stresses shall not exceed the following values:

(a) Flexure

$$\sigma_{ct} = 1.70 f_{ctk} \tag{5.10}$$

(b) Direct tension

$$\sigma_{ct} = f_{ctk} \tag{5.11}$$

(3) In addition to the above, minimum reinforcement in accordance with Chapter 7 shall be provided for the control of cracking.

5.3.4 Limit State of Crack Widths

5.3.4.1 *General*

(1) Adequate protection against corrosion may be assumed provided that the minimum concrete covers in Section 7.1.3 are complied with and provided further that the characteristic crack widths w_k do not exceed the limiting values given in Table 5.2 appropriate to the different conditions of exposure.

Table 5.2 Characteristic Crack Width for Concrete Members

Type of exposure	Dry environment: Interior of buildings of normal habitation or offices (Mild)	Humid environment: Interior components (e.g. laundries); exterior components; components in non-aggressive soil and/or water (Moderate)	Seawater and/or aggressive chemical environment: Components completely or partially submerged in seawater; components in saturated salt air; aggressive industrial atmospheres (Severe)
Characteristic crack width, w_k (mm)	0.4	0.2	0.1

5.3.4.2 *Cracks due to Flexure*

(1) Checking of the limit state of flexural crack widths is generally not necessary for reinforced concrete where

(a) at least the minimum reinforcement given by Section 5.3.2 is provided
(b) the reinforcement consists of deformed bars, and
(c) their diameter does not exceed the maximum values in Table 5.3.

Table 5.3 Maximum Bar Diameter for which Checking Flexural Crack Width may be Omitted

w_k = 0.4 mm		w_k = 0.2 mm	
σ_s (MPa)	ϕ (mm)	σ_s (MPa)	ϕ (mm)
160	40	160	25
200	32	200	16
240	25	240	12
280	20	320	6
320	16	400	4

Note: Where necessary linear interpolation may be used.

In Table 5.2

σ_s is the steel stress under service condition

w_k is the permitted characteristic crack width

(2) If crack widths have to be calculated, the following approximate equations may be used in the absence of more accurate methods

$$w_k = 1.7 w_m \tag{5.12}$$

$$w_m = s_{rm} \epsilon_{sm} \tag{5.13}$$

where w_k is the characteristic crack width

w_m is the mean crack width

s_{rm} is the average distance between cracks

ϵ_{sm} is the mean strain of the reinforcement considering the contribution of concrete in tension.

(3) The average distance between cracks may be obtained from

$$s_{rm} = 50 + 0.25 \kappa_1 \kappa_2 \frac{\phi}{\rho_r} \tag{5.14}$$

where ϕ is the bar diameter

κ_1 is a coefficient which characterizes the bond properties of the bars

$\kappa_1 = 0.8$ for deformed bars

$\kappa_1 = 1.6$ for plain bars

κ_2 is a coefficient representing the influence of the form of the stress diagram

$\kappa_2 = 0.50$ for bending

$\kappa_2 = 1.00$ for pure tension

$\kappa_2 = (\epsilon_1 + \epsilon_2)/2\epsilon_1$ for bending with tension

ϵ_1, ϵ_2 are the larger and the smaller concrete strains, respectively, below the neutral axis of the cracked section given in Fig. 5.1.

In Eq. 5.14, the coefficient ρ_r is defined as

$$\rho_r = \frac{A_s}{A_{c,ef}} \tag{5.15}$$

where A_s is the area of the reinforcement contained in $A_{c,ef}$

$A_{c,ef}$ is the section of the zone of the concrete (effective embedment zone) where the reinforcing bars can effectively influence the crack widths shown by the shaded area in Fig. 5.1.

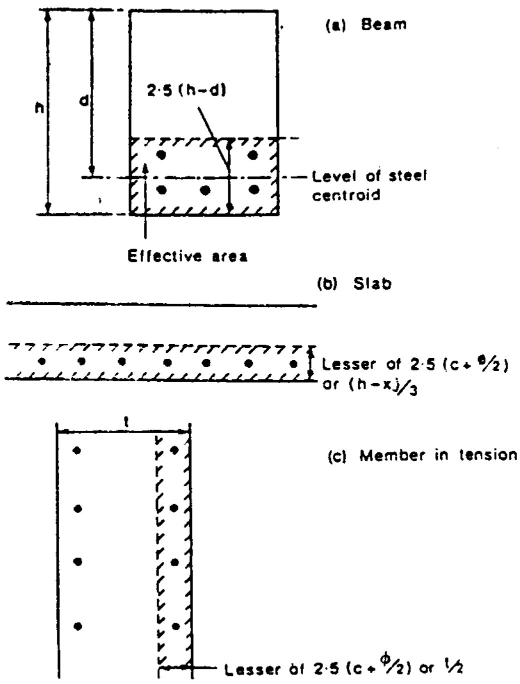

Figure 5.1 Definition of $A_{c,ef}$

(4) The mean strain of the reinforcement may be obtained as

$$\epsilon_{sm} = \frac{\sigma_s}{E_s}\left[1 - \beta_1\beta_2\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2\right] \geq 0.4\frac{\sigma_s}{E_s} \tag{5.16}$$

where σ_s is the service stress in the steel and may be obtained by elastic theory using modular ratio equal to 10

σ_{sr} is the steel stress at rupture of concrete section; i.e., stress for the cracked section under the action of the theoretical moment M_{cr} defined in Section 5.2.4.1

β_1 is a coefficient which characterizes the bond properties of the bars and is equal to

β_1 = 1.0 for high bond bars

β_2 = 0.5 for plain bars

β_2 is a coefficient representing the influence of the duration of the application or repetition of the loads.

β_2 = 1.0 at the first loading

β_2 = 0.5 for sustained loads or for a large number of load cycles

5.3.4.3 *Cracking due to Shear*

(1) Checking of shear crack widths is not necessary in slabs and in the web of beams if the spacing of the stirrups does not exceed the values given in Table 5.4.

Table 5.4 Maximum Spacing (mm) of Vertical Stirrups for which Checking of Shear Crack Width is Omitted

w_k (mm)	0.4				0.2			
f_{yd} (MPa)	220	400	360	500	220	400	360	500
Bond Properties	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
$V_{sd} \leq V_c$	300		250		200		150	
$V_c < V_{sd} \leq 3V_c$	250		200		150		100	
$V_{sd} > 3V_c$	200		150		100		75	

(1) Plain bars (2) High bond bars

In Table 5.4,

w_k is the permitted characteristic crack width
V_{sd} is the shear acting during the combination under consideration
V_c is the shear resistance of concrete; Eqs. 4.29 and 4.30.

(2) If more precise data are available, then the widths of the shear cracks in the webs of beams can be calculated for sustained loads by means of Eq. 5.13 together with the following equations:

$$w_k = 1.7\kappa_w w_m \tag{5.17}$$

$$s_{rm} = 50 + 0.25\kappa_1\kappa_2\frac{\phi}{\rho_r} \leq \frac{d - x}{\sin\alpha} \tag{5.18}$$

$$\epsilon_{sm} = \frac{\sigma_s}{E_s}[1 - (\frac{V_c}{V_{sd}})^2] \geq 0.4\frac{\sigma_s}{E_s} \tag{5.19}$$

$$\sigma_s = \frac{V_{sd} - V_c}{b_w d\rho_w} \cdot \frac{1}{(\sin\alpha + \cos\alpha)} \geq 40 \text{ MPa} \tag{5.20}$$

where w_m is the mean crack width (see Eq. 5.12)
α is the angle of inclination of the stirrup from the horizontal
κ_w is a correction coefficient to take account of the effect of slope of the stirrups on the spacing of the cracks.
$\kappa_w = 1.2$ for vertical stirrups ($\alpha = 90°$)
$\kappa_w = 0.8$ for inclined stirrups with $\alpha = 45°$ to $60°$
ρ_w is the geometric percentage of web reinforcement
x is the height of the compression zone in the cracked section

(3) When several adjacent bars in the same layer are bent in the same zone (for example, at the corners of a frame), the diameter of mandrel shall be chosen with a view to avoiding crushing or splitting of the concrete under the effect of the pressure that occurs inside the bend (see Eq. 7.7).

CHAPTER 6
SPECIAL STRUCTURAL ELEMENTS

6.1 SCOPE

(1) This chapter gives methods of analysis and design of special structural elements that in general ensure that the objectives set out in Chapter 3 are met.

(2) Other methods may be used provided they can be shown to be satisfactory for the types of structure or member considered.

(3) It is assumed that the ultimate limit state is the critical limit state.

6.2 WALLS

6.2.1 Reinforced Concrete Walls

(1) A reinforced concrete wall is a vertical load-bearing member whose greatest lateral dimension is more than four times its least lateral dimension, and in which the reinforcement is taken into account when considering its strength. For walls subjected predominantly to out-of-plane bending, the rules for slabs apply.

(2) The requirements on minimum areas of reinforcement given in Chapter 7 shall be complied with.

(3) A reinforced wall shall be considered as either short or slender and as either braced or unbraced as follows:

Short or Slender Walls: A wall may be considered short when the ratio of its effective height to its thickness does not exceed 7. It shall otherwise be considered slender.

Braced or Unbraced Walls: A wall may be considered as braced if, at right angles to the plane of the wall, lateral stability to the structure as a whole is provided by walls or other suitable bracing designed to resist all lateral forces in that direction. It shall otherwise be considered as unbraced.

(4) The overall stability of a multi-story building shall not, in any direction, depend on unbraced walls alone.

6.2.1.1 *Design of Reinforced Concrete Walls for Flexure and Axial Loads*

(1) Walls subject to combined flexure and axial load shall be designed under the provisions of Chapter 4, unless designed in accordance with Section 6.2.2.

(2) The length of the wall to be considered effective for each concentrated load shall not exceed the center-to-center distance between loads, nor shall it exceed the width of the bearing plus four times the wall thickness.

(3) **Effective Height:** The effective height L_e of reinforced concrete walls in the non-sway mode shall be determined from Eq. 6.1.

$$L_e = \beta L \tag{6.1}$$

where L is the story height of the wall
β is the coefficient defined in Eqs. 6.2 to 6.5

The following values shall be adopted for the coefficient β:

(1) Walls with two edges restrained

$$\beta = 1.00 \tag{6.2}$$

(2) Walls with three edges restrained

$$\beta = \frac{1}{1 + (L/3b)^2} \geq 0.3 \tag{6.3}$$

(3) Walls with four edges restrained

$$\beta = \frac{1}{1 + (L/b)^2} \quad \text{for} \quad L \leq b \tag{6.4}$$

$$\beta = \frac{1}{2(L/b)} \quad \text{for} \quad L > b \tag{6.5}$$

where b is the width of the wall measured center-to-center of the bracing walls, or width measured from the center of a bracing wall to the free edge.

6.2.1.2 *Shear Resistance of Reinforced Walls*

(1) Design for horizontal shear forces in the plane of the wall shall be in accordance with provisions for beam in Section 4.5.3, with the following modifications:

(a) The effective depth d shall be taken as $0.8b$
(b) Sections located closer to the base than a distance $b/2$ or $L/2$, whichever is less, be designed for the shear at $b/2$ or $L/2$
(c) When the applied shear V_{sd} is less than $V_c/2$, the minimum shear reinforcement required by the provisions of Chapter 7 shall be provided.

(2) Design for shear forces perpendicular to the face of the wall shall be in accordance with provisions for slabs in Section 4.5.3.

6.2.2 Plain Concrete Walls

(1) A plain concrete wall is a vertical load bearing concrete member whose greatest lateral dimension is more than four times its least lateral dimension and which is assumed to be without reinforcement when considering its strength, irrespective of whether it is actually reinforced or not. The definitions for a short or slender, or braced or unbraced wall given in Section 6.2.1 for a reinforced concrete wall shall apply also to a plain concrete wall.

6.2.2.1 *Design of Plain Concrete Walls for Flexure and Axial Loads*

(1) The simplified design procedure given below may be used for plain concrete walls with eccentricities of load in the plane of the wall of up to one-third the length of the wall and at right angles to the wall of up to half the thickness of the wall.

(2) The slenderness ratio λ shall not exceed 100.

(3) **Effective Height:** The effective height of plain concrete walls shall be determined from Eq. 6.1 as for reinforced concrete walls.

(4) **Axial Load Capacity:** Design axial load strength of plain concrete wall shall be computed from:

(a) Braced Walls: for short braced walls, the axial load resistance N_{Rd} is given by:

$$N_{Rd} = (1 - 2e/h)A_c f_{cd} \tag{6.6}$$

where e is the resultant eccentricity of load at right angles to the plane of the wall (minimum value of $0.05h$)
h is the thickness of the wall
A_c is the cross-sectional area of the wall.

For slender braced walls, the axial load resistance is given by Eq.6.6 with the eccentricity e redefined and calculated as given below:

$$e = 0.6e_0 + e_2 \tag{6.7}$$

where e_0 is the resultant eccentricity of load at right angles to the plane of the wall (minimum value of $0.05h$).
e_2 is the second order eccentricity is given by $0.4h(L_e/10h)^2$

(b) Unbraced Walls: The axial load resistance N_{Rd} is calculated at the top and at the bottom of the wall using Eq. 6.7 but with e redefined and calculated as given below:

at the top:

$$e = e_{01} \tag{6.8}$$

at the bottom:

$$e = e_{02} + e_2 \tag{6.9}$$

where e_{01} is the first order eccentricity at the top of the wall
e_{02} the first order eccentricity at the bottom of the wall
e_2 is the second order eccentricity given by $0.4h(L_e/10h)^2$

6.2.2.2 *Shear Resistance of Plain Walls*

Design for shear resistance of plain walls shall be in accordance with the provisions for reinforced walls given in Section 6.2.1.2.

6.3 DEEP BEAMS

6.3.1 General

(1) Flexural members with span-to-depth ratios less than 2 for simple spans, or 2.5 for continuous spans are defined as deep beams.

(2) Deep beams under a concentrated load may be designed using a simple strut and tie model.

(3) In some cases, e.g. lower depth/span ratios, distributed loads, more than one concentrated load, etc., models combining strut and tie action with truss action may be used.

(4) Continuous deep beams are sensitive to differential settlement. A range of support reactions, corresponding to possible settlements, should therefore be considered.

(5) Design for shear effects shall be in accordance with Section 6.3.2.

(6) The detailing requirements of Chapter 7 generally, and Section 7.2.6 in particular, shall be met.

6.3.2 Design for Shear

(1) These provisions apply to:

(a) Shear spans supporting a principal load as defined in Section 6.3.2.1 located at a distance a_v not greater than twice the effective depth d.

(b) Shear spans not supporting a principal load or portions of beams supporting uniform loads in which the distance l_s between the points of zero shear and the support is less than three times the effective depth.

(2) In each case, the beams shall be loaded on the top face and supported on the bottom face. For beams loaded by members framing into the sides, the load may be assumed to be applied at the top of the supported member provided that reinforcement satisfying the requirements for indirect supports given in Section 6.6.3 is provided.

(3) Beams supported on members framing into the sides may be assumed to be supported at the level of the bottom of the supporting member.

6.3.2.1 *Definitions and Limitation*

(1) For a given shear span, a principal load is a concentrated load which causes 50 percent or more of the shear at the support of that shear span.

(2) The shear span a_v shall be taken equal to the distance from the center of the principal load to the center of the support. This span shall not be more than 1.15 times the clear distance from the face of the load to the face of the support. The distance l_s shall be taken equal to the distance from the point of zero shear to the center of the support but not more than 1.15 times the clear distance from the point of zero shear to the face of the support.

6.3.2.2 *Shear Strength of Deep Shear Spans*

(1) The shear resistance of deep shear spans S_{Rd} shall be obtained as the sum of the resistances of the concrete V_{cd} and the vertical and horizontal stirrups V_s and V_h, respectively.

(2) The applied shear V_{Sd} shall not exceed the limit imposed by Eq.4.28.

6.3.2.3 *Shear Carried by Deep Shear Spans*

(1) For deep shear spans supporting a principal load,

(a) The shear resistance V_{Rd} shall be computed at $a_v/2$. The shear reinforcement required at this section shall be used throughout the entire shear span.

(b) The shear force V_c carried by the concrete shall be taken as the value obtained from Eq. 4.28 multiplied with

$$\beta = \frac{2d}{a_v} \geq 1.0 \tag{6.10}$$

(c) The shear force V_s transferred by vertical stirrups shall be given by

$$V_s = \frac{A_v f_{yd}\left(a_v - \frac{d}{2}\right)}{s_v} \leq \frac{A_v d f_{yd}}{s_v} \tag{6.11}$$

(d) The shear force V_h transferred by horizontal stirrups shall be given by

$$V_h = \frac{A_{vh} f_{yd}\left(\frac{3d}{2} - a_v\right)}{s_h} \leq \frac{A_{vh} d f_{yd}}{s_h} \tag{6.12}$$

where A_v is the area of vertical stirrups
A_{vh} is the area of horizontal stirrups
s_v is the spacing of the vertical stirrups ($s_v \leq d/4$)
s_h is the spacing of the horizontal stirrups $s_h \leq d/3$)

(2) A_v and A_{vh} shall satisfy the minimum requirement given in Section 7.2.1.2

(3) For deep shear spans not supporting a principal load, the above provisions apply with $a_v/2$ replaced by $l_s/3$.

6.4 CORBELS

6.4.1 Definitions and Limitations

(1) These provisions apply to corbels having a shear span to depth ratio a_v/d of unity or less.

(2) The distance d shall be measured at a section adjacent to the face of the support, but shall not be taken greater than twice the depth of the corbel at the outside edge of the bearing area.

6.4.2 Design

(1) Corbels with $0.4d \leq a_v \leq d$ may be designed using a simple strut and tie model.

(2) For deeper corbels ($a_v > d$), other adequate strut and tie models may be considered.

(3) Corbels for which $a_v > d$ may be designed as cantilever beams.

(4) Unless special provision is made to limit horizontal forces on the support, or other justification is given, the corbel shall be designed for the vertical force F_v, and a horizontal force $H_c \geq 0.2\ F_v$ acting at the bearing area.

(5) The effective depth d of the corbel shall be determined from considerations of shear (see Section 4.5).

(6) The local effects due to the assumed strut and tie system should be considered on the overall design of the supporting member.

(7) The detailing requirements of Chapter 7 generally, and Section 7.2.7 in particular, shall be met.

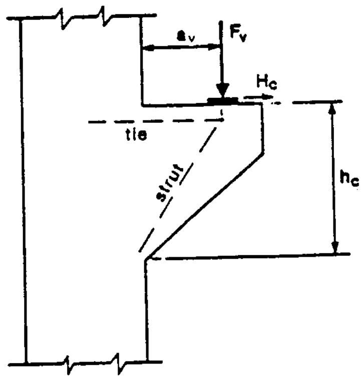

Figure 6.1 Example of a Corbel with a Strut and Tie Model

6.5 FOOTINGS

6.5.1 Moment in Footings

(1) The external moment on any section of a footing shall be determined by passing a vertical plane through the footing, and computing the moment of the forces acting over the entire area of the footing on one side of that vertical plane.

(2) The critical section for moment shall be taken as follows:

(a) At the face of column, pedestal, or wall, for footings supporting a concrete column pedestal or wall
(b) Halfway between middle and edge of wall, for footings supporting a masonry wall.
(c) Halfway between face of column and edge of steel base for footings supporting a column with steel base plates.

6.5.2 Flexural Reinforcement

(1) **Distribution:** In one-way footings and two-way square footings, reinforcement shall be distributed uniformly across the entire width of footing.

(2) In two-way rectangular footings, reinforcement shall be distributed as follows:

(a) Reinforcement in long direction shall be distributed uniformly across the entire width of footing.

(b) For reinforcement in the short direction, a portion of the total reinforcement given by Eq. 6.13 shall be distributed uniformly over a band width (centered on center line of column or pedestal) equal to the length of the short side of footing. The remainder of the reinforcement required in the short direction shall be distributed uniformly outside the center band width of the footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta + 1} \qquad (6.13)$$

where β is the ratio of long side to short side of footing.

(3) **Anchorage:** If the projection of the footing from the critical section for moment defined in Section 6.5.1 does not exceed the effective depth d at that section, the bottom reinforcement shall be provided with full anchorage length measured from the end of the straight portion of the bars.

(4) If the projection exceeds d, the anchorage length may be measured from a section situated at a distance d from the above defined critical section for moment.

6.5.3 Shear in Footings

(1) Design of footings for shear shall be in accordance with provisions for slabs in Section 4.5.

(2) The location of the critical section for shear in accordance with Section 4.5 shall be measured from face of column, pedestal or wall for footings supporting a column, pedestal, or wall.

(3) For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from the location defined in Section 6.5.1.

6.5.4 Bearing

(1) All forces and moments applied at the base of a column or pedestal shall be transferred to the top of the supporting pedestal or footing by bearing on concrete and by reinforcement.

(2) The design bearing strength on concrete shall not exceed the design compressive strength f_{cd}, except as follows:

(a) When the supporting surface is wider on all sides than the loaded area, the design bearing strength on the load area may be multiplied by $\sqrt{(A_2/A_1)}$ but not more than 2.

(b) When the supporting surface is sloped or stepped, A_2 may be taken as the area of the lower base of the largest frustrum of a right pyramid or cone contained wholly within the support and having for its upper base the loaded area, and having side slopes of 2 vertical to 1 horizontal.

In the above A_1 is the loaded area, and A_2 is the maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area.

6.5.5 Minimum Footing Depth

(1) The depth of footing above bottom reinforcement shall not be less than 150 mm for footings on soil, nor 300 mm for footings on piles.

6.5.6 Plain Concrete Pedestals and Footings

(1) The maximum compressive stress in plain concrete pedestals shall not exceed the concrete bearing strength given in Section 6.5.4.

(2) Plain concrete footing may be used provided that the projection of the footing beyond the critical section defined in Section 6.5.1 does not exceed half the thickness of the footing at that section.

(3) Flexural design stresses in plain concrete shall not exceed $1.70 f_{ctd}$.

(4) Shear and punching shall be checked in accordance with Sections 4.8 and 4.10.

(5) Plain concrete shall not be used for pile caps.

(6) The depth of plain concrete footings shall not be less than 200 mm.

6.6 PILE CAPS

6.6.1 Moment in Pile Caps

(1) Section 6.6.1 shall apply to pile caps also.

6.6.2 Flexural Reinforcement

(1) **Distribution:** The bottom reinforcement may consist partly of bars placed in strips between the piles.

(2) **Anchorage:** The reinforcement shall always be arranged in such a way that adequate anchorage is provided beyond the axial plane of the piles.

(3) This may be deemed to be satisfied if the tensile force in the reinforcement crossing a pile, within a width of 3 pile diameters, is not less than the pile reaction, assuming the reinforcement is fully stressed.

6.6.3 Shear

(1) Computation of shear on any section through a pile cap shall be in accordance with the following:

(a) The total reaction from any pile whose center is located half pile diameter or more out-side the section shall be considered as producing shear on that section.
(b) Reaction from any pile whose center is located half pile diameter or more inside the section shall be considered as producing no shear on that section.
(c) For intermediate positions of pile center, linear interpolation between (a) and (b) above may be assumed.

6.6.4 Footings on Two Piles

(1) Secondary reinforcement distributed horizontally and vertically is required only for footings resting on only two piles, in consideration of one-way deep beam action of such footings.

(2) The amount of the secondary reinforcement to be provided shall be as for deep beams (see Section 6.6.2).

6.6.5 Minimum Thickness

(1) The thickness of pile above the bottom reinforcement shall not be less than 300 mm.

6.7 PARTICULAR CASES

6.7.1 Local Forces

(1) When a local compressive stress is applied at the end of a structural member or the intersection of two structural members, transverse reinforcement capable of resisting the resulting transverse tensile stresses shall, in general, be provided. However, this requirement may be waived provided the dispersion of the pressure is not steeper than 2 vertical to 1 horizontal as stipulated in Section 6.7.4.

(2) When transverse reinforcement is provided, it shall be evenly distributed over a distance measured perpendicular to area A_1 between $0.1a_2$ and a_2; a_2 being the side length of area A_2 measured in the direction of the dispersion of the local force (see Fig. 6.2).

(3) The transverse tensile force may be obtained for two-orthogonal directions as:

$$N_{td} = 0.3F_d(1 - \frac{a_1}{a_2}) \qquad (6.14)$$

where F_d is the local force
a_1 is the side length of area A_1 measured parallel to a_2 (see Fig. 6.2).

6.7.2 Concentrated Forces

(1) Where one or more concentrated forces act at the end of a member or at the intersection of two structural members, local supplementary reinforcement should be provided capable of resisting the transverse tensile forces caused by these forces.

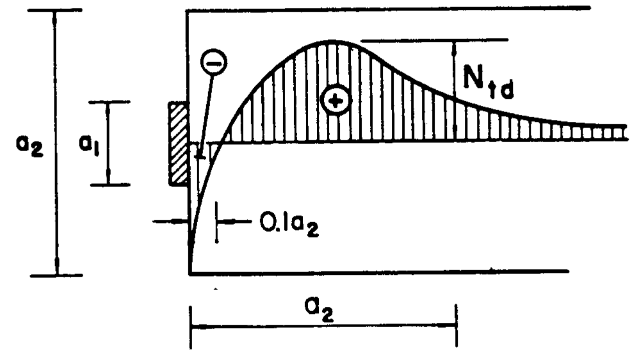

Figure 6.2 Distribution of the Transverse Force

6.7.2 Concentrated Forces

(1) Where one or more concentrated forces act at the end of a member or at the intersection of two structural members, local supplementary reinforcement should be provided capable of resisting the transverse tensile forces caused by these forces.

(2) This supplementary reinforcement may consist of links or of layers of reinforcement bent in the shape of hair pins.

(3) For a uniform distribution of load on area A_{co}, (Fig. 6.3), the concentrated resistance force can be determined as follows:

$$F_{Rdu} = \frac{A_c f_{cd}}{\sqrt{A_{cl}/A_{co}}} \leq 3.3 f_{cd} A_{co} \tag{6.15}$$

where A_{co} denotes the loaded area
A_{c1} denotes the maximum area corresponding geometrically to A_{co}, having the same center of gravity, which it is possible to inscribe in the total area A_c, situated in the same plane as the loaded area.

If A_c and A_{co} correspond geometrically and have the same center of gravity : $A_{c1} = A_c$.

The value of F_{Rdu} obtained from Eq. B.20 should be reduced if the load is not uniformly distributed on area A_{co} or if it is accompanied by large shear forces.

This method applies to post-tensioned members and does not apply to the anchorages of prestressing tendons (Section B.4.2.1).

6.7.3 Bursting Forces

(1) Concentrated bursting forces which occur when there are major changes in the direction in which internal forces act as in frame joints, for example, shall be resisted by additional suitably anchored reinforcement (see Fig. 6.4).

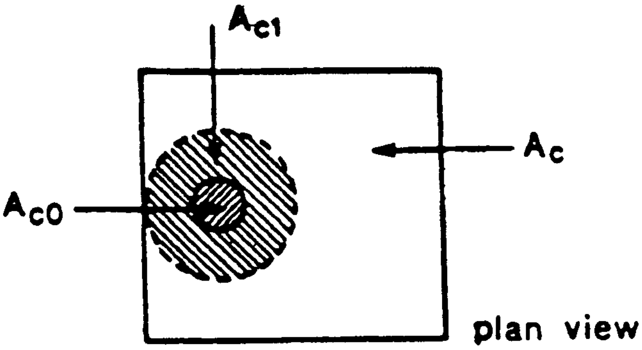

Figure 6.3 Definition of the Areas to be introduced in Eq.B.1

6.7.4 Indirect Supports

(1) The junction between a bearing beam or girder and supported beam is defined as an indirect support.

(2) The reinforcement needed for the transmission of the load from the beam to the girder may be determined by truss analogy.

(3) The hanging or transmission reinforcement shall normally be calculated for the total reaction acting at the support but may be reduced in the ratio of h_1/h_2 if the height h_1 of the supported beam is smaller than the height h_2 of the girder provided that the top surfaces of the two beams are at the same level (see Fig. 6.5). No reduction shall be made when loads are suspended at the lower part of a member.

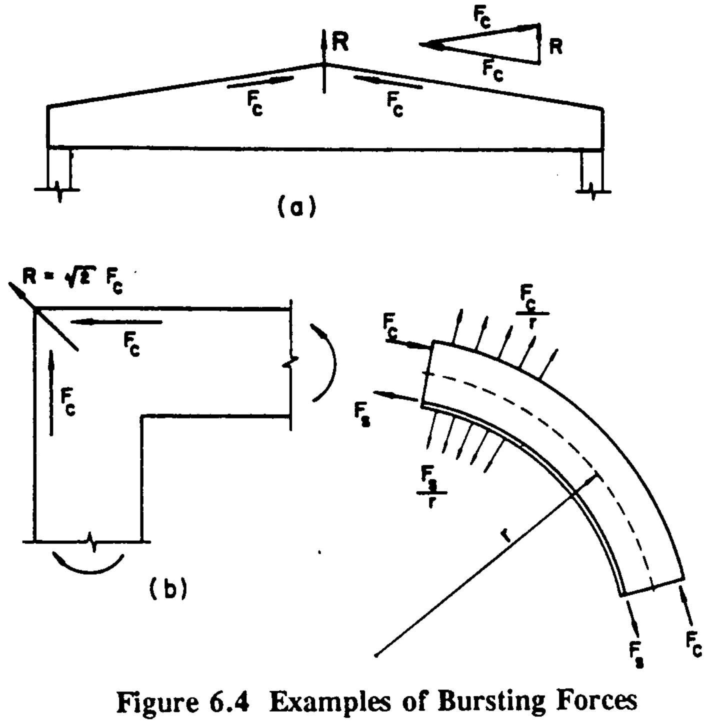

Figure 6.4 Examples of Bursting Forces

(4) Transmission reinforcement shall be composed preferably of stirrups surrounding the main reinforcement of the girder. The stirrups shall be distributed in the girder within a distance $0.5h_1$ on either side of the beams.

(5) The main reinforcement in the supported beam shall be placed above that of the girder.

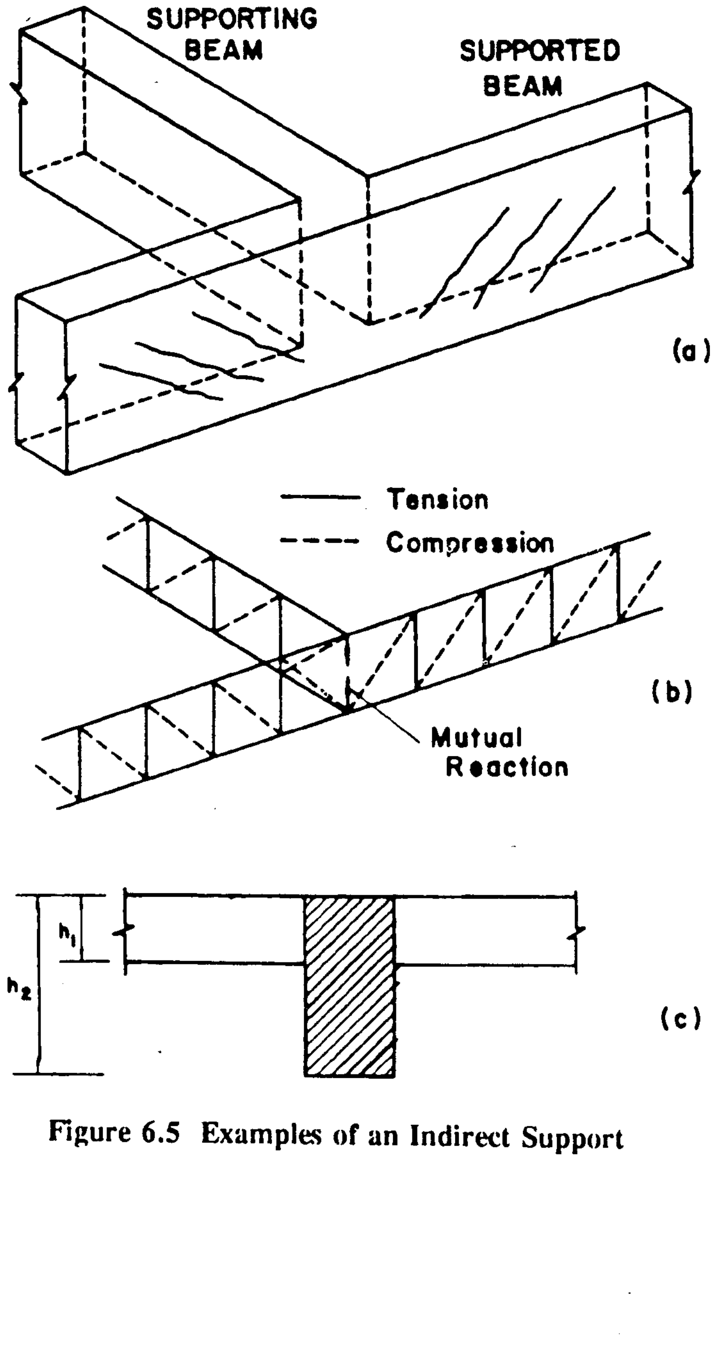

Figure 6.5 Examples of an Indirect Support

CHAPTER 7
DETAILING PROVISIONS

7.1 DETAILING OF REINFORCEMENT

7.1.1 General

(1) The mechanical and bonding properties of the reinforcement shall meet the requirements of the specified standard.

7.1.2 Bending of Bars

(1) The minimum diameter to which a bar is bent shall be such as to avoid crushing or splitting of the concrete inside the bend of the bar, and to avoid bending cracks in the bar.

(2) The minimum diameter of the mandrel used shall be at least equal to the minimum specified for the bend-rebend test of the reinforcement.

(3) For bars or wires, the minimum diameter of the mandrel used should be not less than the values given in Table 7.1.

TabLe 7.1 - Minimum Diameter of Bend

Bar size	Main Reinforcement	Stirrups and Ties
$\phi \leq 16$	5ϕ	4ϕ
$16 < \phi \leq 25$	6ϕ	6ϕ
$25 < \phi \quad 32$	8ϕ	-
$\phi > 32$	10ϕ	-

(4) When several adjacent bars in the same layer are bent in the same zone (for example, at the corners of a frame), the diameter of mandrel shall be chosen with a view to avoiding crushing or splitting of the concrete under the effect of the pressure that occurs inside the bend (see Eq. 7.7)

7.1.3 Concrete Cover to Reinforcement

(1) The concrete cover is the distance between the outer surface of the reinforcement (including links and stirrups) and the nearest concrete surface.

(2) A minimum concrete cover shall be provided in order to ensure:

(a) the safe transmission of bond forces;
(b) that spalling will not occur;
(c) an adequate fire resistance;
(d) the protection of the steel against corrosion.

(3) The protection of reinforcement against corrosion depends upon the continuing presence of a surrounding alkaline environment provided by an adequate thickness of good quality, well-cured concrete. The thickness of cover required depends both upon the exposure conditions and on the concrete quality.

(4) The minimum concrete cover required for the criterion in (3) above shall first be determined. This shall be increased by an allowable (Δh) for tolerances, which is dependent on the type and size of structural element, the type of construction, standards of workmanship and quality control, and detailing practice. The result is the required nominal cover which shall be specified on the drawings.

(5) To transmit bond forces safely, and to ensure adequate compaction, the concrete cover, to the bar or tendon being considered, should never be less than:

(a) ϕ or ϕ_n ($\leq$ 40 mm), or
(b) (ϕ + 5 mm) or (ϕ_n + 5 mm) if $d_g > 32$ mm

where ϕ is the diameter of the bar
ϕ_n is the equivalent diameter for a bundle
d_g is the largest nominal maximum aggregate size.

(6) The minimum concrete cover to all reinforcement including links and stirrups should not be less than the appropriate values given in Table 7.2.

Table 7.2 Minimum Cover Requirements for Concrete Members

Type of exposure	Dry environment: Interior of buildings of normal habitation or offices **(Mild)**	Humid environment: Interior components (e.g. laundries); exterior components; components in non-aggressive soil and/or water **(Moderate)**	Seawater and/or aggressive chemical environment: Components completely or partially submerged in seawater; components in saturated salt air; aggressive industrial atmospheres **(Severe)**
Minimum cover (mm)	15	25	50

(7) Where surface reinforcement is used, the cover should either comply with (6) above, or protective measures should be taken (e.g. protective coatings); in any case, the minimum cover shall not be less than 20 mm.

(8) The allowance (Δh) for tolerance will usually be in the range of 0 mm $< \Delta h <$ 5 mm, for precast elements, if production control can guarantee these values and if this is verified by quality control. The allowance will be in the range of 5 mm $< \Delta h <$ 10 mm for insitu reinforced concrete construction.

(9) For concrete cast against uneven surfaces, the minimum covers given in Table 7.1 should generally be increased by larger allowances for tolerances. For example, for concrete cast directly against the earth, the minimum cover should be greater than 75 mm; for concrete cast against prepared ground (including blinding) the minimum cover should be greater than 40 mm. Surfaces having design features, such as ribbed finishes or exposed aggregate, also require increased cover.

(10) The nominal cover shall always be at least equal to the diameter of the bar ϕ and in the case of bundles to the size of a single bar of equivalent area given by Eq. 7.1.

$$\phi_e = \phi_b \sqrt{n} \qquad (7.1)$$

where ϕ_e is the effective diameter of the bundle
ϕ_b is the diameter of bars forming the bundle
n is the number of bars in the bundle

(11) The required minimum covers given in Table 7.1, as modified to allow for tolerances, may be insufficient for fire protection. Particular requirements for fire resistance are given in the National Building Code.

7.1.4 Spacing of Reinforcement

(1) The spacing of bars shall be suitable for the proper compaction of concrete and when an internal vibrator is likely to be used, sufficient space shall be left between reinforcement to enable the vibrator to be inserted.

(2) The maximum aggregate size d_g should be chosen to permit adequate compaction of the concrete round the bars.

(3) The clear horizontal and vertical distance between bars shall be at least equal to the largest of the following values:

(a) 20 mm
(b) The diameter of the largest bar or effective diameter of the bundle
(c) The maximum size of the aggregate d_g plus 5 mm

(4) Where bars are positioned in separate horizontal layers, the bars in each layer should be located vertically above each other and the space between the resulting columns of bars should permit the passage of an internal vibrator.

(5) Lapped bars may touch one another within the lap length.

(6) Maximum distance between bars shall comply with the requirements of Section 7.2.

7.1.5 Bond

7.1.5.1 ***Design Bond Strength***

(1) The design bond strength f_{bd} depends on the type of reinforcement, the concrete strength and the position of the bar during concreting.

(2) The bond conditions are considered to be good for:

(a) All bars which are in the lower half of an element
(b) All bars in elements whose depth does not exceed 300 mm
(c) All bars which are at least 300 mm from the top of an element in which they are placed
(d) All bars with an inclination of 45° to 90° to the horizontal during concreting.

(3) For good bond conditions, the design bond strength of plain bars may be obtained from Eq. 7.2

$$f_{bd} = f_{ctd} \tag{7.2}$$

(4) For deformed bars twice the value for plain bars may be used.

(5) For other bond conditions, the design bond strength may be taken as 0.7 times the value for good bond conditions.

7.1.6 Anchorage of Reinforcement

(1) All reinforcement shall be properly anchored at each end with due consideration for the effect of arch action and shear cracks.

(2) To prevent bond failure, the tension or compression in any bar at any section due to ultimate loads shall be developed on each side of the section by an appropriate embedment length or end anchorage or a combination thereof. Hooks may be used in developing bars in tension.

7.1.6.1 *Basic Anchorage Length*

(1) The basic anchorage length is the embedment length required to develop the full design strength of a straight reinforcing bar.

(2) The basic anchorage length l_b for a bar of diameter ϕ is:

$$l_b = \frac{\phi}{4} \cdot \frac{f_{yd}}{f_{bd}} \tag{7.3}$$

7.1.6.2 *Required Anchorage Length*

(1) The required anchorage length $l_{b,net}$ depends on the type of anchorage and on the stress in the reinforcement and can be calculated as:

$$l_b,net = a l_b \frac{A_{s,cal}}{A_{s,ef}} \geq l_{b,min} \tag{7.4}$$

where $A_{s,cal}$ is the theoretical area of reinforcement required by the design
$A_{s,ef}$ is the area of reinforcement actually provided
a = 1.0 for straight bar anchorage in tension or compression
= 0.7 for anchorage in tension with the standard hooks of Fig. 7.2
$l_{b,min}$ is the minimum anchorage length

(2) For bars in tension,

$$l_{b,min} = 0.3 l_b \geq 10\phi \quad \text{or} \quad \geq 200 \text{ mm} \tag{7.5}$$

(3) For bars in compression,

$$l_{b,min} = 0.6 l_b \geq 10\phi \quad \text{or} \quad \geq 200 \text{ mm} \tag{7.6}$$

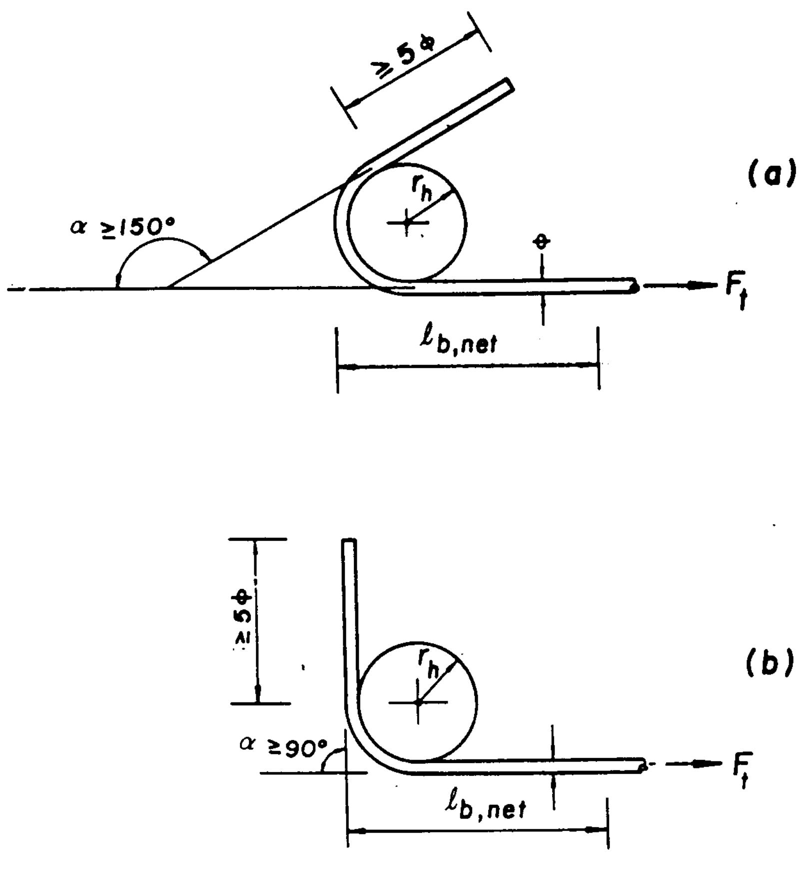

Figure 7.1 Standard Hooks

7.1.6.3 *Additional Requirements for Loops*

(1) In order to prevent concrete failure in the plane of a loop anchorage, the diameter of the mandrel used must satisfy Eq. 7.7.

$$d \geq (0.7 + 1.4\frac{\phi}{s})\frac{\sigma_{sd}}{1.5f_{cd}}\phi \qquad (7.7)$$

where σ_{sd} is the stress in the bar at the start of the bend
s is the smaller of:
- the spacing between the loops
- the cover c increased by half the diameter ϕ

7.1.6.4 *Ties and Stirrups*

(1) The type of anchorage used shall not induce splitting or spalling of the concrete cover.

(2) Anchorage by hooks (135° to 180°) is required for plain bars.

(3) Anchorage by bends (90° to 135°) is only allowed for deformed bars.

(4) The required anchorage $l_{b,net}$ shall be measured from the mid-depth of the member.

7.1.6.5 *Laps and Joints*

(1) The length of lap l_o shall be at least equal to:

$$l_o \geq a_1 l_{b,net} \geq l_{o,min} \tag{7.8}$$

where $l_{o,min} = 0.3 a a_1 l_b \geq 15\phi$
or ≥ 200 mm

$l_{b,net}$ and a are given in Section 7.1.6.2.

a_1 is a function of the percentage of the reinforcement lapped at any one section as given in Table 7.3. Lapped joints are considered to be at the same section if the distance between their centers does not exceed the required lap length.

Table 7.3 Values of a_1 for Eq. 7.8 and 7.9

Distance Between Two Adjacent Laps	Distance to Nearest Surface	Percentage of Reinforcement Lapped Within Required Lap Length				
a	b	20%	25%	33%	50%	100%
$a \leq 10\phi$ and/or	$b \leq 5\phi$	1.2	1.4	1.6	1.8	2.0
$a > 10\phi$ and	$b > 5\phi$	1.0	1.1	1.2	1.3	1.4

(2) The lap length l_o shall be at least equal to the basic anchorage length l_b.

(3) The percentage of lapped bars in compression in any one section may be 100% of the total steel cross section.

(4) The separation of the bars at the joint shall be as small as possible and shall not exceed 4ϕ except in slabs and walls. The distance between two adjacent laps shall be equal to:

(a) In the transverse direction: $2\phi \geq 20$ mm (clear distance).
(b) In the longitudinal direction: $1.5 l_o$ (center-to-center distance)

(5) Transverse forces in lapped joints shall be checked where

(a) $\phi \geq 16$ mm, or
(b) the joint affects more than one-half of the total area of the bars.

7.1.6.6 *Additional Rules for Deformed Bars of Large Diameter* ($\phi > 32$ mm)

(1) Bars of Diameter $\phi > 32$ mm shall be used only in elements of thickness at least equal to 15 ϕ.

(2) When large bars are used at relatively wide spacings, skin reinforcement is required for adequate crack control.

(3) The design bond strength f_{bd} from Section 7.5.1 shall be reduced by the factor:

$$\eta = \frac{132 - \phi}{100} \tag{7.9}$$

where ϕ is in mm.

(4) As a general rule only straight anchorage or mechanical anchorage is allowed.

(5) Lap joints are not allowed at joints.

7.1.6.7 *Additional Rules for Bundled Bars*

(1) For design, bundles of bars containing *n* bars having the same diameter are replaced by a single notional bar having the same center of gravity, and an equivalent diameter:

$$\phi_n = \phi\sqrt{n} \leq 55 \text{ mm} \tag{7.10}$$

where n is the number of bars in a bundle, which must be limited to
$n \leq 4$ for vertical bars in compression and for bars of a lapped joint
$n \leq 3$ for all other cases.

(2) The equivalent diameter ϕ_n is taken into account in evaluating the minimum cover. However, the cover provided shall be measured from the actual outside contour of the bundle.

(3) The anchorages of the bars of a bundle can only be straight anchorages.

(4) The anchorage of a bundle is dependent upon the anchorage of the individual bars.

(5) The anchorages shall be staggered; for bundles of 2, 3, or 4 bars the staggering shall be respectively, 1.2, 1.3 or 1.4 times the anchorage length of the individual bars.

(6) Joints can be made on only one bar at a time but at any section there shall be no more than four bars in a bundle. The laps of the individual bars shall be staggered in accordance with Section 7.6.7.3

7.1.7 Curtailment of Longitudinal Flexural Reinforcement

7.1.7.1 *Staggering Rule*

(1) The tensile force diagram or M/z diagram for a flexural member shall be obtained by dividing the moment diagram by the appropriate lever arm z and displacing the resulting curves horizontally by the amounts a_l as shown in Fig. 7.2.

(2) The displacement a_l depends on the spacing of potential shear cracks and may be taken as follows, in the absence of more accurate determination:

(a)	Members without shear reinforcement (e.g slabs)	$a_l = 1.0d$
(b)	members with $V_{sd} < 2V_c$	$a_l = 0.75d$
(c)	members with $V_{sd} \geq 2V_c$	$a_l = 0.50d$

where V_{sd} is the applied design shear force.

(3) Near points of zero moment, $a_l \geq d$ shall be taken for both positive and negative moments.

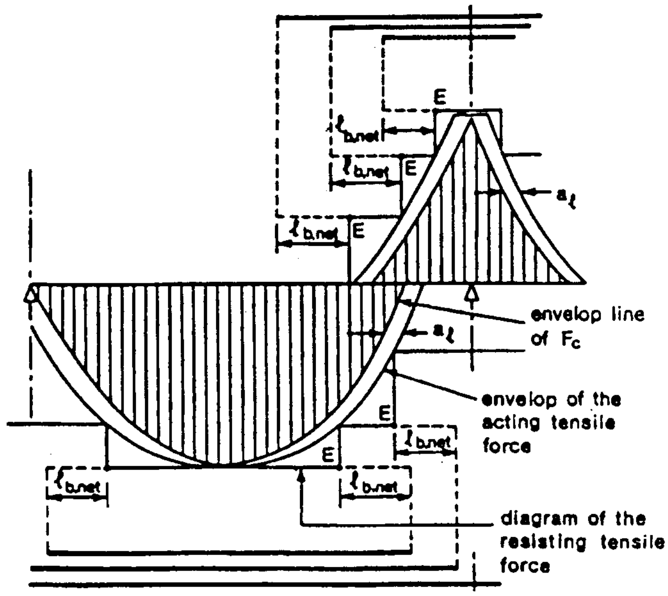

Figure 7.2 Tensile Force Diagram

7.1.7.2 *Anchorage Length of Reinforcement*

(1) Reinforcement shall extend beyond the point at which it is no longer required to resist tension for a length given by:

(a) l_b according to Eq. 7.3, or
(b) $l_{b,net} \geq d$ according to Eq. 7.4 provided that in this case, the continuing bars are capable of resisting twice the applied moment at the section.

(2) The anchorage length of bars that are bent up as shear reinforcement shall be at least equal to $1.3l_{b,net}$ in zones subjected to tension and to $0.7l_{b,net}$ in zones subjected to compression.

7.1.7.3 *Anchorage of Bottom Reinforcement at Supports*

(3) At least one-quarter of the positive moment reinforcement in simple beams and one-half of the positive moment reinforcement in slabs shall extend along the same face of the member into the support.

(4) The anchorage of this reinforcement shall be capable of developing the following tensile force F_t.

$$F_t = V_{sd}\frac{a_l}{d} \geq 0.5V_{sd} \qquad (7.11)$$

(5) The anchorage length is measured from:

(a) The face of the support, for a direct support
(b) A plane inside the support located at a distance of 1/3 the width of the support from the face of the support, for an indirect support.

(6) The anchorage length of the bottom reinforcement at intermediate supports shall be at least 10ϕ.

7.2 DETAILING OF STRUCTURAL MEMBERS

7.2.1 Beams

7.2.1.1 *Longitudinal Reinforcement*

(1) The geometrical ratio of reinforcement ρ at any section of a beam where positive reinforcement is required by analysis shall not be less than that given by

$$\rho_{min} = \frac{0.6}{f_{yk}} \tag{7.12}$$

where f_{yk} is in MPa.

(2) In T-beams and joists where the web is in tension, the ratio ρ shall be computed for this purpose using width of web.

(3) The maximum reinforcement ratio ρ_{max} for either tensile or compressive reinforcement shall be 0.04.

7.2.1.2 *Shear Reinforcement*

(1) All beams, except joists of ribbed slabs, shall be provided with at least the minimum web reinforcement given by:

$$\rho_{\omega,min} = \frac{0.4}{f_{yk}} \tag{7.13}$$

where f_{yk} is in MPa.

(2) The maximum spacing s_{max} between stirrups, in the longitudinal direction, shall be as given below:

$$s_{max} = 0.5d \leq 300 \text{ mm if } V_{sd} \leq \frac{2}{3}V_{RD} \tag{7.14}$$

$$s_{max} = 0.3d \leq 200 \text{ mm if } V_{sd} > \frac{2}{3}V_{Rd} \tag{7.15}$$

(3) The transverse spacing of legs of stirrups shall not exceed d, or 800 mm, whichever is the smaller.

7.2.1.3 *Torsional Reinforcement*

(1) The minimum web reinforcement given by Eq. 7.2 shall be provided in the form of closed stirrups for the case of torsional reinforcement.

(2) The spacing of the stirrups shall not exceed $u_{ef}/8$.

(3) The longitudinal bars required for torsion shall be distributed uniformly around the perimeter of the closed stirrups at a spacing not exceeding 350 mm.

(4) At least one longitudinal bar shall be placed in each corner of the closed stirrups.

7.2.2 Slabs

7.2.2.1 *Thickness*

(1) The following minimum thicknesses shall be adopted in design:

(a) 60 mm for slabs not exposed to concentrated loads (e.g inaccessible roofs)
(b) 80 mm for slabs exposed mainly to distributed loads.
(c) 100 mm for slabs exposed to light moving concentrated loads (e.g slabs accessible to light motor vehicles)
(d) 120 mm for slabs exposed to heavy dynamic moving loads (eg. slabs accessible to heavy vehicles)
(e) 150 mm for slabs on point supports (e.g flat slabs)

7.2.2.2 *Flexural Reinforcement*

(1) The ratio of the secondary reinforcement to the main reinforcement shall be at least equal to 0.2.

(2) The geometrical ratio of main reinforcement in a slab shall not be less than:

$$\rho_{min} = \frac{0.5}{f_{yk}} \qquad (7.16)$$

where f_{yk} is in MPa.

(3) The spacing between main bars for slabs shall not exceed the smaller of $2h$ or 350 mm.

(4) The spacing between secondary bars shall not exceed 400 mm.

7.2.3 Hollow or Ribbed Slabs

7.2.3.1 *Sizes*

(1) Ribs shall not be less than 70 mm in width; and shall have a depth, excluding any topping, of not more than 4 times the minimum width of the rib. The rib spacing shall not exceed 1.0 m.

(2) Thickness of topping shall not be less than 40 mm, nor less than 1/10 the clear distance between ribs.

7.2.3.2 *Minimum Reinforcement*

(3) The topping shall be provided with a reinforcement mesh providing in each direction a cross-sectional area not less than 0.001 of the section of the slab.

(4) If the rib spacing exceeds 1.0 m, the topping shall be designed as a slab resting on ribs, considering load concentrations, if any.

(5) The web-flange connections shall be checked in accordance with Section 4.5.5.

7.2.3.3 *Transverse Ribs*

(1) Transverse ribs shall be provided if the span of the ribbed slab exceeds 6.0 m.

(2) When transverse ribs are provided, the center-to-center distance shall not exceed 20 times the overall depth of the ribbed slab.

(3) The transverse ribs shall be designed for at least half the values of maximum moments and shear force in the longitudinal ribs.

7.2.4 Columns

7.2.4.1 *Size*

(1) The minimum lateral dimension of a column shall be at least 150 mm.

7.2.4.2 *Longitudinal Reinforcement*

(1) The area of longitudinal reinforcement shall not be less than $0.008A_c$ nor more than $0.08A_c$. The upper limit shall be observed even where bars overlap.

(2) For columns with a larger cross-section than required by considerations of loading, a reduced effective area not less than one-half the total area may be used to determine minimum reinforcement and design strength.

(3) The minimum number of longitudinal reinforcing bars shall be 6 for bars in a circular arrangement and 4 for bars in a rectangular arrangement.

(4) The diameter of longitudinal bars shall not be less than 12 mm.

7.2.4.3 *Lateral Reinforcement*

(1) The diameter of ties or spirals shall not be less than 6 mm or one quarter of the diameter of the longitudinal bars.

(2) The center-to-center spacing of lateral reinforcement shall not exceed:
- (a) 12 times the minimum diameter of longitudinal bars.
- (b) least dimension of column
- (c) 300 mm

(3) Ties shall be arranged such that every bar or group of bars placed in a corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135° and no bar shall be further than 150 mm clear on each side along the tie from such a laterally supported bar (see Fig. 7.3).

(4) Up to five longitudinal bars in each corner may be secured against lateral buckling by means of the main ties. The center-to-center distance between the outermost of these bars and the corner bar shall not exceed 15 times the diameter of the tie (see Fig. 7.4).

$$s_{max} = 350 \text{ mm}$$

(5) Spirals or circular ties may be used for longitudinal bars located around the perimeter of a circle. The pitch of spirals shall not exceed 100 mm.

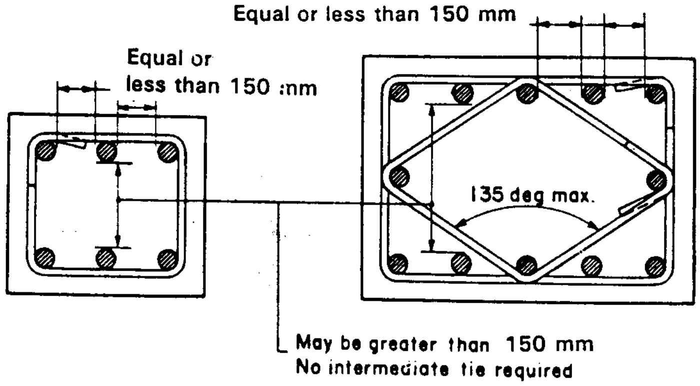

Figure 7.3 Measurements Between Laterally Supported Column Bars

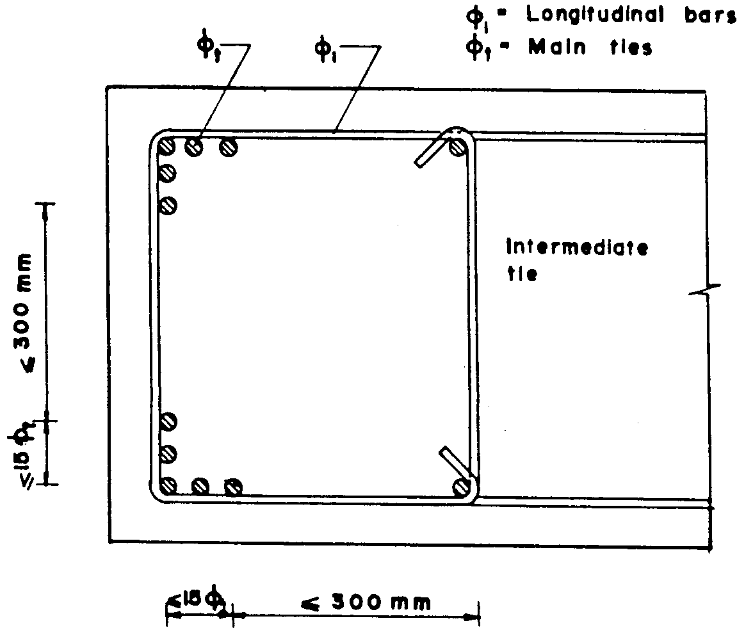

Figure 7.4 Requirements for Main and Intermediate Ties

7.2.5 WALLS

7.2.5.1 *Sizes*

(1) The thickness of load bearing walls shall not be less than 1/25 of the unsupported height or width, whichever is shorter, nor less than 150 mm.

(2) The overall thickness of panel and partition walls shall not be less than 1/30 of the distance between supporting or enclosing members, nor less than 100 mm.

7.2.5.2 *Vertical Reinforcement*

(1) The area of vertical reinforcement shall not be less than $0.004A_c$ nor more than $0.04A_c$. The upper limit shall be observed even where bars overlap.

(2) For walls with a larger cross-section than required by considerations of loading, a reduced effective area not less than one-half the total area may be used to determine minimum reinforcement and design strength.

(3) The diameter of vertical bars shall not be less than 8 mm.

(4) The spacing of vertical bars shall not exceed twice the wall thickness nor 300 mm.

7.2.5.3 *Horizontal Reinforcement*

(1) The area of horizontal reinforcement shall not be less than one-half of that of the vertical reinforcement.

(2) The spacing of horizontal bars shall not exceed 300 mm. The diameter of horizontal bars shall not be less than one quarter of that of the vertical bars.

(3) Horizontal reinforcement shall enclose the vertical reinforcement. The horizontal bars shall be tied to the vertical bars so as to form a rigid mat.

7.2.5.4 *Transverse Reinforcement*

(1) The mats at the two faces of a wall shall be connected to each other by at least 4 transverse S-ties per m^2, when the diameter of the vertical reinforcement is 16 mm or greater.

(2) If the area of required reinforcement exceeds $0.02A_c$, then ties as required for columns (see Section 7.2.4.3) shall be provided.

7.2.6 Deep Beams

7.2.6.1 *Thickness*

(1) The thickness of deep beams shall not be less than 100 mm.

7.2.6.2 *Supplementary Reinforcement*

(1) To supplement the main reinforcement, one layer of mesh reinforcement shall be provided near each face of deep beams. The minimum percentage of reinforcement of each mesh in each direction shall be given by:

$$\rho_{min} = \frac{0.3}{f_{yk}} \tag{7.17}$$

where f_{yk} is in MPa.

(1) The spacing between adjacent bars shall not exceed twice the thickness of the deep beam or 300 mm.

7.2.7 Corbels

(1) The reinforcement, corresponding to the ties considered in the design model (Section 6.4), should be fully anchored beyond the node under the bearing plate by using U-hoops or anchorage devices unless a length $l_{b,net}$ is available between the node and the front of the corbel. The length $l_{b,net}$ should be measured from the point where the compression stresses change their direction.

(2) In corbel with $h_c \geq 300$ mm, when the area of the primary horizontal tie A_s is such that

$$A_s \geq \frac{0.4 A_c f_{cd}}{f_{yd}} \qquad (7.18)$$

(where A_c is the sectional area of the concrete in the corbel at the column), then closed stirrups, having a total area not less than $0.4A_s$, should be distributed over the effective depth d in order to cater for splitting stresses in the concrete strut. They can be placed either horizontally (Fig. 7.5(a) or inclined (Fig. 7.5(b)).

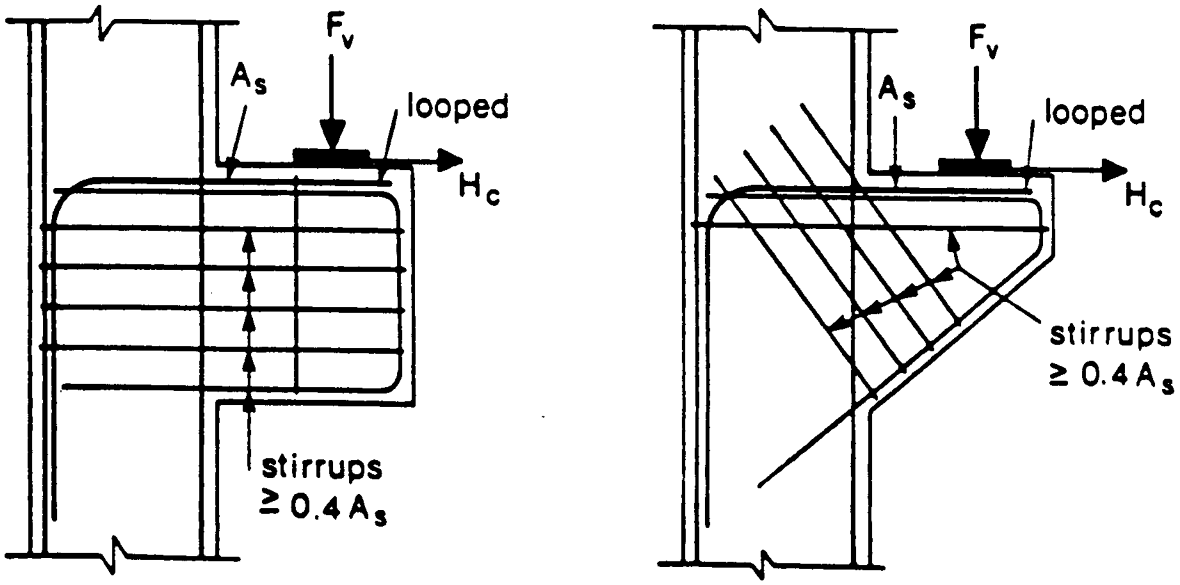

Figure 7.5 Reinforcement of a Corbel

CHAPTER 8
MATERIALS AND WORKMANSHIP

8.1 SCOPE

(1) This Chapter provides minimum specification requirements for materials and for the standard of workmanship that must be achieved on site in order to ensure that the design assumptions in this Code are valid and hence that the intended levels of safety and of durability will be attained.

(2) This Chapter is neither intended as, nor extensive enough for, a contract document.

8.2 SPECIFICATION OF CONCRETE

8.2.1 Methods of Specifying Concrete

(1) Concrete may be specified in one of three ways:

(a) **Designed mixes:** With this method the required compressive strength is specified, together with any other limits that may be required, such as maximum aggregate size, minimum cement content, and workability.
(b) **Prescribed mixes:** With this method, the designer assumes responsibility for designing the mix and stipulates to the producer the mix proportions and the materials which shall be employed.
(c) **Standard (or Nominal) mixes:** The mix proportions which are appropriate for grades C5 to C30 may be taken from Table 8.1. These standard mixes which are rich in cement, and are intended for use where the cost of trial mixes or of acceptance cure testing is not justified, may be used without verification of compressive strength by testing.

(2) The limitation on constituent materials given in Section 8.2.2 shall be complied with.

8.2.2 Constituent Materials of Concrete

8.2.2.1 Cement

(1) The cement used shall be Portland or Portland-Pozzolana cement complying with the requirements of the latest Ethiopian Standards on such cements.

(2) Where cements other than those complying with these standards are used, account shall be taken of their properties and any particular conditions of use.

8.2.2.2 Aggregates

(1) In general aggregates shall comply with the requirements of the latest Ethiopian Standards for aggregates.

8.2.2.3 Water

(1) Mixing water shall be clean and free from harmful matter.

Table 8.1 Standard Mixes for Ordinary Structural Concrete per 50 kg Bag of Cement

Concrete Grade	Nominal max. size of Aggregate (mm)	40		20		14		10	
	Workability	Medium	High	Medium	High	Medium	High	Medium	High
	Limits of slump that may be expected (mm)	30 to 60	60 to 120	20 to 50	50 to 100	10 to 30	30 to 60	10 to 25	25 to 50
C 5	Total Aggregate (kg) Fine Aggregate (%) Vol. of finished concrete (m^3)	640 30 - 45 0.312	550 30 - 45 0.275	540 35 - 50 0.277	480 35 - 50 0.252	–	–	–	–
C 15	Total Aggregate (kg) Fine Aggregate (%) Vol. of finished concrete (m^3)	370 30 - 45 0.200	330 30 - 45 0.183	320 35 - 50 0.178	280 35 - 50 0.160	–	–	–	–
C 20	Total Aggregate (kg) Fine Aggregate (%) Vol. of finished concrete (m^3)	305 30 - 35 0.165	270 30 - 40 0.155	280 30 - 40 0.156	250 35 - 45 0.143	255 35 - 45 0.146	220 40 - 50 0.130	240 40 - 50 0.137	200 45 - 55 0.121
C25	Total Aggregate (kg) Fine Aggregate (%) Vol. of finished concrete (m^3)	265 30 - 35 0.147	240 30 - 40 0.137	240 30 - 40 0.137	215 35 - 45 0.127	220 35 - 45 0.130	195 40 - 50 0.118	210 40 - 50 0.124	175 45 - 55 0.110
C 30	Total Aggregate (kg) Fine Aggregate (%) Vol. of finished concrete (m^3)	235 30 - 35 0.134	215 30 - 40 0.127	210 30 - 40 0.124	190 35 - 45 0.115	195 35 - 45 0.115	170 40 - 50 0.106	180 40 - 50 0.109	150 45 - 55 0.097

8.2.2.4 Admixtures

(1) Suitable admixtures may be used in concrete mixes, in special cases, with the prior approval of the engineer.

(2) Many admixtures are highly active chemicals and may impart undesirable as well as desirable properties to the concrete; their suitability shall generally be verified by trial mixes. Chlorides, in particular, may increase the risk of corrosion.

8.2.3 Composition of the Concrete

(1) The choice of the constituents and of their mix proportions shall be such as to satisfy requirements concerning:

(a) The properties of the fresh concrete (see Section 8.2.4 to 8.2.6)
(b) The specified properties of the hardened concrete (strength or other limit requirements, see Section 8.2.1)
(c) The durability, taking account of the conditions of exposure. In particular, the total content of deleterious substances shall be restricted.

8.2.4 Requirements of Fresh Concrete

8.2.4.1 Workability

(1) The workability of the fresh concrete shall be such that the concrete is suitable for the conditions of handling and placing so that after compaction it surrounds all reinforcement and completely fills the formwork.

8.2.4.2 Temperature

(1) Where the minimum dimension of concrete to be placed at a single time is greater than 600 mm and especially where the cement content is likely to be 400 kg/m^3 or more, measures to reduce the temperature, such as the selection of a cement type with a slower release of heat of hydration shall be considered. In exceptional cases other measures to reduce the temperature or to remove evolved heat may be necessary.

8.2.5 Hot Weather Concreting

8.2.5.1 General

(1) Hot weather is defined as any combination of high air temperature, low relative humidity, and wind velocity tending to impair the quality of fresh or hardened concrete or otherwise resulting in abnormal properties. The effects of hot weather are most critical during periods of rising temperature, falling relative humidity, or both.

(2) Hot weather introduces problems in preparation, placing, and curing cement concrete that can adversely affect the properties and serviceability of the hardened concrete.

8.2.5.2 Placing of Concrete

(1) If concrete temperatures as placed are expected to be abnormally high, preparation shall be made to place, consolidate and finish the concrete at the fastest possible rate.

(2) For best assurance of good results with concrete placing in hot weather, the initial concrete placement should be limited between 25°C and 40°C. Every effort shall be made to keep the concrete temperature uniform.

(3) Under extreme conditions of high ambient temperature, exposure to direct rays of the sun, low relative humidity, and wind, it is suggested to restrict concrete placement to late afternoon or evening.

8.2.5.3 Curing of Concrete

(1) In hot weather there is great need for continuous curing, preferably by water. The need is greatest during the first few hours, and throughout the first day after the concrete is placed.

(2) In hot weather, forms shall be covered and kept moist. The forms shall be loosened, as soon as this can be done without damage to concrete, and provisions made for the curing water to run down inside them. During form removal, care shall be taken to provide wet cover to newly exposed surfaces to avoid exposure to hot sun and wind. At the end of the prescribed curing period (10 days is recommended), the covering shall be left in place without wetting for at least four days, so that the concrete surface will dry slowly and be less subject to surface shrinkage cracking.

8.2.6 Minimum Cement Content

(1) One of the main characteristics influencing the durability of any concrete is its permeability.

(2) With strong, dense aggregates, a suitably low permeability is achieved by having a sufficiently low water/cement ratio, by ensuring complete compaction of the concrete, and by ensuring sufficient hydration of the cement through proper curing methods.

(3) The cement content shall be sufficient to provide adequate workability with low water/cement ratio so that the concrete can be completely compacted with the means available.

(4) Table 8.2 gives the minimum cement content required and maximum net water/cement ratio recommended, when using a particular size of aggregate in Portland cement concrete, to provide acceptable durability under the appropriate conditions of exposure.

(5) The cement contents in Table 8.2 may be reduced by 20 kg/m^3 when trial mixes have verified that a concrete with a maximum net water/cement ratio not greater than that given for the particular condition, can be consistently produced and that it is suitable for the conditions of placing and compaction.

8.2.7 Maximum Cement Content

(1) Cement contents in excess of 550 kg/m^3 shall not be used unless special consideration has been given in design to the increased risk of cracking due to drying shrinkage in thin sections or to thermal stresses in thicker sections.

Table 8.2 Minimum Cement Content per m³ of Concrete to Ensure Durability under Specified Conditions of Exposure

Exposure	Reinforced Concrete					Plain Concrete				
	Nominal Maximum Size of Aggregate				Max *w/c*	Nominal Maximum Size of Aggregate				Max *w/c*
	40	30	20	10		40	30	20	10	
Mild: E.g. Completely protected against weather, or aggressive conditions, except for a brief period of exposure to normal weather conditions during construction	230	260	280	300	0.65	220	230	260	280	0.70
Moderate: E.g. Sheltered from sever rain. Buried concrete and concrete continuous under water	270	300	330	350	0.55	230	260	290	310	0.60
Sever: E.g. Exposed to sea water, driving main alternate wetting and drying. Subject to heavy condensation or corrosive fumes	330	370	400	420	0.45	280	320	340	370	0.50

8.3 SPECIFICATION OF REINFORCEMENT

8.3.1 Basic requirements

(1) Reinforcing steel shall comply with the requirements of Sections 2.6 to 2.10 of this Code. Reinforcing steel shall comply with the requirements of the latest Ethiopian Standards for reinforcement.

(2) Only steel specified in the design documents may be used as reinforcements.

8.4 CONCRETE CONSTRUCTION RULES

8.4.1 General

(1) The supervision employed shall be such as to ensure the required standard of control over materials and workmanship. The engineer shall be afforded all reasonable opportunity and facility to inspect the materials and the manufacture of concrete and to take any samples or to make any tests. All such inspection, sampling and testing shall be carried out with the process of manufacture and delivery.

8.4.2 Handling and Storage of the Materials used for Making Concrete

8.4.2.1 Cement

(1) Cement shall be transported and stored in clean containers and protected from moisture both in transit and during storage.

(2) Provision shall be made to prevent accidental mixing of different types.

8.4.2.2 Aggregates

(1) Aggregates shall be handled and stored so as to minimize segregation and contamination with undesirable constituents. Separate storage facilities with adequate provision for drainage shall be provided for each different nominal size of aggregate used.

8.4.3 Batching and Mixing

(1) The mixing shall be carried out in such a way that the constituent materials are uniformly distributed and the mixture has uniform workability.

(2) The quantity of cement, the quantity of fine aggregate and the quantities of the various sizes of coarse aggregates shall be measured by weight except that aggregates may be measured by volume for Class II Concrete or for standard mixes.

(3) The batch weights of aggregates shall be adjusted to allow for a moisture content typical of the aggregates being used.

8.4.4 Transporting, Placing and Compacting

(1) Concrete shall be transported from the mixer to the formwork as rapidly as practicable by methods which will prevent the segregation or loss of any of the ingredients, and maintain the required workability. It shall be deposited as nearly as practicable in its final position to avoid rehandling.

(2) All placing and compacting shall be carried out under the direct supervision of a competent member of the contractor's (or manufacturer's) staff. Class I concrete of grades C20 and above shall be compacted by using vibrators.

(3) Concrete shall be placed soon after mixing and thoroughly compacted during the operation of placing. It shall be thoroughly worked around the reinforcement, tendons or duct formers, around embedded fixtures and into corners of formwork to form a solid mass free from voids.

(4) Care shall be taken to avoid the displacement of reinforcement or movement of formwork and damage to faces of formwork.

(5) The depth of lift to be concreted shall be determined by the contractor or the manufacturer in consultation with the engineer.

(6) In order to avoid segregation, the free fall of concrete mass shall be restricted to a maximum of three meters unless the system of placing concrete is approved by the designer.

(7) When vibrators are used to compact the concrete,vibration shall be applied continuously during the placing of each batch of concrete until the expulsion of air has practically ceased and in a manner which does not promote segregation of the ingredients.

(8) The mix shall be such that there will not be excess water on the top surface on completion of compaction.

8.4.5 Construction Joints

(1) The number of construction joints shall be kept as few as possible consistent with reasonable precautions against shrinkage. Concreting shall be carried out continuously up to construction joints.

(2) Where it is necessary to introduce construction joints, careful consideration shall be given to their exact location, which shall be indicated on the drawings. Alternatively, the location and details of joints shall be subject to the agreement between the engineer and the contractor before any work commences. Construction joints shall be at right angles to the general direction of the member and shall take due account of shear and other stresses.

(3) Particular care shall be taken in the placing of the new concrete close to the joint. The surface of concrete construction joints shall be thoroughly cleaned and laitance removed. Immediately before new concrete is placed, all construction joints shall be wetted and standing water removed.

8.4.6 Formwork

8.4.6.1 Basic Requirements

(1) Formwork and falsework shall be designed and constructed so that they are capable of resisting all actions which may occur during the construction process. They shall remain undisturbed until the concrete has achieved sufficient strength to withstand the stresses to which it will be subjected on stripping or release, with an acceptable margin of safety.

(2) The formwork and falsework shall be sufficiently stiff and tight to ensure that the tolerances for the structure are satisfied and that its loadbearing capacity is not affected and to prevent loss of grout or mortar from the concrete at all stages and for the appropriate method of placing and compacting.

(3) The general lay-out of the formwork shall be such that the correct placing of reinforcement as well as correct compaction of the concrete is possible.

(4) The formwork and the falsework shall be designed and erected by suitably trained persons. Supervision and control shall be such as to ensure that the erection is completed in accordance with the drawings and specifications.

(5) The formwork shall be capable of being removed from the concrete without causing shock or damage.

(6) Where necessary, the camper built into the formwork should be that required by the designer of the structure and falsework.

(7) Ground support for the falsework should also be constructed by suitably trained personnel in accordance with the drawings and specifications. Deformations and displacements imposed by prestressing should be taken into account in the design of the falsework.

(8) Joints between the panels of the formwork should be adequately tight.

(9) The internal surface of the formwork must be clean. Approved mould-release agents should be applied in continuous and uniform layers on the internal surface and the concrete should then be placed while these agents are still effective. Any possible detrimental influence of these agents on the concrete surface has to be taken into consideration.

(10) Formwork spacers left in the concrete should not impair its durability or appearance.

8.4.6.2 Surface Finish

(1) The formwork shall be designed and constructed so that there is no loss of fines, or blemish of the concrete surface.

(2) Where a particular grade or type of finish is required for practical or aesthetic reasons, the requirements shall be specified directly or by reference to appropriate national or international documents or by sample surfaces.

8.4.6.3 Temporary Work Inserts

(1) Temporary works inserts may be necessary to assist in maintaining formwork, or bar reinforcement or ducts or other similar items, in place until the concrete has hardened.

(2) Such inserts shall not introduce unacceptable loading on the structure, shall not react harmfully with the constituents of the concrete or reinforcement , and shall not produce unacceptable surface blemishes.

8.4.6.4 Removal of Formwork and Falsework

(1) The formwork shall be removed slowly, as the sudden removal of wedges is equivalent to a shock load on the partly hardened concrete.

(2) The time at which formwork and falsework is removed shall be determined by consideration of the following criteria:

(a) The stresses that will be induced in the concrete when the formwork/falsework has been removed;
(b) The concrete strength at the time of removal;
(c) The ambient climatic conditions and the measures available to protect the concrete once the formwork is removed;
(d) The presence, or otherwise, of re-entrant angle formwork, which should be removed as soon as possible, while complying with other removal criteria.

(3) The formwork shall not be removed before the structure has gained enough strength to safely carry all the possible loads. The time at which formwork is struck will be influenced by the following factors:

(a) concrete strength
(b) stresses in the concrete at any stage in the construction period
(c) curing (Section 8.4.7)
(d) subsequent surface treatment requirements
(e) presence of re-entrant angles requiring formwork to be removed as soon as possible after concrete has set to avoid shrinkage cracks.

(4) Provided the concrete strength is confirmed by tests on cubes stored as far as possible under the same conditions, formwork supporting cast-in-situ concrete may be removed when the cube strength is 50% if the nominal strength or twice the stress to which it will then be subjected whichever is greater, provided that such earlier removal will not result in unacceptable deflections such as due to shrinkage and creep.

(5) The time between casting and removal of the formwork depends mainly on the strength development of the concrete and on the function of the formwork. In the absence of more accurate data, the following minimum periods are recommended:

(a)	For non-load bearing parts of formwork (e.g. vertical formwork of beams; formwork for columns and walls)	18 hours
(b)	For soffit formwork to slabs	7 days
(c)	For props to slabs	14 days
(d)	For soffit formwork to beams	14 days
(e)	For props to beams	21 days

(5) Where sliding or climbing formwork is used, shorter periods than those recommended above may be permitted.

8.4.7 Curing

(1) The methods of curing and their duration shall be such that the concrete will have satisfactory durability and strength and the member will suffer a minimum of distortion, be free of excessive efflorescence and will not cause, by its shrinkage, undue cracking in the structure.

8.5 REINFORCING STEEL CONSTRUCTION RULES

8.5.1 Transport, Storage and Fabrication of the Reinforcement

(1) Steel reinforcing bars, welded mesh reinforcement and prefabricated reinforcement cages shall be transported, stored, bent and placed in position so that they do not suffer any damage.

(2) The surface condition of the reinforcement shall be examined prior to use, to ensure that it is free from deleterious substances which may adversely affect the steel or concrete or the bond between them.

(3) Reinforcing steel shall be cut and bent in accordance with appropriate international or national standards.

(4) The following should be avoided:

(a) Mechanical damage (e.g. notches of dents);
(b) Rupture of welds in prefabricated reinforcement cages and in welded fabrics;
(c) Surface deposits damaging bond properties;
(d) Lack of identification of reinforcement;
(e) Reduction of the section through corrosion, beyond certain permissible limiting values.

8.5.2 Surface Condition

Reinforcement shall not be surrounded by concrete unless it is free from mud, oil, paint, retarders, loose rust, loose mill scale, grease or any other substance which can be shown to affect adversely the steel or concrete chemically, or reduce bond.

8.5.3 Welding

(1) Welding must only be carried out on reinforcing steel that is suitable for welding.

(2) Welding connections must be made and checked by persons suitably trained in welding of reinforcement.

(3) Welding shall be used in accordance with international or national standards.

(4) Where a risk of fatigue exists, the welding of reinforcement must conform to special requirements as given in relevant standards.

(5) The production and checking of the welded connections shall comply with the relevant requirements in international or national standards.

(6) Welding methods permitted include:
- (a) electric flash welding;
- (b) electric resistance welding
- (c) electric arc welding (with coated electrodes or under a protective gas envelope);
- (d) high pressure gas welding.

8.5.4 Joints

(1) The length and position of lapped joints shall be in accordance with the design and the drawings. If the bar lengths delivered to the site do not conform with the drawings, then modifications shall only be introduced with the approval of the designer or of the supervisory authority.

(2) In general, reinforcing bars shall not be welded at or near bends in a bar.

8.5.5 Fabrication, Assembly and Placing of the Steel

(1) The assembly of the reinforcement shall be robust enough to ensure that the bars do not shift their prescribed position during transportation, placing and concreting. The specified cover to the reinforcement shall be maintained by the use of approved chairs and spacers.

(2) The tolerances required for the fixing of reinforcement shall be as given in Section 8.2. Alternatively, they shall be stated in the contract documents.

(3) Bending should be carried out by mechanical methods, at constant speed without jerking, with the aid of mandrels so that the bent part has a constant curvature. If the ambient temperature is lower than a specified value, additional precautions may be needed.

(4) The reinforcement shall be secured against any displacement and the position of the reinforcement shall be checked before concreting.

(5) In areas of congested reinforcement, sufficient spacing of the bars shall be provided to allow proper compaction of the concrete.

8.6 TOLERANCES

8.6.1 General

(1) In order to ensure the required properties of the structure, the tolerances must be clearly defined before construction work starts.

(2) For durability reasons, independently from the defined tolerances, the cover to reinforcements shall not be less than the minimum values given in Section 7.1.

(3) The dimensions given on the working drawings shall be observed with the appropriate tolerance.

8.6.2 Tolerances with regard to Structural Safety

(1) The following permitted deviations Δl with respect to the nominal cross sectional dimension l can (except for concrete cover, see Section 8.6.3 below) be regarded as admissible on the basis of the partial safety coefficients γ_F and γ_M as given in Sections 3.5.3.1 and 3.6.1, respectively.

(2) In relation to the dimensions of the concrete section (total depth of a beam or of a slab, width of a beam or web, lateral dimensions of a column) and in relation to the effective depth:

for $l \leq 150$ mm

$$\Delta l = \pm\ 5 \text{ mm} \qquad (8.1)$$

for $l \leq 400$ mm

$$\Delta l = \pm\ 15 \text{ mm} \qquad (8.2)$$

for $l \geq 2500$ mm

$$\Delta l = \pm\ 30 \text{ mm} \qquad (8.3)$$

with linear interpolation for other values of l.

(3) Tolerances other than those defined in (1) above can also be specified provided that it can be demonstrated that they do not reduce the required level fo safety.

8.6.3 Tolerances for Concrete Cover

(1) For the tolerances of concrete cover to reinforcement, e.g. the difference between the nominal and the minimum cover, Section 7.1 (8) applies. No positive permitted deviation is specified.

8.6.4 Tolerances for Construction Purposes

(1) For other purposes, e.g. construction or dimensional tolerances in buildings as a whole, stricter tolerances than defined is Section 8.6.2 may required. These values should be specified separately from this Code. For the maximum sag of slabs, however Section 5.2.2 (1) and (2) apply.

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CHAPTER 9
QUALITY CONTROL

9.1 DEFINITIONS

(1) Quality Control: Comprises a combination of actions and decisions taken in compliance with specifications and checks to ensure that these are satisfied. Quality control consists of two distinct, but interconnected parts, namely production control and compliance control.

(2) Production Control: Comprises a combination of actions and decisions taken during production to check the operation and to obtain a reasonable assurance that the specifications will be satisfied.

(3) Compliance Control: Comprises a combination of actions and decisions, in accordance with compliance rules adopted in advance, to check the compliance of the product with the specifications.

9.2 PRODUCTION CONTROL

9.2.1 Inspection of Materials

(1) Inspection of materials on site shall be made at delivery to check compliance with the specifications and the requirements of this Code (Chapter 8).

9.2.2 Inspection Prior to Concreting

(1) This inspection shall be made to check:

(a) the rigidity of the scaffolding and shuttering
(b) the leak-tightness of joints between formwork elements
(c) conformity of the dimensions of the formwork with the drawings
(d) the cleanliness of the formwork
(e) the surface condition of the reinforcement
(f) the position and size of reinforcement
(g) the rigidity of the reinforcement securing systems, and the quality of the joints between bars.

9.2.3 Control of Mixing, Transportation and Placement of Concrete

(1) The accuracy of the mix proportions shall be checked regularly. The consistency of the fresh concrete shall be checked periodically with the slump test.

(2) During concreting, checks shall be made on the deformations of the formwork and its supporting structure and on any leakage of water.

9.2.4 Control for Curing the Concrete

(1) It must be checked that curing complies with approved method curing depending on the environment and on any special requirements.

9.2.5 Information of Construction Procedures

(1) A site book shall be kept and for large structures, it shall contain the following information:

(a) dates on which concreting and stripping of formwork has taken place
(b) acceptance of materials and components
(c) results of tests and measurements
(d) concrete mix used (type and origin of cement and aggregates)
(e) inspection and measurement reports of the positioning of reinforcement
(f) important instructions received
(g) description of any incidents.

9.3 COMPLIANCE CONTROLS

9.3.1 Compliance Controls for Concrete

(1) Compliance with specified properties of concrete shall be judged by tests made on proper specimens at an age of 28 days unless there is evidence, satisfactory to the authority having jurisdiction, that a particular testing regime is capable of predicting the strength at 28 days of concrete tested at an earlier age, in which case compliance may be based on the results of such tests alone.

(2) The concrete for the specimen shall normally be taken when the concrete is actually being poured.

(3) Compliance of prescribed and standard mixes (Section 8.2) shall be based on checks made on the mix properties (such as aggregate gradation, cement content, mix proportions, and workability); but, because strength tests provide an implicit check on the quality of the mix, they may, alternatively, be used for the acceptance of concretes made with prescribed and standard mixes.

9.3.1.1 *Sampling and Testing Methods*

(1) In general, it is sufficient to make only one test specimen from a single representative sample for each mix of concrete. If more than one specimen is taken, the mix shall be considered as being represented by the mean value of the test results obtained from the various specimens.

(2) Each mix from which a sample is taken shall be chosen at random from among the possible mixes.

(3) The samples shall, where practicable, be taken at the point of discharge from the mixer or, in the case of ready-mix concrete, at the point of discharge from the delivery vehicle.

9.3.1.2 *Size of Lot and Frequency of Sampling*

(1) The lot is defined as the quantity of concrete produced in the same essential conditions and subjected to individual assessment.

(2) The lots shall be defined before the commencement of construction, by taking into account the number of tests required for a decision (see Section 9.3.1.3) as well as the frequency of sampling and testing to be adopted.

(3) The minimum rate of sampling shall be decided by the engineer taking into account the nature of the work. Higher rates would be appropriate at the start of the work, to establish quickly the level of quality, or during periods of production when quality is in doubt, or for highly-stressed structural elements.

(4) In general, the following may be adopted as the minimum requirement on size of lot and frequency of sampling, except for the special cases given hereunder:

(a) No individual sampling can represent, on the average, more than 100 mixes or 100 m^3, whichever is the smaller volume of concrete.
(b) For each grade of concrete, at least one sample shall be taken every week
(c) For each grade of concrete, at least two lots shall be made.

(4) Exception: For small buildings (e.g., having a total volume of less than 100 m^3 of concrete) using concrete grade C30 or lower, Condition (3) need not be complied with.

9.3.1.3 Compliance Criteria

(1) Two compliance criteria are envisaged:

(2) **Criterion 1:** This criterion may be applied in all cases but is less suited to large-scale sampling. Each lot is represented by three samples, the strength of which are $x_1 < x_2 < x_3$.

(3) The lot is accepted automatically if the following conditions are satisfied simultaneously:

$$\bar{m}_3 \geq f_{ck} + k_1 \quad (9.1)$$

$$\bar{x}_1 \geq f_{ck} - k_2 \quad (9.2)$$

where,
$\bar{m}_3$ is the mean value
f_{ck} is the specified characteristic strength
k_1, k_2 are the margins of strength given in Table 9.1
$\bar{x}_1$ is the average strength of the minimum strengths for the several lots.

Table 9.1 Margins of Strength in MPa

Margin of Strength	First two lots	Third and fourth lot	Fifth lot and above
k_1	5	4	3
k_2	1	2	3

(4) **Criterion 2:** This criterion is suitable for large lots.

(5) Each lot represented by not less than 15 test specimens ($n \geq 15$)

(6) The lot is accepted automatically if the following conditions are satisfied simultaneously:

$$\bar{m}_n - \lambda s_n \geq f_{ck} \quad (9.3)$$

$$\bar{x}_1 \geq f_{ck} - k_2 \quad (9.4)$$

where,
$\bar{m}_n$ is the mean value
s_n is the standard deviation of the set of sample results
f_{ck} is the characteristic strength

λ is the coefficient (may be taken as 1.4)
k_2 is the margin of strength (may be taken as 4 MPa)
n is the number of specimen

(7) If the test results do not satisfy the requirements of the selected acceptance criterion, measures specified in Section 9.4 shall be taken.

9.3.2 Compliance Controls for the Completed Structure

(1) The acceptance of a completed structure involves a decision on each portion of the work subject to acceptance (corresponding to the concrete lots) and a decision on the behavior of the structure as a whole.

9.4 MEASURES TO BE TAKEN IN CASE OF NON-COMPLIANCE

9.4.1 General

(1) If the quality of the structure is found to be in doubt after an inspection or from the test results, then a special examination shall be made to verify the soundness of the information received and to assess the actual strength of the structure as constructed with possible recourse to more accurate methods of calculation.

9.4.2 Sequence of Measures

(1) The following sequential measures shall be taken where the results of compliance control tests or inspection are unsatisfactory:

(a) The position of concrete which does not fulfil the compliance criterion shall be identified.
(b) The structural safety shall be checked by appropriate calculations on the basis of the actual test results which did not comply. If safety is assured, the concrete can be accepted.
(c) If such structural safety or durability are not assured, then the strength of the concrete shall be examined by taking drilled cores or by non-destructive methods (see Section 9.4.3). The results of such tests shall be assessed on the basis of the prescribed acceptance criterion, taking into account any differences in age.
(e) If this new information shows that structural safety is assured, the concrete may be accepted after it has been decided whether repairs are necessary to ensure durability.
(f) If the results of check tests by non-destructive methods (3) show that the quality of concrete is inadequate or show other defects, the engineer may require a loading test to be made which shall then be carried out in accordance with Section 9.4.4.
(g) If structural safety and durability are not assured, then the possibility of strengthening the structure must be investigated. If strengthening is not feasible, then the concrete shall be rejected, and the structure or member demolished or given a reduced structural grading by limiting its service rating, as appropriate.

9.4.3 Check Tests on Structural Concrete

9.4.3.1 *General*

(1) Check tests by non-destructive methods are applicable to hardened concrete in the finished parts of a structure or in precast units. They may be used in routine inspection for quality control. They are also of use when concrete is found defective from visual inspection and when low cube strengths are obtained when assessing the strength of the concrete used.

9.4.3.2 *Types of Check Tests*

(1) The following types of check tests may be used for different types of checks:
- (a) Drilled Cores
- (b) Gamma radiography
- (c) Ultrasonic test
- (d) Electromagnetic cover measuring devices
- (e) Rebound hammer test.

(2) The tests must be conducted by appropriately trained personnel and the accuracy of each type of tests must be considered in interpreting the results obtained.

9.4.4 Load Tests of Structure or Parts of Structures

9.4.4.1 *General*

(1) Test loads are to be applied and removed incrementally.

(2) The test should be carried out after the expiry of 28 days from the time of placing concrete. When the test is for a reason other than the quality of the concrete in the structure being in doubt, the test may be carried out earlier, provided that the concrete has already reached its specified strength.

9.4.4.2 *Test Loads*

(1) The test loads to be applied for the limit states of deflection and local damage are the appropriate design loads, i.e., the characteristic dead and imposed loads.

(2) When the ultimate limit state is being considered, the test load shall be equal to the sum of the characteristic dead load plus 1.25 times the characteristic imposed load and shall be maintained for a period of 24 hours.

(3) If any of the final dead load is not in position on the structure, compensating loads shall be added as necessary.

(4) During the test, struts and bracing, strong enough to support the whole load, shall be placed in position leaving a gap under the members to be tested, and adequate precautions shall be taken to safeguard persons in the vicinity of structure.

9.4.4.3 *Measurements During the Tests*

(1) Measurements of deflection and crack width shall be taken immediately after the application of load and in the case of 24 hours sustained load test, at the end of the 24 hours loaded period, after removal of the load and after the 24 hours recovery period. Sufficient measurements shall be taken to enable side effects to be taken into account. Temperature and weather conditions shall be recorded during the test.

9.4.4.4 ***Assessment of Results***

(1) In assessing the serviceability of a structure or part of a structure following a loading test, the possible effects of variation in temperature and humidity during the period of the test shall be considered.

(2) The following requirements shall be met:

(a) The maximum width of any crack measured immediately on application of the test load for local damage shall not be more than two-thirds of the value for the limit state requirement (see Section 5.3.4)

(b) For members spanning between two supports, the deflection measured immediately after application of the test load for deflection is to be not more than 1/500 of the effective span. Limits shall be agreed upon before testing cantilevered portions of structures.

(c) If the maximum deflection in millimeters observed during 24 hours under load is less than 40 L_e^2/h where L_e is effective span in meter and h the overall depth of construction in millimeters, it is not necessary for the recovery to be measured and the requirements in item (d) below do not apply.

(d) If, within 24 hours of the removal of the test load for the ultimate limit state as calculated in Section 9.4.4.2, a structure does not show a recovery of at least 75% of the maximum deflection shown during the 24 hours under load, the loading shall be repeated. The structure shall be considered to have failed to pass the test if the recovery after the second loading is not at least 75% of the maximum deflection observed during the second loading.

APPENDIX A
ANALYSIS OF SLABS

A.1 SCOPE

(1) This appendix provides methods of analysis for one-way slabs, two-way slabs and flat slabs which are based on the principles set out on Section 3.8.

A.2 ONE-WAY SLABS

A.2.1 General

(1) One-way slabs transmit their load mainly in one direction (i.e. the direction of span). There is no need to analyze the action effects transverse to the direction of span arising as a result of restrained lateral strain or the transverse distribution of concentrated or line loads, or caused by a support parallel to the direction of span, which has not bean taken into account in the calculation. These effects shall, however, be taken into account by making suitable detailing provisions.

A.2.2 Distribution of Concentrated Loads

(1) The width of slab which may be assumed to be effective in carrying a concentrated load may be taken as follows:

(a) For solid slabs, the effective width may be taken as the sum of the load width and $2.4x(1-x/L)$ where x is the distance from the nearer support to the section under consideration and L is the span.

(b) For other slabs, except where specially provided for, the effective width will depend on the ratio of the transverse and longitudinal flexural rigidities of the slab. When these are approximately equal, the value for the effective width as given for solid slabs may be used, but as the ratio decreases a smaller value shall be taken. The minimum value which need be taken, however, is the load width plus $4x/L(1 - x/L)$ meters where x and L are as defined in (a) above so that, for a section at mid-span, the effective width is equal to 1.0 meter plus the load width.

(c) Where the concentrated load is near an unsupported edge of a slab the effective width shall not exceed the value in (a) or (b) above as appropriate, nor half that value plus the distance of the center of the load from the unsupported edge (see Fig. A-1).

A.3 TWO-WAY SLABS

A.3.1 General

(1) The type of slab dealt with here is one composed of rectangular panels supported at all four edges by walls or beams stiff enough to be treated as unyielding. This may be assumed to be the case if the requirements for the ratio between the depth of a beam and its span are in accordance with Fig. A-2.
(2) These methods are intended for slabs with uniformly distributed loads. If a slab is subjected to concentrated or line loads, in addition to a uniform load, these can generally be treated by considering them as equivalent uniform loads using approximate rules, provided that the sum of the non-uniform loads on a panel does not exceed 20 percent of the total load.

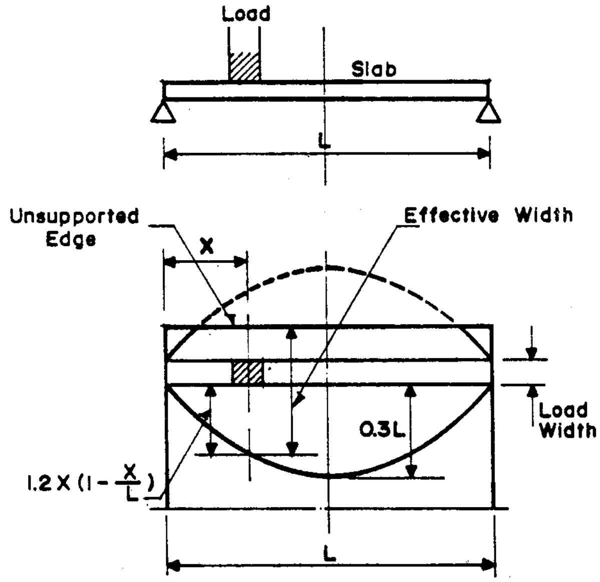


Figure A-1 Effective Width of Solid Slab Carrying a Concentrated Load near an Unsupported Edge

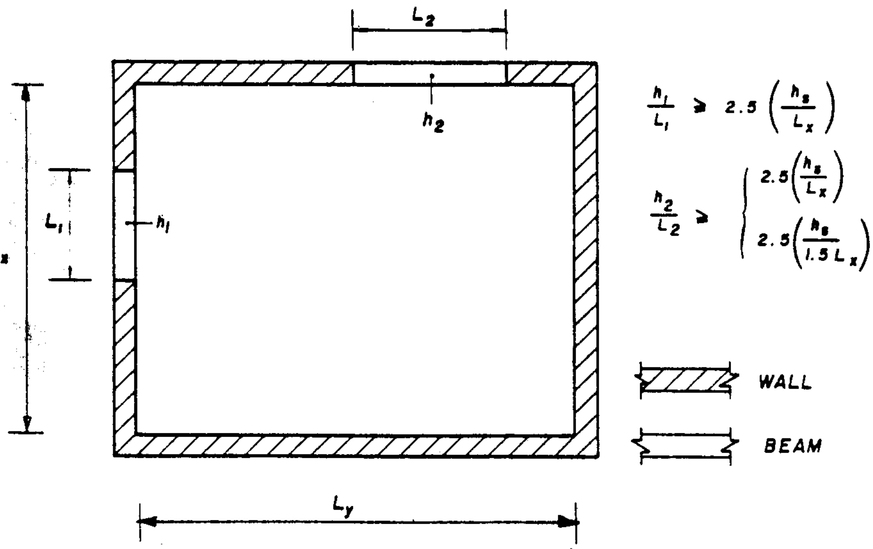


Figure A-2 Support for Two-Way Slabs

A.3.2 Individual Panel Moments

(1) Moments for individual panels with edges either simply supported or fully fixed are calculated as:

$$m_i = \alpha_i (g_d + q_d) L_x^2 \qquad \text{(A-1)}$$

where
- m_i is the design moment per unit width at the point of reference
- α_i is the coefficient given in Table A-1 as function of aspect ratio L_y/L_x and support conditions
- g_d is the uniformly distributed design permanent load
- q_d is the uniformly distributed design live load
- L_x is the shorter span of the panel
- L_y is the longer span of the panel

Subscripts for moments and moment coefficients (α_i) have the following meanings:

- s support
- f field (span)
- x direction of shorter span
- y direction of longer span

(2) Notations for different critical moments and edge numbers are shown in Fig. A-3. Division of slab into middle and edge strips is illustrated in Fig. A-4.

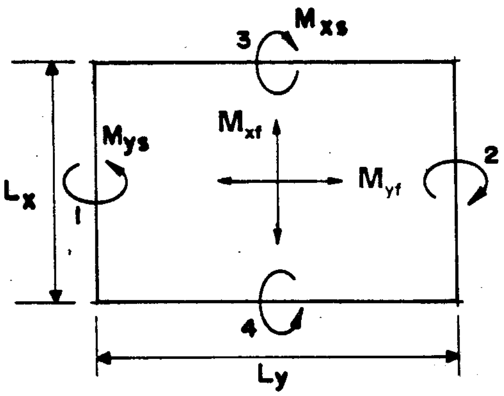


Figure A-3 Notations for Critical Moments

(3) The positive moment coefficients in Table A-1 may be derived from the following equations. The negative moment coefficients are taken as 4/3 times the positive moment coefficients for the same direction.

$$\alpha_{yf} = \frac{(24 + 2n_d + 1.5n_d^2)}{1000} \tag{A.2}$$

$$\alpha_{xf} = \frac{\beta}{(\sqrt{1 + r_3} + \sqrt{1 + r_4})^2} \tag{A.3}$$

$$\beta = \frac{2}{3}\{1 - \frac{L_x}{L_y}\sqrt{2\alpha_{yf}}\,(\sqrt{1 + r_1} + \sqrt{1 + r_2})\} \tag{A.4}$$

where n_d is the number of discontinuous edges ($0 \le n_d \le 4$)

r_1, r_2, r_3, r_4 are the ratios of negative moment capacity at edges 1 to 4, respectively, to the span moment capacity in the same direction and take values of 4/3 for continuous edges or zero for discontinuous edges.

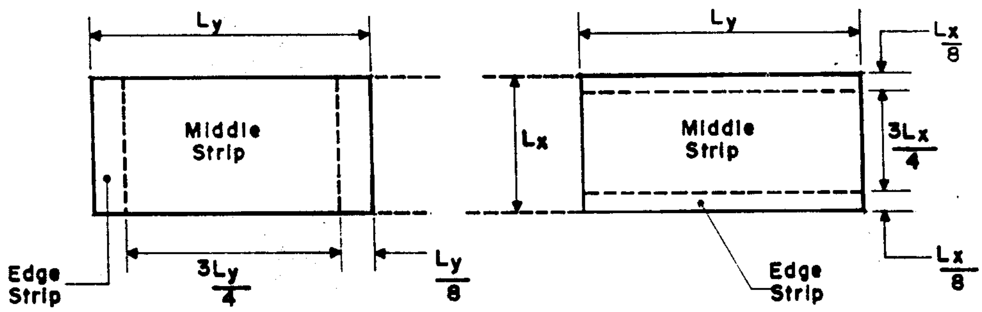


Figure A-4 Division of Slab into Middle and Edge Strips

(4) Slabs are considered as divided in each direction into middle strips and edge strips as shown in Fig. A-4, the middle strip being three quarters of the width and each edge strip one eighth of the width.

(5) The maximum design moments calculated as above apply only to the middle strips and no redistribution shall be made.

(6) Reinforcement in the middle strips shall be detailed in accordance with Section 7.1.7.

(7) Reinforcement in an edge strip, parallel to the edge, need not be less than the minimum given in Section 7.2.2.2 (minimum areas of tension reinforcement).

A.3.3 Moments in Continuous Slabs

A.3.3.1 General

(1) The first stage of design is to determine support and span moments for all panels individually by treating their edges as either simply supported or fully fixed (see Section A.3.2). External edges are generally considered as simply supported and continuous edges are considered as fully fixed in this stage.

(2) If the slab is connected with an external wall or if any of its edges is partly fixed and partly simply supported, the following procedure may be adopted:

(a) The ratio of the actual support moment to the bending moment of fully fixed slab, or the ratio of the width of fixed part to the width of the simply supported part of the edge is evaluated.
(b) The bending moments of the slab are then computed by interpolating between different support conditions.

(3) For each support over which the slab is continuous there will thus generally be two different support moments. The difference may be distributed between the panels on either side of the support to equalize their moments, as in the moment distribution method for frames.

(4) Two methods of differing accuracy, are given here for treating the effects of this redistribution on moments away from the support.

A.3.3.2 Method I

(1) Method I may be used:

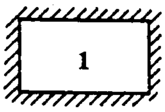
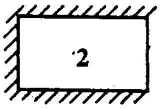
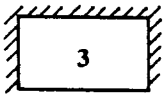
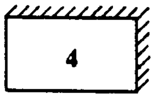
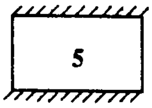
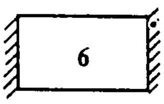
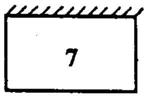
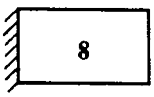
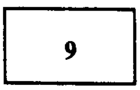
(a) When differences between initial support moments are less than 20 percent of the larger moment, and
(b) only for internal structures where the live load does not exceed 2.5 times the permanent load ($q_k \leq 2.5g_k$) or 0.8 times the dead load for external structures ($q_k \leq 0.8g_k$).

In other cases either Method II or other more accurate methods shall be used.

(2) When Method I is used, dimensioning is normally carried out either using:

(a) Initial moments directly, or
(b) based on the average initial moment at the support.

Table A-1 Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

Support Condition	Coeff.	Values of L_y/L_x								Long span coefficients, α_{ys} and α_{yf}, for all values of L_y/L_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
 1	α_{xs} α_{xf}	0.032 0.024	0.037 0.028	0.042 0.032	0.046 0.035	0.050 0.037	0.053 0.040	0.059 0.044	0.063 0.048	0.032 0.024
 2	α_{xs} α_{xf}	0.039 0.029	0.044 0.033	0.048 0.036	0.052 0.039	0.055 0.041	0.058 0.043	0.063 0.047	0.067 0.050	0.039 0.029
 3	α_{xs} α_{xf}	0.039 0.030	0.049 0.036	0.056 0.042	0.062 0.047	0.068 0.051	0.073 0.055	0.082 0.062	0.089 0.067	0.039 0.030
 4	α_{xs} α_{xf}	0.047 0.036	0.056 0.042	0.063 0.047	0.069 0.051	0.074 0.055	0.078 0.059	0.087 0.065	0.093 0.070	0.047 0.036
 5	α_{xs} α_{xf}	0.046 0.034	0.050 0.038	0.054 0.040	0.057 0.043	0.060 0.045	0.062 0.047	0.067 0.050	0.070 0.053	- 0.034
 6	α_{xs} α_{xf}	- 0.034	- 0.046	- 0.056	- 0.065	- 0.072	- 0.078	- 0.091	- 0.100	0.045 0.034
 7	α_{xs} α_{xf}	0.057 0.043	0.065 0.048	0.071 0.053	0.076 0.057	0.081 0.060	0.084 0.063	0.092 0.069	0.098 0.074	- 0.044
 8	α_{xs} α_{xf}	- 0.044	- 0.054	- 0.063	- 0.071	- 0.078	- 0.084	- 0.096	- 0.105	0.058 0.044
 9	α_{xf}	0.056	0.065	0.074	0.081	0.087	0.092	0.103	0.111	0.056

A.3.3.3 Method II

(1) In this method consideration of the effects of changes of support moments is limited to the adjacent spans. Since no effects on neighboring support sections need be considered, only a simple balancing operation is required at each edge and no iterative process is involved.

(2) The procedure for applying Method II, is as follows:

(a) Support and span moments are first calculated for individual panels by assuming each panel to be fully loaded. This is done by using the coefficients given in Table A-1 as described in Section A.3.2.

(b) The unbalanced moment is distributed using the moment distribution method. The relative stiffness of each panel shall be taken proportional to its gross moment of inertia divided by the smaller span.

(c) If the support moment is decreased, the span moments m_{xf} and m_{yf} are then increased to allow for the changes of support moments. This increase is calculated as being equal to the change of the support moment multiplied by the factors given in Table A-2. If a support moment is increased, no adjustment shall be made to the span moments.

A.3.4 Elastic Values of Support Moments

(1) The above methods give average values of support moments. In cases where maximum elastic moments should be considered (e.g. in watertight structures), elastic theory must be used.

A.3.5 Loads on Supporting Beams

(1) The design loads on beams supporting solid slabs spanning in two directions at right angles supporting uniformly distributed loads may be assessed from the following equations:

$$V_x = \beta_{vx}(g_d + q_d)L_x \tag{A.5}$$

$$V_y = \beta_{vy}(g_d + q_d)L_x \tag{A.6}$$

(2) Table A-3 gives values of load transfer coefficients. The assumed distribution of the load on a supporting beam is shown in Fig. A-5.

(3) The design load on a beam determined in accordance with (1) and (2) above, may be taken as the maximum shear in the slab at the center of support.

Table A-2 Factors for Adjusting Span Moments m_{xf} and m_{yf}

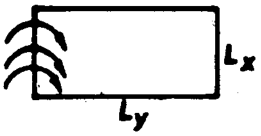
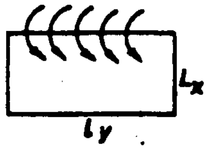
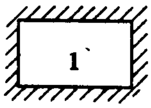
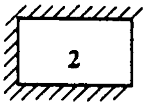
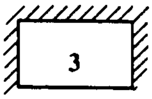
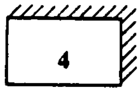
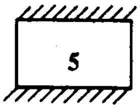
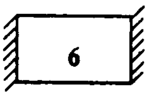
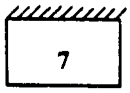
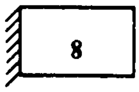
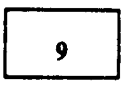
L_y/L_x				
	c_x	c_y	c_x	c_y
1.0	0.380	0.280	0.280	0.380
1.1	0.356	0.220	0.314	0.374
1.2	0.338	0.172	0.344	0.364
1.3	0.325	0.135	0.373	0.350
1.4	0.315	0.110	0.398	0.331
1.5	0.305	0.094	0.421	0.310
1.6	0.295	0.083	0.443	0.289
1.7	0.285	0.074	0.461	0.272
1.8	0.274	0.066	0.473	0.258
1.9	0.258	0.060	0.481	0.251
2.0	0.238	0.055	0.484	0.248

Table A-3 Shear Force Coefficients for Uniformly Loaded Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners

Type of panel and location	Edge	β_{vx} for values of L_y/L_x								β_{vy}
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
 1	Continuous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
 2	Continuous	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
	Discontinous	-	-	-	-	-	-	-	-	0.24
 3	Continuous	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
	Discontinous	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	-
 4	Continuous	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
	Discontinous	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
 5	Continuous	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	-
	Discontinous	-	-	-	-	-	-	-	-	0.26
 6	Continuous	-	-	-	-	-	-	-	-	0.40
	Discontinous	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	-
 7	Continuous	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	-
	Discontinous	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.30
 8	Continuous	-	-	-	-	-	-	-	-	0.45
	Discontinous	0.30	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
 9	Discontinous	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

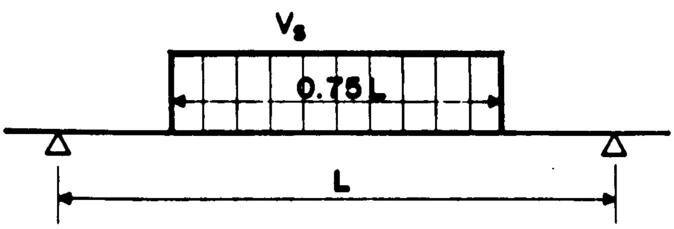


Figure A-5 Distribution of Load on a Beam Supporting a Two-Way Spanning Slab

A.4 FLAT SLABS

A.4.1 Scope

(1) The provision given in this chapter are for the design of flat slabs supported by a generally rectangular arrangement of columns and where the ratio of the longer to the shorter spans does not exceed 2.

A.4.2 Definitions

(1) Column strip is a design strip with a width on each side of a column center-line equal to $0.25L_x$ or if drops with dimension not less than $L_x/3$ are used, a width equal to the drop dimension.

(2) Middle strip is a design strip bounded by two column strips.

(3) The division of panels in flat slabs into column and middle strip is illustrated in Fig. A-6.

A.4.3 Analysis of Flat Slab Structures

A.4.3.1 General

(1) A flat slab including supporting columns or walls may be analyzed using the equivalent frame method (Section A.4.3.2) or, where applicable, the simplified method (Section A.4.3.3).

(2) For both methods of analysis, the negative moments greater than those at a distance $h_c/2$ from the center-line of the column may be ignored provided the moment M_o obtained as the sum of the maximum positive design moment and the average of the negative design moments in any one span of the slab for the whole panel width is such that:

$$M_0 \geq \frac{(g_d + q_d)L_2}{8}\left(L_1 - \frac{2h_c}{3}\right)^2 \qquad \text{(A.7)}$$

where L_1 is the panel length parallel to span, measured from centers of columns
L_2 is the panel width, measured from centers of columns
h_c is the effective diameter of a column or column head (see (3) below).

When the above condition is not satisfied, the negative design moments shall be increased.

(3) The effective diameter of a column or column head h_c is the diameter of a circle whose area equals the cross-sectional area of the column or, if column heads are used, the area of the column head based on the effective dimensions as defined in (4) below. In no case shall h_c be taken as greater than one-quarter of the shortest span framing into the column.

(4) The effective dimensions of a column head for use in calculation of h_c (see (3) above) are limited according to the depth of the head. In any direction, the effective dimension of a head L_h shall be taken as

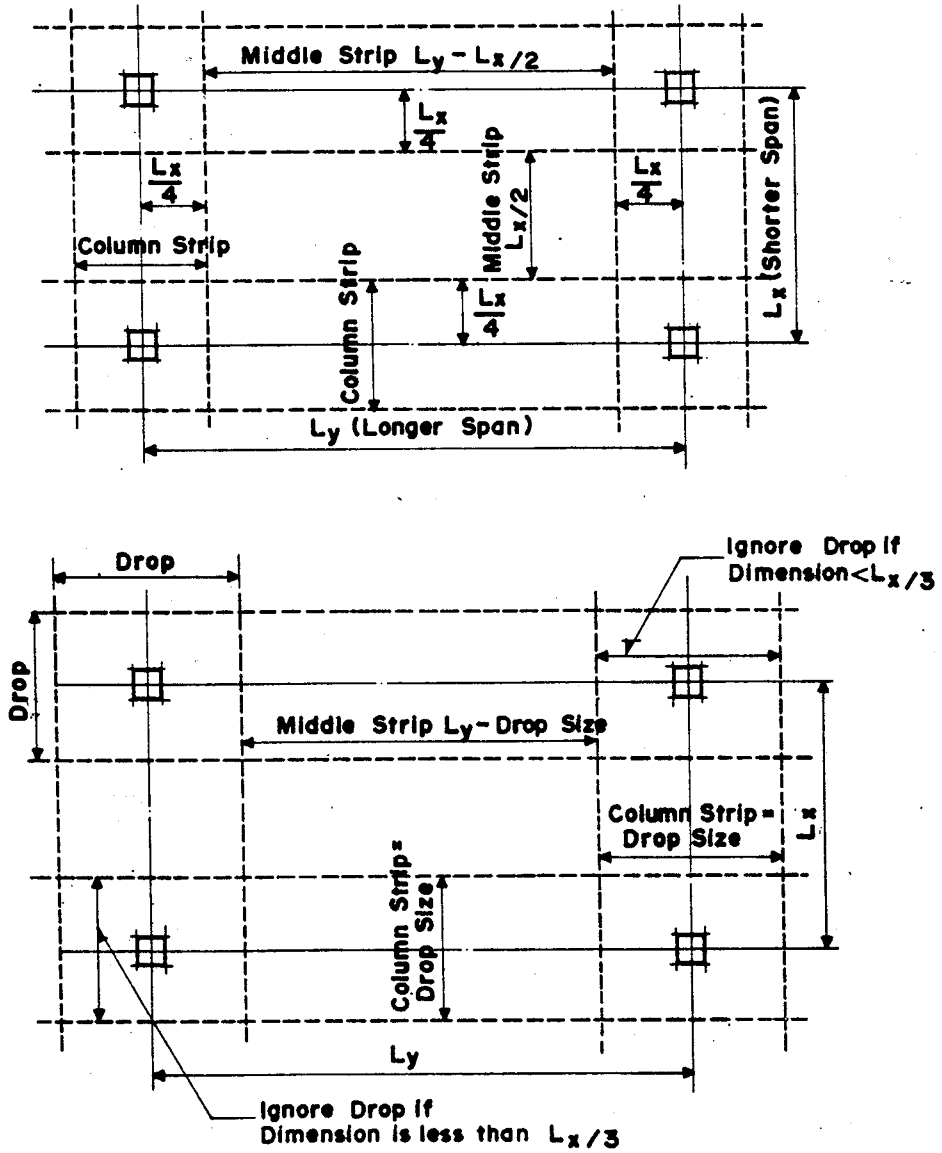


Figure A-6 Division of Panels in Flat Slabs

the lesser of the actual dimension L_{ho}, or $L_{h,max}$, where $L_{h,max}$ is given by:

$$L_{h,max} = L_c + 2d_h \tag{A.8}$$

For a flared head, the actual dimension L_{ho} is that measured to the center of the reinforcing steel (see Fig. A-7).

(5) For the purposes this section a drop may only be considered to influence the distribution of moments within the slab where the smaller dimension of the drop is at least one third of the smaller dimension of the surrounding panels. Smaller drops may, however, still be taken into account when assessing the resistance to punching shear.

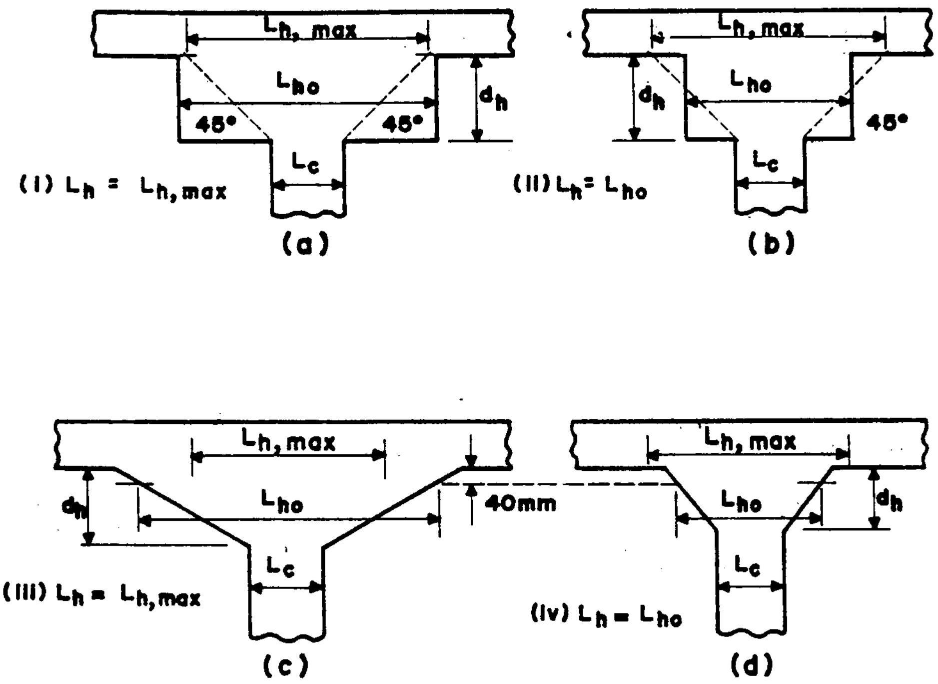


Figure A-7 Types of Column Head

A.4.3.2 Equivalent Frame Method

(1) The structure may be divided longitudinally and transversely into frames consisting of columns and strips of slab.

(2) The width of slab used to define the effective stiffness of the slab will depend upon the aspect ratio of the panels and the type of loading, but the following provisions may be applied in the absence of more accurate methods:

(a) In the case of vertical loading, the full width of the panel, and
(b) for lateral loading, half the width of the panel, may be used to calculate the stiffness of the slab.

(3) The moment of inertia of any section of slab or column used in calculating the relative stiffness of members may be assumed to be that of the cross section of the concrete alone.

(4) Moments and forces within a system of flat slab panels may be obtained from analysis of the structure under the single load case of maximum design load on all spans or panels simultaneously, provided:

(a) The ratio of the characteristic imposed load to the characteristic dead load does not exceed 1.25.
(b) The characteristic imposed load does not exceed 5.0 kN/m^2 excluding partitions.

(5) Where it is not appropriate to analyze for the single load case of maximum design load on all spans, it will be sufficient to consider following the arrangements of vertical loads:

(a) All spans loaded with the maximum design ultimate load, and

(b) Alternate spans with the maximum design ultimate load and all other spans loaded with the minimum design ultimate load ($1.0G_k$).

(6) Each frame may be analyzed in its entirety by any elastic method. Alternatively, for vertical loads only, each strip of floor and roof may be analyzed as a separate frame with the columns above and below fixed in position and direction at their extremities. In either case, the analysis shall be carried out for the appropriate design ultimate loads on each span calculated for a strip of slab of width equal to the distance between center lines of the panels on each side of the columns.

A.4.3.3 Simplified Method

(1) Moments and shear forces in non-sway flat slab structures may be determined using Table A-4, subject to the conditions in (2) below.

(2) The following limitations shall be observed when using the simplified method:

- (a) Design is based on the single load case of all spans loaded with the maximum design ultimate load.
- (b) There are at least three rows of panels of approximately equal span in the direction being considered.
- (c) Successive span length in each direction shall not differ by more than one-third of the longer span.
- (d) Maximum offsets of columns from either axis between center lines of successive columns shall not exceed 10% of the span (in the direction of the offset).

Table A-4 Bending Moment and Shear Force Coefficients for Flat Slabs of Three or More Equal Spans

	Outer support		Near center of first span	First interior support	Center of interior span	Interior support
	Column	Wall				
Moment	-0.040FL	-0.020FL	0.083FL	-0.063FL	0.071FL	-0.055FL
Shear	0.45F	0.40F	-	0.60F	-	0.50F
Total column moments	0.040FL	-	-	0.022FL	-	0.022FL

NOTE 1. F is the total design ultimate load on the strip of slab between adjacent columns considered.
2. L is the effective span $= L_1 - 2h_c/3$.
3. The limitations of Section A.4.3.1(2) need not be checked.
4. The moments shall not be redistributed.

A.4.3.4 Division of Moments Between Column and Middle Strips

(1) The design moments obtained from analysis of the continuous frames using the Equivalent Frame Method (see Section A.4.3.2) or from Table A-4 shall be divided between the column and middle strips in the proportions given in Table A-5.

Table A-5 Distribution of Design Moments in Panels of Flat Slabs

	Apportionment between column and middle strip expressed as percentages of the total negative or positive design moment	
	Column strip (%)	Middle strip (%)
Negative	75	25
Positive	55	45

NOTE: For the case where the width of the column strip is taken as equal to that of the drop, and the middle strip is thereby increased in width, the design moments to be resisted by the middle strip shall be increased in proportion to its increased width. The design moments to be resisted by the column strip may be decreased by an amount such that the total positive and the total negative design moments resisted by the column strip and middle strip together are unchanged.

A.4.4 Design Considerations

A.4.4.1 General

(1) Details of reinforcement in flat slabs shall be as follows:

(a) The reinforcement in flat slabs shall have minimum bend point locations and extensions for reinforcement as prescribed in Fig. A-8.

(b) Where adjacent spans are unequal, extension of negative reinforcement beyond the face of support as prescribed in Fig. A-8 shall be based on requirements of longer span.

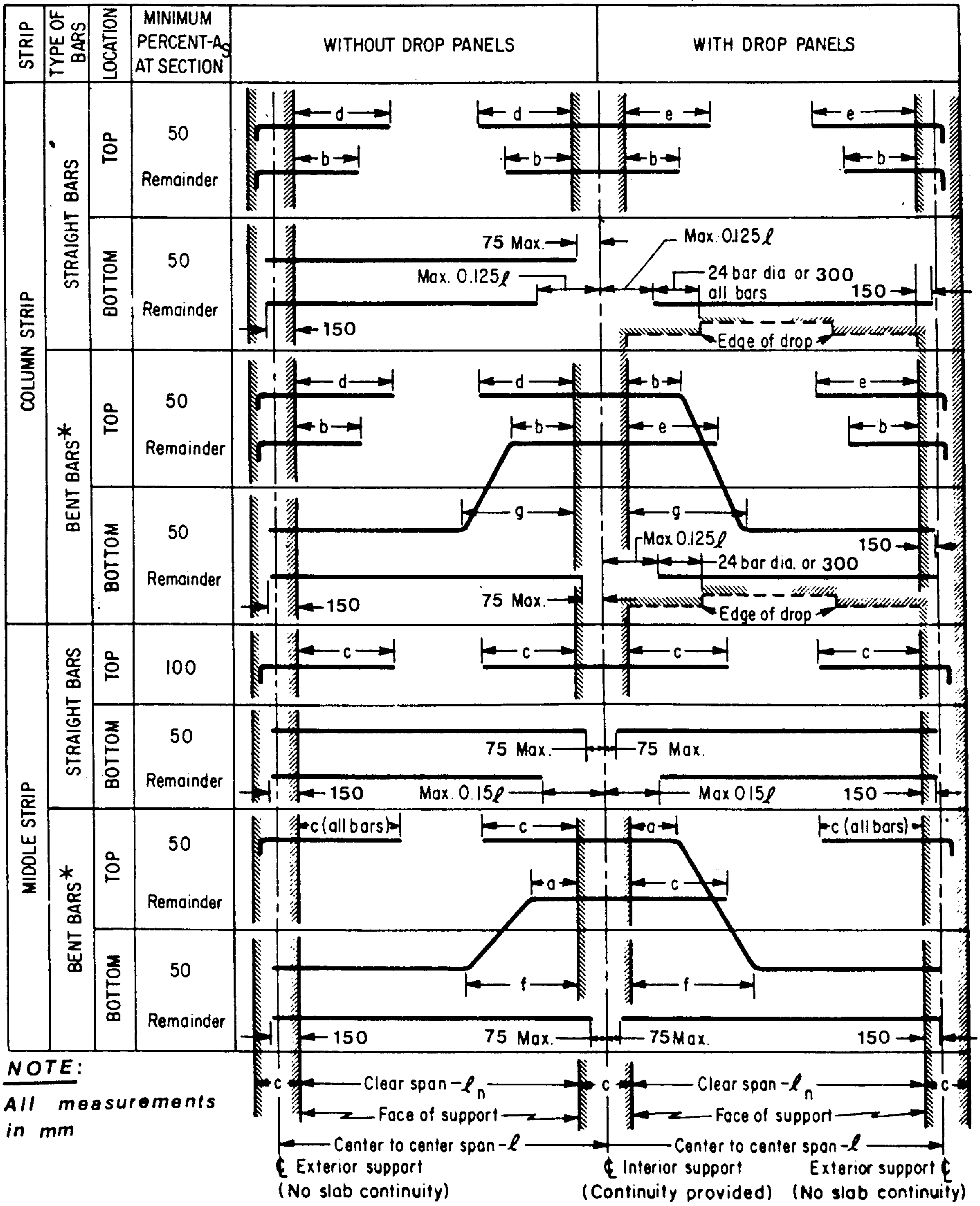
(c) Bent bars may be used only when depth-to-span ratio permits use of bends 45° or less.

(2) For flat slabs in frames not braced against sidesway and for flat slabs resisting lateral loads, lengths of reinforcement shall be determined by analysis but shall not be less than those prescribed in Fig. A-8.

A.4.4.2 Internal Panels

(1) The column and middle strips shall be designed to withstand the design moments obtained from Section A.4.3.

(2) Two-thirds of the amount of reinforcement required to resist the negative design moment in the column strip shall be placed in a width equal to half that of the column strip and central with the column. This concentration of reinforcement over the column will increase the capacity of the slab for transfer of moment to the column by flexure (see Section A.4.4.4)



* Bent bars at exterior supports may be used if a general analysis is made

	BAR LENGTH FROM FACE OF SUPPORT						
	MINIMUM LENGTH					MAXIMUM LENGTH	
MARK	a	b	c	d	e	f	g
LENGTH	$0.14\ell_n$	$0.20\ell_n$	$0.22\ell_n$	$0.30\ell_n$	$0.33\ell_n$	$0.20\ell_n$	$0.24\ell_n$

Figure A-8 Minimum Bend Point Locations and Extensions for Reinforcement in Flat Slabs

A.4.4.3 Edge Panels

(1) The design moments shall be apportioned and designed exactly as for an internal panel, using the same column and middle strips as for an internal panel.

A.4.4.4 Moment Transfer between Slab and Column

(1) When gravity load, wind, earthquake, or other lateral forces cause transfer of moment between slab and column, a fraction of the unbalanced moment shall be transferred by flexure. Fraction of unbalanced moment not transferred by flexure shall be transferred by eccentricity of shear in accordance with Section 4.7.4.

(2) A fraction of the unbalanced moment given by

$$\eta = \frac{1}{1 + \sqrt{b_1/b_2}} \tag{A.9}$$

shall be considered transferred by flexure over an effective slab width between lines that are one and one half slab or drop panel thickness (1.5*h)* outside opposite faces of the column or capital.

(3) Concentration of reinforcement over the column by closer spacing as specified in Section A.4.2(2), or additional reinforcement must be used to resist the unbalanced moment on the effective slab width defined in (2) above.

(4) The design for transfer of load from slab to supporting columns or walls through shear and torsion shall be in accordance with Chapter 4.

(5) As an alternative to (2) above, the slab may be designed for the minimum bending moments per unit width, m_{Sdx} and m_{Sdy} in the x and y direction, respectively, given by Eq. A.10 (see Fig. A-9).

$$m_{Sdx} \text{ (or } m_{Sdy}) \geq nV_{Sd} \tag{A.10}$$

where V_{Sd} is the shear force developed along the critical section
n is the moment coefficient given in Table A-6.

(6) In checking the corresponding resisting moments, only those reinforcing bars shall be taken into account, which are appropriately anchored beyond the critical area (Fig. A-10)

(7) Where analysis of the structure indicates a design column moment larger than the moment $M_{t,max}$ which can be transferred by flexure and shear combined (in accordance with (2) and (4) above), the design edge moment in the slab shall be reduced to a value not greater than $M_{t,max}$ and the positive design moments in the span adjusted accordingly. The normal limitations on redistributions and neutral axis depth may be disregarded in this case.

(8) Moments in excess of $M_{t,max}$ may only be transferred to a column if an edge beam or strip of slab along the free edge is reinforced to carry the extra moment into the column by torsion.

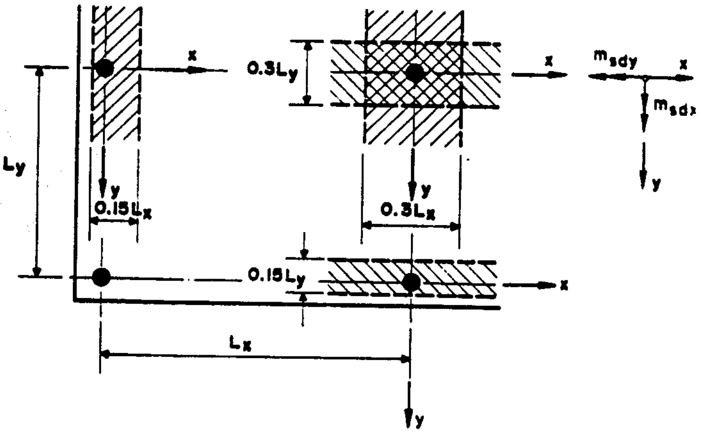


Figure A-9 **Bending Moments m_{Sdx} and m_{Sdy} in Slab-Column Joints subjected to Eccentric Loading, and Effective Width for resisting these Moments**

(9) In the absence of an edge beam, an appropriate breadth of slab may be assessed by using the principles illustrated in Fig. A-11, for transfer of moments between the slab and an edge or corner column.

Table A-6 Moment Coefficient n for Equation (A.10)

Position of column	n for m_{sdx}		Effective width	n for m_{sdy}		Effective width
	top	bottom		top	bottom	
Internal column	-0.125	0	$0.3L_y$	-0.125	0	$0.3L_x$
Edge columns edge of slab parallel, to x-axis	-0.25	0	$0.15L_y$	-0.125	0.125	(per m)
Edge columns edge of slab parallel to y-axis	-0.125	0.125	(per m)	-0.25	0	$0.15.L_x$
Corner column	-0.5	0.5	(per m)	-0.5	0.5	(per m)

A.4.4.5 Panel with Marginal Beams or Walls

(1) Where the slab is supported by a marginal beam with a depth greater than 1.5 times the thickness of the slab, or by a wall then:

(a) the total design load to be carried by the beam or wall shall comprise those loads directly on the wall or beam plus a uniformly distributed load equal to one-quarter of the total design load on the panel; and

(b) the design moments of the half-column strip adjacent to the beam or wall shall be one-quarter of the design moments obtained from Section A.4.3.

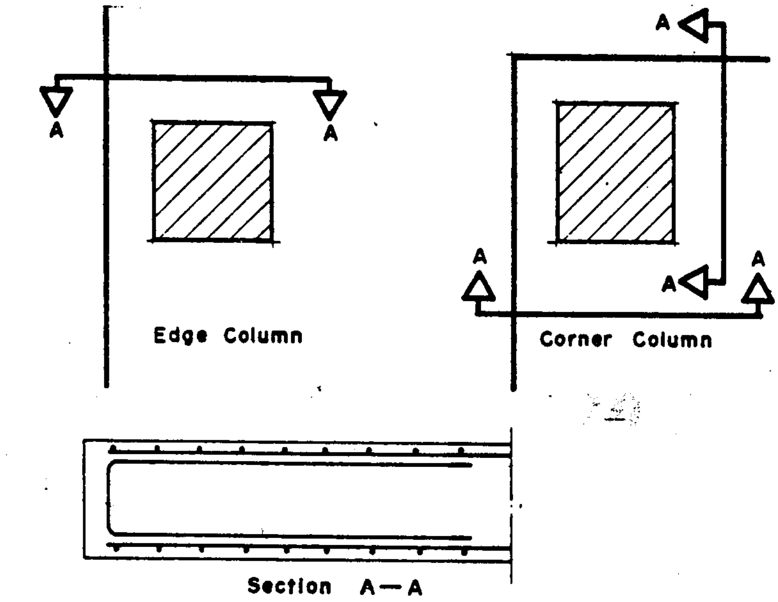


Figure A-10 Detailing Reinforcement over Edge and Corner Columns

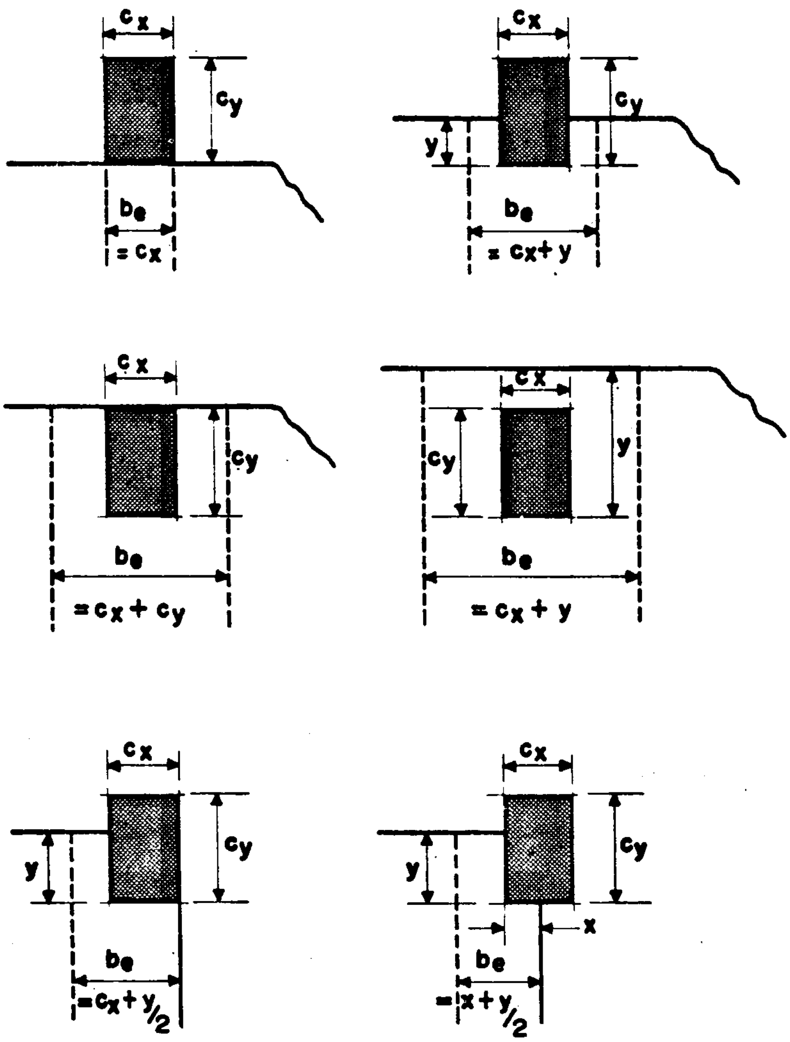


Figure A-11 Definition of Breadth of Effective Moment Transfer Strip b_e for Typical Cases

A.4.4.6 Negative Moments at Free Edge

(1) Reinforcement for negative design moments (other than in the column strip) is only needed where moments arise from loading on any extension of the slab beyond the column center-lines. However, top reinforcement at least equal to the minimum reinforcement defined in Section 7.2.2.2 shall be provided, extending at least $0.1L$ or an anchorage length, whichever is the greater, into the span.

A.4.5 Opening in Panels

A.4.5.1 General

(1) Except for openings complying with Sections A.4.5.2, A.4.5.3 and A.4.5.4, openings shall be completely framed on all sides with beams to carry the loads to the columns.

(2) No opening shall encroach upon a column head.

A.4.5.2 Holes in Areas Bounded by Column Strips

(1) Holes in areas bounded by column strips may be formed provided:

(a) their greatest dimension in a direction parallel to a center-line of the panel does not exceed $0.4L$; and
(b) the total positive and negative design moments are redistributed between the remaining structure to meet the changed conditions.

A.4.5.3 Holes in Areas Common to Two Column Strips

(1) Holes in areas common to two column strips may be formed provided:

(a) in aggregate their length or width does not exceed one-tenth of the width of the column strip;
(b) the reduced sections are capable of resisting the appropriate moments; and
(c) the perimeter for calculating the design shear stress is reduced if appropriate.

A.4.5.4 Holes in Areas Common to a Column Strip and a Middle Strip

(1) Holes in areas common to a column strip a middle may be formed provided:

(a) in aggregate their length or width does not exceed one quarter of the width of the column strip; and
(b) the reduced sections are capable of resisting the appropriate design moments.

APPENDIX B

PRESTRESSED CONCRETE

B.1 SCOPE

(1) Provisions in this chapter apply to structural members prestressed with high strength steel meeting the requirements for prestressing steels in Section B.2.2.

(2) All provisions of this Code not specifically excluded, and not in conflict with the provisions of this chapter, are to be considered applicable to prestressed concrete.

(3) The following provisions shall not apply to prestressed concrete unless specifically noted: Sections 3.7.8, 3.7.9, 6.2, 7.2.1, 7.2.2, 7.2.4, 7.2.5, and Appendix A.

B.2 DATA ON PRESTRESSED STEEL AND PRESTRESSING DEVICES

B.2.1 Prestressing Steel

B.2.1.1 *General*

(1) This section applies to wires, bars and strands used as prestressing tendons in concrete structures.

(2) The requirements apply to the product in the condition in which it is delivered.

(3) The methods of production, the specified characteristics, the methods of testing and the methods of attestation of conformity shall be in accordance with relevant Standards for prestressing materials.

(4) Each product shall be clearly identifiable with respect to the classification system in Section B.2.1.2.

(5) Tensile strength (f_p), 0.1% proof stress ($f_{p0.1}$) and elongation at maximum load (ε_u) shall be appropriately specified in relevant Standards, and established by standard tests.

(6) For steels complying with this Code, tensile strength, 0.1% proof stress, and elongation at maximum load are specified in terms of characteristic values; these values are designated respectively f_{pk}, $f_{p0.1k}$ and ε_{uk}.

B.2.1.2 *Classification and Geometry*

(1) The products (wires, strands and bars) shall be classified according to:

(a) Grade, denoting the value of the 0.1% proof stress ($f_{p0.1k}$) and the value of the tensile strength (f_{pk}) in MPa.
(b) Class, indicating the relaxation behavior
(c) Size
(d) Surface characteristics.

(2) Each consignment shall be accompanied by a certificate containing all the information necessary for its identification with regard to (a) - (d) in (1) above and additional information where necessary.

(3) The actual cross sectional area of the products shall not differ from their normal cross sectional area by more than the limits specified in the relevant Standards.

(4) There shall be no welds in wires and bars. Individual wires of strands may contain staggered welds made only before cold drawing.

(5) For coiled products, after uncoiling a length of wire or strand lying free on a flat surface, the maximum bow height from a base line of specified length shall be less than the values specified in the relevant Standards.

(6) In this Code, three classes of relaxation are defined (see Section B.2.1.5.2)

(a) Class 1 : for wires and strands, high relaxation
(b) Class 2 : for wires and strands, low relaxation
(c) Class 3 : for bars.

(7) Where required, surface characteristics of prestressing steel shall comply with relevant Standards.

B.2.1.3 ***Physical Properties***

(1) The following mean values may be assumed :

(a) Density	: 7850 kg/m³
(b) Coefficient of thermal expansion	: 10×10^{-6}/°C.

B.2.1.4 ***Mechanical Properties***

B.2.1.4.1 *Strength*

(1) The 0.1% proof stress ($f_{p,01k}$) and the specified value of the tensile strength (f_{pk}) are defined as the characteristic value of the 0.1% proof load and the characteristic maximum load in axial tension respectively, divided by the nominal cross sectional area.

(2) The ratio of the actual maximum load to the specified maximum load shall not exceed the values specified relevant Standards.

B.2.1.4.2 *Stress-Strain Diagram*

(1) Stress-strain diagrams for the products, based on production data, shall be prepared and made available by the producer as an annex to the certificate accompanying the consignment.

B.2.1.4.3 *Ductility Characteristics*

(1) The products shall have adequate ductility in elongation, as specified in relevant Standards.

(2) The products shall be assumed to have adequate ductility in bending if the characteristic elongation of the prestressing steel at maximum load ϵ_{uk} is at least 3.5‰.

(3) Adequate ductility in bending may be assumed if the products satisfy the requirements for bendability of the relevant Standards.

B.2.1.4.4 *Modulus of Elasticity*

(1) A mean value of 200 GPa may be assumed for wires and bars. The actual value can range from 195 to 205 GPa, depending on the manufacturing process.

(2) A value of 190 GPa may be assumed for strand. The actual value can range from 175 to 195 GPa, depending on the manufacturing process. Certificates accompanying the consignment should give the appropriate value.

B.2.1.4.5 *Fatigue*

(1) The products shall have adequate fatigue strength.

(2) For fatigue requirements of prestressing steel refer to relevant Standards.

B.2.1.4.6 *Multi-Axial Stresses*

(1) The behavior of the products under multi-axial stresses shall be adequate.

(2) Adequate behavior under multi-axial stresses may be assumed if the products satisfy the requirements specified in the relevant Standards.

B.2.1.5 ***Technological Properties***

B.2.1.5.1 *Surface Condition*

(1) The products shall be free from defects which could impair their performance as prestressing tendons.

(2) Longitudinal cracks need not be considered as defects if their depth is less than the values specified in relevant Standards.

B.2.1.5.2 *Relaxation*

(1) The products shall be classified for relaxation purposes, according to the maximum percentages of loss of stress.

B.2.1.5.3 *Susceptibility to Stress Corrosion*

(1) The products shall have an acceptably low level of susceptibility to stress corrosion.

(2) The level of susceptibility to stress corrosion may be assumed to be acceptably low if the products comply with the criteria specified in relevant Standards.

B.2.2 Prestressing Devices

B.2.2.1 *Anchorages and Couplers*

B.2.2.1.1 *General*

(1) This section applies to anchoring devices (anchorages) and coupling devices (couplers) for application in post-tensioned construction, where :

(a) Anchorages are used to transmit the forces in tendons to the concrete in the anchorage zone;
(b) Couplers are used to connect individual lengths of tendon to make continuous tendons.

(2) The performance requirements, the methods of testing and the methods of attestation of conformity shall be defined in relevant, Standards.

(3) In establishing performance requirements, consideration shall be given to :

(a) The relative efficiency of the tendon anchorage/coupler assembly in comparing the actual value of the failure load of the assembly with that of the tendon.
(b) The elongation of the anchored/coupled tendon at failure.
(c) The fatigue strength of the anchored/coupled tendon .
(d) The load which can be transferred by the anchorage to the concrete, taking account of the location of the anchorage in the cross-section, the spacing between anchorages, the concrete strength and the reinforcement in the anchorage zone.

(4) Requirements for the use of anchorages and couplers, shall be defined in technical approval documents. Detailing of anchorage zones shall comply with Sections B.5 and B.6.

(5) When defining test methods, consideration shall be given to two modes of testing ;

(a) **Mode a** : when components of known geometry and material specification have been taken at random out of production or form stock.
(b) **Mode b** : when components have been selected by the producer of the components or when prototype anchorages or couplers are to be tested.

B.2.2.1.2 *Mechanical Properties*

(1) Tendon-anchorage assemblies and tendon-coupler assemblies shall have strength, elongation and fatigue characteristics sufficient to meet the basic requirements of Chapter 3.

(2) This may be assumed if :

(a) The geometry and material characteristics of the anchorage and coupler components are such that their premature failure is precluded.
(b) The elongation at failure of the assemblies is not excessive.
(c) Tendon-anchorage assemblies are not located in otherwise highly-stressed zones.

For the fatigue requirements of anchorages and couplers, refer to relevant Standards.

(3) The strength of the anchorage devices and zones shall be adequate for the transfer of the tendon force to the concrete and the formation of cracks in the anchorage zone does not impair the function of the anchorage.

(4) This may be assumed if :

(a) The strength of the anchorage devices exceeds the characteristic breaking load of the tendon, either under static loading conditions or a limited number of load cycles.
(b) The detailing provisions of this Code are met.

B.2.2.2 ***Ducts and Sheaths***

B.2.2.2.1 *General*

(1) This section applied to post-tensioned concrete construction where the tendons are tensioned in internal ducts.

(2) For bonded tendons, where the ducts are grouted after tensioning, the shape (profile) of the duct shall permit the proper transfer of forces from the tendons to the concrete.

(3) The performance requirements, the methods of testing and the methods of attestation of conformity shall be defined in relevant Standards.

(4) Requirements covering the use of ducts and sheaths shall be defined in technical approval documents.

(5) Sheaths should consist of adequate materials as specified in relevant Standards.

B.3 BASIS OF DESIGN

(1) All provisions in Chapter 3 shall apply to prestressed concrete.

B.3.1 Partial Safety Factors for Materials

(1) Partial safety factors for material properties are given in Section 3.5.3.

B.3.2 Partial Safety Factors for Action on Building Structures

(1) Partial safety factors for different effects of action are given in Table B.1 in addition to the requirements specified in Table 3.3.

Table B.1: Partial Factors for Action in Building Structures

Types of Effect	Prestressing, γ_p
Favorable effect	0.9 or 1.0
Unfavorable effect	1.2 or 1.0

(2) For the evaluation of local effects (anchorage zones, bursting pressure) an effort equivalent to the ultimate characteristic strength shall be applied to the tendons (see Section B.4.3).

(3) For the verification of the design of prestressed elements, the γ_p values in Table B.1 should generally be used. However, for the evaluation of the combined effects of prestressing and of self-weight, reduced values of partial safety factors, which do not include allowances for analytical uncertainty, may be used (e.g. γ_p = 1.0 and γ_G = 1.2).

B.4 ANALYSIS

(1) All provisions in Chapter 4 shall apply to prestressed concrete members.

B.4.1 Prestressed Slabs

(1) The rules given in (2)-(4) below complement those given in Section B.4.3.

(2) Regardless of the type of tendons used (e.g. bonded or unbonded), the contact forces due to the curvature and friction of the tendons and the forces acting on the anchorage devices may be treated as external loads in the serviceability limit states.

(3) For the ductility classification of prestressed tendons see Section B.2.1.4.3(3).

(4) Plastic analysis should not be applied to members where pretensioned tendons are used, unless justified.

B.4.2. Anchorage Zones for Post-Tensioning Forces

(1) Such zones, which are subjected to concentrated forces, shall be analyzed and designed to take account of:

(a) the overall equilibrium of the zone;
(b) the transverse tensile effects due to the anchorages, individually and as a whole;
(c) compression struts, which develop in the anchorage zone of post-tensioned members, and local bearing stresses under the anchorages.

(2) Such zones in post-tensioned members may be designed using the methods given in Section B.4.2.3 or by using adequate strut and tie models based on Section 3.8.1.3.

(3) Three-dimensional models should be considered, where the dimensions of the bearing area are small compared with the cross-section of the anchorage zone.

(4) The detailing requirements of Chapter 7 generally, and Section B.6.5 and B.6.7.1 in particular, shall be met.

B.4.3 Determination of the Effects of Prestressing

B.4.3.1 *General*

(1) This section relates to structures where prestress is provided by fully bonded internal tendons.

(2) The effects to be considered are:

(a) Local effects around anchorages and where tendons change direction.
(b) Direct effects in determinate structures.
(c) Direct and secondary indirect effects due to redundant restraints in indeterminate structures.

(3) For members containing permanently unbonded tendons refer to relevant Standards.

(4) Members containing tendons which are temporarily unbonded during construction may be treated using simplified assumptions. In general, they may be treated as members with bonded tendons,

except that at the ultimate limit state. The stress in tendons is assumed not to have increased due to loading.

B.4.3.2 *Determination of Prestressing Force*

(1) The mean value of the prestressing force is given by (a) or (b) below, whichever is appropriate:

(a) For pre-tensioned members :

$$P_{m,t} = P_o - \Delta P_c - \Delta P_t(t) - \Delta P_\mu(x) \tag{B.1}$$

$\Delta P_\mu(x)$ may require consideration where deflected tendons are use.

(b) For post-tensioned members :

$$P_{m,t} = P_o - \Delta P_c - \Delta P_\mu(x) - \Delta P_{s1} - \Delta P_t(t) \tag{B.2}$$

where $P_{m,t}$ is the mean value of the prestressing at time t and at a particular point along the member

P_o is the initial force at the active end of the tendon immediately after stressing.

$\Delta P_\mu(x)$ is the loss due to friction

ΔP_{s1} is the loss due to anchorage slip

ΔP_c is the loss due to elastic deformation of the member at transfer

$\Delta P_t(t)$ is the loss due to creep, shrinkage and relaxation at time t.

(2) For limits on the initial prestress and methods of calculating losses, see Section B.5. For transmission lengths and the dispersion of prestress, see Section B.5.5.

(3) For serviceability calculations, allowance shall be made for possible variations in prestress. Two characteristic values of the prestressing force at the serviceability limit state are estimated from:

$$\begin{aligned} P_{k,sup} &= r_{sup} P_{m,t} \\ P_{k,inf} &= r_{inf} P_{m,t} \end{aligned} \tag{B.3}$$

where $P_{k,sup}$ and $P_{k,inf}$ are respectively the upper and lower characteristic values.

$P_{m,t}$ is the mean prestressing force estimated using the mean values for the deformation properties and the losses calculated in accordance with Section B.5.

r_{sup} and r_{inf} may be taken as 1.1 and 0.9 respectively in absence of a more rigorous determination and provided that the sum of the losses due to friction and time dependent effects is $\leq$ 30% of the initial prestress.

(4) The values of $P_{m,t}$ which will generally be used in design are :

$P_{m,o}$ is the initial prestress at time $t = 0$

P_m is the prestress after occurrence of all losses

(5) At the ultimate limit state the design value of prestress is given by:

$$P_d = \gamma_p P_{m,t} \tag{B.4}$$

(6) Values for γ_p are given in Table B.1.

(7) For considering local effects at the ultimate limit state, the prestressing force shall be taken as equal to the characteristic strength of the tendons.

(8) This applies when checking the influence of concentrated forces or bursting effects at anchorages or where tendons change direction Section B.5

B.4.3.3 ***Effects of Prestressing under Service Conditions***

(1) The statically determinate and indeterminate internal forces and moments caused by prestressing shall be calculated by elastic theory.

(2) For normal buildings where the calculation of crack width is not considered necessary, the mean values of prestress may be used.

(3) In other cases, where the structural response is highly sensitive to the influence of prestress, the effects of prestress may be determined according to (a) or (b) below, as appropriate.

- (a) For checking cracking or decompression see Section 5.3), the opening of joints between precast elements and fatigue effects, the relevant estimated characteristic values of the prestress are used.
- (b) For checking compressive stresses the mean values of prestress are used.

B.4.3.4 ***Effects of Prestressing at the Ultimate Limit States***

B.4.3.4.1 *Structural Analysis - Linear Methods*

(1) Statically determinate and indeterminate effects of prestress shall be calculated using the appropriate ultimate design value of the prestressing force.

(2) In linear structural analysis γ_p may be taken as 1.0.

(3) Where linear analysis with redistribution is used the moments to which the redistribution is applied shall be calculated including any statically indeterminate effects of prestress.

B.4.3.4.2 *Design of Sections*

(1) When assessing the behavior of a section at the ultimate limit state, the prestressing force acting on the section is taken as the design value, P_d . The prestrain corresponding to this force shall be taken into account in the assessment of section strength.

(2) The prestrain may be taken into account by shifting the origin of the design stress-strain diagram for the prestressing tendons by an amount corresponding to the design prestress.

(3) γ_p may be taken as 1.0 provided the following conditions are both met:

- (a) Not more than 25% of the total area of prestressed steel is located within the compression zone at the ultimate limit state, and
- (b) The stress at ultimate in the prestressing steel closest to the tension face exceeds $f_{p0.1k}/\gamma_m$.

If the conditions are not met, the lower value of γ_p given in Table B.1 should be applied to all tendons.

(4) For the effects of inclined tendons, see Section B.5.5.8.

(5) Any indirect prestressing moments due to redundant restraints should be taken at their characteristic values.

B.4.3.5 ***Determination of the Effects of Time Dependent Deformation of Concrete***

B.4.3.5.1 *General*

(1) The accuracy of the procedures for the calculation of the effects of creep and shrinkage of concrete shall be consistent with the reliability of the data available for the description of these phenomena and the importance of their effects on the limit state considered.

(2) In general, the effects of creep and shrinkage shall be taken into account only for the serviceability limit states. An important exception concerns second order effects.

(3) Special investigations shall be considered when the concrete is subjected to extremes of temperature.

(4) The effects of steam curing may be taken into account by means of simplified assumptions.

(5) The following assumptions may be adopted to give an acceptable estimate of the behavior of a concrete section if the stresses are kept within the limits corresponding to the normal service conditions :

(a) creep and shrinkage are independent
(b) a linear relationship is assumed between creep and the stress causing the creep
(c) non-uniform temperature and moisture effects are neglected
(d) the principle of superposition is assumed to apply for actions occurring at different ages
(e) the above assumptions also apply to concrete in tension

(6) For the evaluation of the time dependent losses of prestress, the effects of creep, shrinkage and relaxation of the tendons shall be taken into account (see Section B.5.5).

(7) The creep function is given by the relationship :

$$J(t, t_o) = 1/E_c(t_o) + \phi\,(t,t_o)/E_{c28} \qquad \text{(B.5)}$$

where t_o is the time at initial loading of the concrete
t is the time considered
$J(t,t_o)$ is the creep function at time t
$E_c(t_o)$ is the tangent modulus of elasticity at time t_o
E_{c28} is the tangent modulus of elasticity at 28 days
$\phi_{(t,to)}$ is the creep coefficient related to the elastic deformation at 28 days

Values are given in Chapter 2 for final creep coefficients ϕ for typical situations. It should be noted, however, that the definitions of $E_{c(to)}$ and E_{c28} above, differ from that in Section 2.5 where the secant modulus E_{cm} is defined. Hence, where the creep coefficients of Table 2.6 are used in connection with $E_c(t_o)$ and E_{cm}, respectively, and where creep deformations are significant, the values of Table 2.6 should be multiplied by 1.05.

(8) On the basis of the assumptions listed in (5) above, the total strain for concrete subjected to initial loading at time t_o with a stress σ_o and subjected to subsequent stress variations $\Delta\sigma(t_i)$ at time t_i may be expressed as follows :

$$\epsilon_{tot}(t,t_o) = \epsilon_n(t) + \sigma_o J(t,t_o) + \sum J(t,t_i)\Delta\sigma(t_i) \tag{B.6}$$

In this expression $\epsilon_n(t)$ denotes an imposed deformation independent of the stresses (e.g. shrinkage, temperature effects).

(9) For the purpose of structural analysis, Eq. B.6 may be written as follows:

$$\epsilon_{tot}(t,t_o) = \epsilon_n(t) + \sigma(t_o)\, J(t,t_o) + [\sigma(t) - \sigma(t_o)]\left[\frac{1}{E_c(t_o)} + \chi\frac{\phi(t,t_o)}{F_{c28}}\right] \tag{B.7}$$

where the ageing coefficient x depends on the development of strain with time.

(10) In normal cases, x may be taken as 0.8. This simplification is good in the case of pure relaxation of the effects of a constant imposed deformation but is also adequate in cases where only long term effects are considered.

(11) If the stresses in the concrete only vary slightly, the deformations may be calculated using an effective modulus of elasticity :

$$E_{c,eff} = E_c(t_o)/(1 + \phi(t,t_o)) \tag{B.8}$$

For the notation see (7) above.

B.5. SECTION AND MEMBER DESIGN

B.5.1 Prestressing Steel: General

(1) Data on material properties given in this section are either representative values, corresponding to the relevant steel grade specified in appropriate Standards, or are idealizations suitable for design purposes

(2) In general, the properties specified are those given in Section B.2.1.1(5) or other appropriate Standards.

(3) Unless stated otherwise, design shall be based on a specified grade, represented by its characteristic 0.1% proof stress ($f_{p0.1k}$).

(4) All types of prestressing steel specified in Section B.2.1, which satisfy the mechanical, physical and technological requirements or other relevant Standards may generally be used in design, in accordance with the data given below, unless greater accuracy is required.

B.5.2 Physical Properties of Prestressing Steel

(1) The values given in Section B.2.1.3 may be used as design data. They may be assumed to be valid in the range from - 20°C to 200°C.

B.5.3 Mechanical Properties of Prestressing Steel

B.5.3.1 *Strength*

(1) For all types of prestressing steel the values for $f_{p0.1k}$, ϵ_{uk} and f_{pk} shall be defined.

(2) Relevant properties for defined types and grades of steel may be taken from relevant Standards. For other types of steel, the properties are to be confirmed by technical approval documents.

(3) Design calculations may be based on the nominal size or the nominal cross-sectional area of the prestressing steel.

B.5.3.2 *Modulus of Elasticity*

(1) The values given in Section B.2.1.4.4 apply.

B.5.3.3 Stress-Strain Diagram

(1) The general ductility requirements shall be in accordance with Section B.2.1.4.3 and as specified in relevant Standards.

(2) An idealized bi-linear diagram is given in Fig. B-1. This diagram is valid for temperatures from -20°C to 200°C.

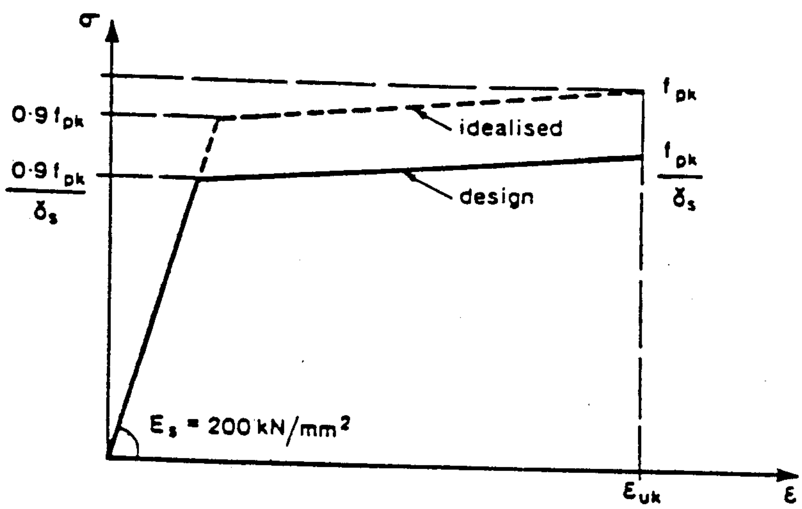


Figure B-1 Design Stress-Strain Diagram for Prestressing Steel

(3) Figure B-1 may generally be used for overall analysis, local verifications and the checking of section capacity.

(4) Figure B-1 may be modified, e.g with a flatter or horizontal top branch, for local verifications or section design.

(5) Design values for the steel stress are derived from the idealized characteristic diagram by dividing by γ_s, the partial factor for prestressing steel (see Section B.3.2)

B.5.3 Mechanical Properties of Prestressing Steel

B.5.3.1 *Strength*

(1) For all types of prestressing steel the values for $f_{p0.1k}$, ϵ_{uk} and f_{pk} shall be defined.

(2) Relevant properties for defined types and grades of steel may be taken from relevant Standards. For other types of steel, the properties are to be confirmed by technical approval documents.

(3) Design calculations may be based on the nominal size or the nominal cross-sectional area of the prestressing steel.

B.5.3.2 *Modulus of Elasticity*

(1) The values given in Section B.2.1.4.4 apply.

B.5.3.3 Stress-Strain Diagram

(1) The general ductility requirements shall be in accordance with Section B.2.1.4.3 and as specified in relevant Standards.

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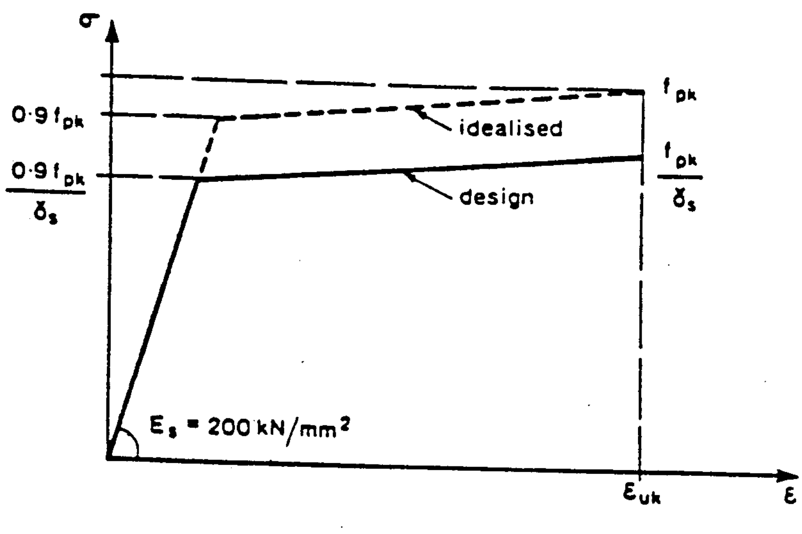

Figure B-1 Design Stress-Strain Diagram for Prestressing Steel

(3) Figure B-1 may generally be used for overall analysis, local verifications and the checking of section capacity.

(4) Figure B-1 may be modified, e.g with a flatter or horizontal top branch, for local verifications or section design.

(5) Design values for the steel stress are derived from the idealized characteristic diagram by dividing by γ_s, the partial factor for prestressing steel (see Section B.3.2)

(6) For section design, either of the following assumptions may be made:

(a) a horizontal top branch to the design curve in Fig. B-1, the stress in the prestressing steel is limited to 0.9 f_{pk}/γ_s with no limit to the steel strain, although in some cases it may be convenient to assume a limit.
(b) an inclined top branch, with the increasing steel strain limited to 0.01.

B.5.3.4 ***Ductility***

(1)The provisions of Section B.2.1.4.3 shall apply.

(2) For structural analysis, if not stated otherwise, post-tensioned tendons may be assumed as having high ductility: pre-tensioned tendons are assumed as having normal ductility.

B.5.3.5 ***Fatigue***

(1) For fatigue requirements for prestressing steel, refer to relevant standards.

B.5.3.6 ***Multi-Axial Stresses***

(1) If not stated otherwise in technical approval documents, tendons assembled from prestressing steel satisfying the requirements of Section B.2.1.4.6 may be considered to withstand the full specified tensile strength, if the bending radius of the saddle, which is supporting the tendon at its point of deviation, satisfies the requirements of Table B.2.

(2) The values in Table B.2 do not relate to the coefficients of friction in Section B.5.5.5(8).

Table B.2 Criteria for Satisfying Multi-Axial Conditions in Tendons

Type of tendon	Ratio = $\frac{\text{minimum bending radius}}{\text{nominal diameter}}$
Single wire or strand, deflected after tensioning	15
Single wire or strand, tensioned in smooth duct	20
Single wire or strand, tensioned in ribbed duct	40
Multi wire or strand tendon	Preceding values multiplied by n_1/n_2

in which: n_1 = total number of wires or strands in the tendon
n_2 = number of wires or strands transferring the radial force of all wires or strands in the tendon to the deviator (see Fig. B-2).

B.5.3.7 ***Anchorage or Coupler Assemblies of Tendons***

(1) Tendon anchorage assemblies and tendon coupler assemblies satisfying the performance requirements of Section B.2.2.1.2 may be considered to withstand the full characteristic strength of the tendon.

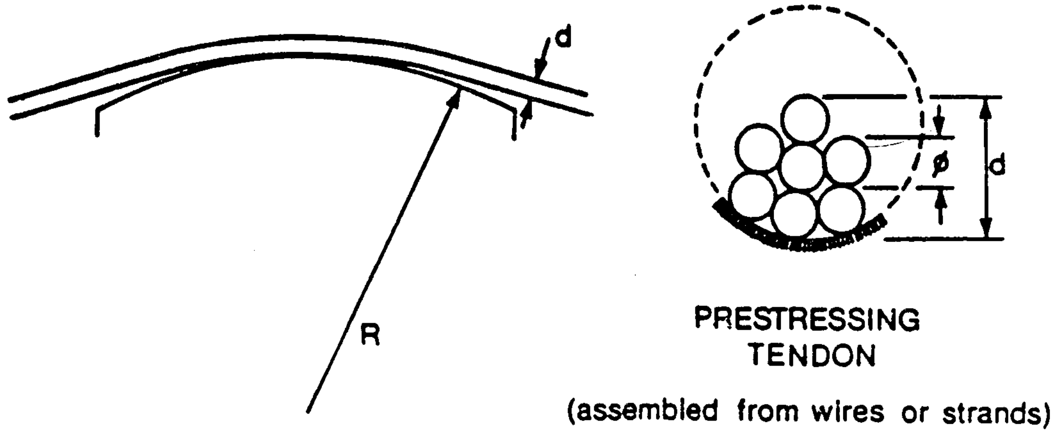

Figure B-3 Example of n_1/n_2 value in Table B-2 (in this case n_1/n_2 = 7/3

B.5.4 Technological Properties of Prestressing Steel

B.5.4.1 *Relaxation*

(1) Certificates accompanying the consignments shall indicate the class and relevant relaxation data of the prestressing steel (see Section B.2.1.5 and relevant standards).

(2) For design calculations, the values which may be taken into account for losses at 1000 h are either those given in the certificate or those assumed in Fig. B-3 for the three classes of steel shown. The long term values of the relaxation losses may be assumed to be three times the relaxation losses after 1000 h.

(3) An indication of how relaxation losses increase between 0-1000 hours is given in Table B.3.

Table B.3 Indication of Relationship Between Relaxation Losses and Time up to 1000 hours

Time in hours	1	5	20	100	200	500	1000
Relaxation losses as percentages of losses after 1000 hours	15	25	35	55	65	85	100

(4) Relaxation at temperatures of the structure over 20°C will be higher than given in Fig. B-4. This may affect building structures in hot climates, power plants, etc. If necessary the producer should be asked to include relevant information in the certificate Section B.2.1.2(2).

(5) Short-term relaxation losses at a temperature of the structure exceeding 60°C can be 2 to 3 times those at 20°C. However, in general, heat curing, over a short period, may be considered to have no effect on long term relaxation results (see Section B.5.5.5).

B.5.4.2 *Susceptibility to Stress Corrosion*

(1) The provisions of Section B.2.1.5.3 apply.

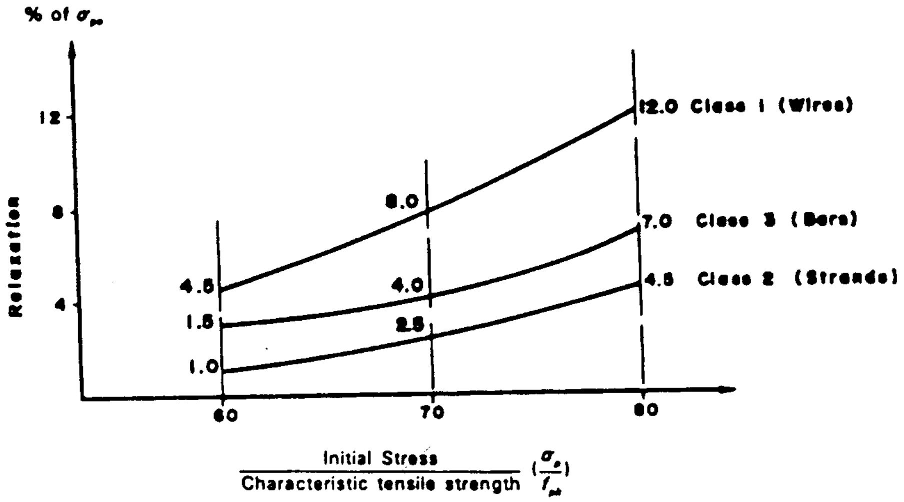

Figure B-3 Relaxation Losses after 1000 h at 20 °C

B.5.4.3 *Temperature Dependent Behaviour*

(1) For temperature dependent behaviour refer to relevant Standards on Fire Resistance.

B.5.5 Design of Members in Prestressed Concrete

B.5.5.1 *General*

(1) This section relates to structures where prestress is provided by fully bonded internal tendons.

(2) The effects of prestressing to be considered include:
- (a) minimum requirements for concrete classes (Section B.5.5.2)
- (b) minimum requirements for prestressing units (Section B.5.5.3)
- (c) determination of the relevant prestressing force (Section B.4.2)
- (d) initial prestressing force section (Section B.5.5.4)
- (e) loss of prestress (Section B.5.5.5)
- (f) transfer of prestressing forces and anchorage zone design for pre-tensioned members section (Section B.5.5.6)
- (g) anchorage zones in post-tensioned members (Section B.5.5.7)

(3) The provisions Section B.4.3 should be applied in all calculations relating to the effects of prestress both in global and local analysis and in section design for the ultimate and serviceability limit states.

B.5.5.2 *Minimum Strength Class for Prestressed Concrete*

(1) The minimum class for post-tensioned members is C30, and for pre-tensioned members is C40.

B.5.5.3 *Minimum Number of Prestressing Units in Isolated Structural Elements*

(1) Isolated prestressed concrete members shall contain in the pre-compressed tensile zone a minimum number of prestressing units in order to ensure that, with an adequate reliability, a failure of a certain number of bars, wires or tendons does not lead to a failure of the member.

(2) Item (1) above applies to structural prestressed members in which no additional load-carrying capacity due to redistribution of internal forces and moments, transverse redistribution of loads or due to other measures (e.g normal steel reinforcement) exists.

(3) The requirement of (1) above may be considered to be met if the minimum number of bars, wires or tendons given in Table B.4 is provided. Table B.4 assumes equal diameters of all bars, wires or tendons.

(4) The requirement may also be assumed to be satisfied if at least one strand with seven or more wires (wire diameter $\geq$ 4.0 mm) is provided in the isolated member.

(5) If the actual number of bars, wires or tendons in the isolated member is less than the values given in Table B.4, adequate reliability against failure should be demonstrated.

Table B.4: Minimum Number of Bars, Wires and Tendons in the Pre-Compressed Tensile Zone of Isolated Members

Type of Units	Minimum number
Individual bars and wires	3
Bars and wires, forming a strand or a tendon	7
Tendons except strands (see Item (4) above)	3

B.5.5.4 *Initial Prestressing Force*

(1) The initial prestressing force shall be determined in accordance with Section B.4.3, which also lists relevant factors affecting loss of prestress.

(2) The maximum force applied to a tendon P_o (i.e, the force at the active end, immediately after stressing, $x = 0$, see section B.4.3.2) shall not exceed $A_p.\sigma_{o,max}$, where:

A_p is the cross-sectional area of the tendon
$\sigma_{o,max}$ is the maximum stress applied to the tendon

$$\sigma_{o,max} \leq 0.80\ f_{pk} \text{ or } \leq 0.90\ f_{p0.1k}, \text{ whichever is the lesser} \qquad \text{(B.9)}$$

(3) The prestressing force applied to the concrete immediately after tensioning (post-tensioning) or after transfer (pre-tensioning), i.e, $P_{mo} = A_p\sigma_{pmo}$, shall not exceed the lesser of the forces determined from:

$$A_p\sigma_{pmo} = 0.75 f_{pk}\ A_p, \text{ or } 0.85 f_{p\,0.1k} A_p \qquad \text{(B.10)}$$

where σ_{pmo} is the stress in the tendon immediately after tensioning or transfer.

(4) For pre-tensioned members, $P_{m,o}$, in (3) above, is calculated from Eq.(B.13) below:

$$P_{m,o} = P_o - \Delta P_c - \Delta_{lr}\ [-\Delta P\mu\ (x)] \qquad \text{(B.11)}$$

where ΔP_c, and $\Delta P\mu(x)$ are defined in Section B.4.3.2.
ΔP_{lr} is the short-term relaxation loss.

(5) For post-tensioned members, $P_{m,o}$ is calculated from Eq.(B.14) below.

$$P_{m,o} = P_o - \Delta P_{sl} - \Delta P_c - \Delta P\mu(x) \qquad \text{(B.12)}$$

(6) Methods for evaluating ΔP_{sl}, ΔP_{ir}, ΔP_c and $\Delta P\mu(x)$ are given in Section B.5.5.5.

(7) The minimum concrete strength required at the time of tensioning or stress transfer shall be indicated in technical approval documents for the prestressing system concerned. Where such documents do not exist, requirements concerning reliability and performance should be considered.

(8) The limiting values of (2) and (3) above are generally valid; they may be modified, however, depending on a number of factors, e.g.

- (a) whether it is possible to replace a damaged tendon,
- (b) the consequences of the fracture of a tendons, in particular danger to human life.
- (c) the stress levels in the concrete due to prestressing
- (d) the grade of steel and type of tendon used,
- (e) whether or not the tendons are subsequently bonded,
- (f) the time when the grout is injected into the ducts,
- (g) the possibility of achieving the required prestressing force in the tendon by overstressing when unexpectedly high friction is met : in this exceptional case, the maximum initial force P_o may be increased to $0.95 f_{p0.1k} A_p$.

B.5.5.5 ***Loss of Prestress***

(1) Loss of prestress shall be calculated in accordance with the principles in Section B.4.3.2.

(2) An estimate is required of the effective prestress at various stages considered in the design, and hence an allowance has to be made for appropriate losses of prestress due to the different factors given in Section B.4.3.2. Whenever possible, these calculations should be based on experience or on experimental data relating to the materials and prestressing methods to be used. For a wide range of structures, and in the absence of such data, the general recommendations given in (5)-(11) may be used, in approximately estimating the total loss of prestress.

(3) It is recommended that the actual values of prestressing losses at tensioning should be checked by measuring the prestressing force transferred from one end of the tendon to the other.

(4) Immediate losses should be calculated in accordance with (5) to (8) below. Time dependent losses should be calculated in accordance with (9)-(10) below.

(5) Loss of prestress due to anchorage slip (ΔP_{sl}) should be determined from experience and technical approval documents relating to the prestressing system to be used.

(6) Calculation of the immediate loss of force in the tendons due to elastic deformation of the concrete (ΔP_c) may be based on the values of the modulus of elasticity of the concrete given in Table 2.5 and on the values for the prestressing steel given in Section B.3.2.4.4.

For pre-tensioning, the losses of prestress should be calculated on a modular ratio basis, using the stress in the adjacent concrete.

For post-tensioning, a progressive loss occurs when tendons are not stressed simultaneously. Where greater accuracy is not required, this should be calculated on the basis of half the product of the modular ratio and the stress in the adjacent concrete averaged along the length of the tendons.

(7) The short-term relaxation loss (ΔP_{tr}), which occurs in pre-tensioning between stressing the tendons and transferring the stress to the concrete, should be estimated using the data in section B.5.4.1.

(8) The loss of prestress in post-tensioned tendons due to friction [$\Delta P\mu(x)$] may be estimated from:

$$\Delta P\mu(x) = P_o\,(1 - e^{-\mu(\theta + k.x)}) \tag{B.13}$$

where μ is the coefficient of friction between the tendons and their ducts
θ is the sum of the angular displacements over a distance x (irrespective of direction or sign)
k is an unintentional angular displacement (per unit length) related to the profile of the tendons.

μ depends on the surface characteristics of the tendons and the duct, on the presence of rust, on the elongation of the tendon and on the tendon profile. In the absence of more exact data, for tendons which fill about 50% of the duct, the following values for μ may be assumed, when using Eq.(B-15).

cold drawn wire	0.17
strand	0.19
deformed bar	0.65
smooth round bar	0.33

Values for k should be given in technical approval documents, and will generally be in the range $0.005 < k < 0.01$ per meter. The value depends on the quality of workmanship, on the distance between tendon supports, on the type of duct or sheath employed, and on the degree of vibration used in placing the concrete.

The above recommended values for μ and k are mean values. The actual values used in design may be increased or decreased, depending on standards of control, workmanship, special precautions, etc., provided that the selected values can be justified.

(9) Time dependent losses should be calculated from:

$$\Delta\sigma_{p,\,c+s+r} = \frac{\epsilon_s(t,\,t_o)E_s + \Delta\sigma_{pr} + n\phi(t,\,t_o)(\sigma_{cg} + \sigma_{cpo})}{1 + n\frac{A_p}{A_c}[(1 + \frac{A_c}{I_c}z_{cp}^2)(1 + 0.8\,\phi(t,\,t_o))]} \tag{B-14}$$

where $\Delta\sigma_{p,c+s+r}$ is the variation of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t.
$\epsilon_s(t,\,t_o)$ is the estimated shrinkage strain, derived from the values in Table 2.7 for final shrinkage.
n is E_s/E_{cm}
E_s is the modulus of elasticity for the prestressing steel, taken from Section B.2.1.4.4.
E_{cm} is the modulus of elasticity for the concrete Table 2.5
$\Delta\sigma_{pr}$ is the variation of stress in the tendons at section x due to relaxation. This may be derived from Fig. B.4 for a ratio of Initial stress/characteristic tensile stress, (σ_p/f_{pk}) calculated from:

$$\sigma_p = \sigma_{pgo} - 0.3\Delta\sigma_{p,c+s+r} \qquad \text{(B.15)}$$

where σ_{pgo} is the initial stress in the tendons due to prestress and permanent actions.

For simplification and conservatively, the second term in Eq.(B.15) may be ignored. For normal buildings, σ_p may be taken as 0.85 σ_{pgo}.

$\phi(t, t_o)$ is a creep coefficient, as defined in Section 2.5.4
σ_{cg} is the stress in the concrete adjacent to the tendons, due to self-weight and any other permanent actions.
σ_{cpo} is the initial stress in the concrete adjacent to the tendons, due to prestress.
A_p is the area of all the prestressing tendons at the level being considered.
A_c is the area of the concrete section.
I_c is the second moment of area of the concrete section.
z_{cp} is the distance between the center of gravity of the concrete section and the tendons.

In using Eq.(B.14), an assumed value of total loss will be required initially, to permit the term $\Delta\sigma_{pr}$ on the right hand side to be evaluated (this term depends on the level of final prestress). An iterative process is therefore necessary to solve and balance the two sides of Eq.(B.14).

(10) The loss of prestress calculated in accordance with above should be added to that determined by above to assess the final prestress $(P_{m\infty})$. It is important to remember that these procedures are approximate, and may be adjusted to suit particular materials, stressing or design conditions.

(11) The design procedures to take account of the effects of prestress should be in accordance with Section B.4.3.

B.5.5.6 *Anchorage Zones of Pretensioned Members*

(1) Where tensile forces can occur, they should be carried by additional reinforcement.

(2) A distinction has to be made (see Fig. B-4(a)) between:

(a) Transmission length l_{bp}, over which the prestressing force (P_o) from a pretensioned tendon is fully transmitted to the concrete.
(b) Dispersion length $i_{p,eff}$ over which the concrete stresses gradually disperse to a linear distribution across the concrete section.
(c) anchorage length l_{ba}, over which the ultimate tendon force (F_{pu}) in pretensioned members is fully transmitted to the concrete.

(3) The transmission length l_{bp} is influenced by the size and type of tendon, the surface condition of the tendon, the concrete strength, the degree of compaction of the concrete. Values should be based on experimental data or experience with the type of tendon to be used. For design purposes, Fig. B-4(b) the transmission length is defined as a multiple of the nominal diameter (ϕ) of the strand or wire.

$$l_{bp} = \beta_b \phi \qquad \text{(B.16)}$$

For strands having a cross-sectional area $\leq$ 100 mm^2, and for indented wires with diameter $\leq$ 8 mm, all complying with surface characteristics specified in relevant standards and tensioned according to the values given in Section B.5.5.4, the β_b values given in Table B.5 may be adopted. The concrete strength taken should be that at the moment of transfer. Where the use of ribbed wires is proposed,

with diameter $\leq$ 12 mm, values for β_b should be based on test data; as a guide, the values in Table B.5 may be adopted.

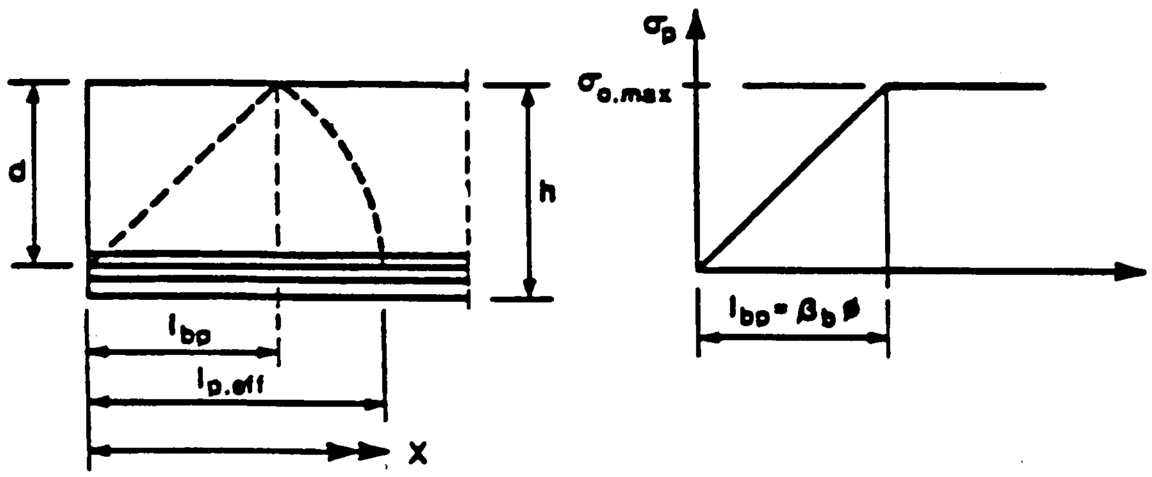

Figure B-4 Transfer of Prestress in Pretensioned Elements

Table B.5 Factors β_b to be taken for Transmission Length of Prestressing Strands and Wires (Smooth or Indented) in Relation to Concrete Strength at the Moment of Transfer

Actual concrete strength at transfer (MPa)		25	30	35	40	45	50
β_b	Strands and smooth or indented wires	75	70	65	60	55	50
	Ribbed wires	55	50	45	40	35	30

(4) The design value l_{bpd} is to be taken at 0.8 l_{bp} or 1.2 l_{bp} whichever is less favorable for the effect considered.

(5) Transmission length, anchorage length and dispersion length are to be taken from the start of effective bond.

Start of effective bond should be taken account of:

(a) tendons purposely debonded, at the end.
(b) a neutralized zone $l_{bp,o}$ in the case of sudden release

(6) For rectangular cross-sections and straight tendons, situated near the bottom of the section, the dispersion length can be established as:

$$l_{p,eff} = \sqrt{l_{bpd}^2 + d^2} \qquad \text{(B.17)}$$

(7) The anchorage of pretensioning tendons in flexural members at the ultimate limit state is influenced by the condition, cracked or uncracked, of the anchorage zone. The part of the beam

where tendons are anchored (Fig. B-4(a)] may be considered as uncracked if the concrete tensile stress at the Ultimate Limit State (flexural and principal stresses) does not exceed f_{ctd}, taking account of the relevant value of P_d. (see Section B.4.4.5).

(3) If the tensile stress does not exceed f_{ctd}, the condition of anchorage may be assumed to be fulfilled without further checks.

(8) If the tensile stress does not exceed f_{ctd}, it should be shown that the envelope of the acting tensile force does not exceed the resisting tensile force provided by the tendons and the reinforcing steel within the anchorage zone. The ultimate resisting force F_{px} of the tendons according to Fig. B-8(b) may be determined as:

$$F_{px} = \frac{x}{l_{bpd}} P_o \leq \frac{A_p f_{P\,0.1k}}{\gamma_s} \tag{B-18}$$

P_o as defined in Section B.4.3.2(1)
l_{bpd} as defined in above

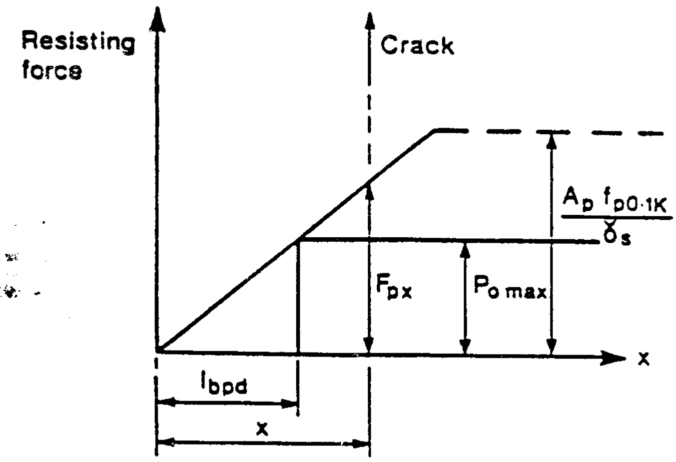

Figure B-5: Derivation of Eq. (B.18)

B.5.5.7 Anchorage Zones of Post-tensioned Members

(1) The design of anchorage zones shall be in accordance with the procedures in this section and those Sections B.4.3, B.5 B.6.5, B.6.6 and B.7.1.

(2) When considering the effects of the prestress as a concentrated force on the anchorage zone, the characteristic tensile strength of the tendon shall be used.

(3) The bearing stress behind anchorage plates should be calculated in accordance Section B.6.7.1.

(4) Tensile forces due to concentrated forces should be assessed by a strut and the tie model, or other appropriate representation (see Section B.4.2). The resulting reinforcement should be detailed in accordance with Section B.6.5 assuming that it is acting at its design strength.

(5) The prestressing force may be assumed to disperse at an angle of spread 2β (see Fig. B-6) starting at the end of the anchorage device, where β may be assumed to be arc tangent 2/3.

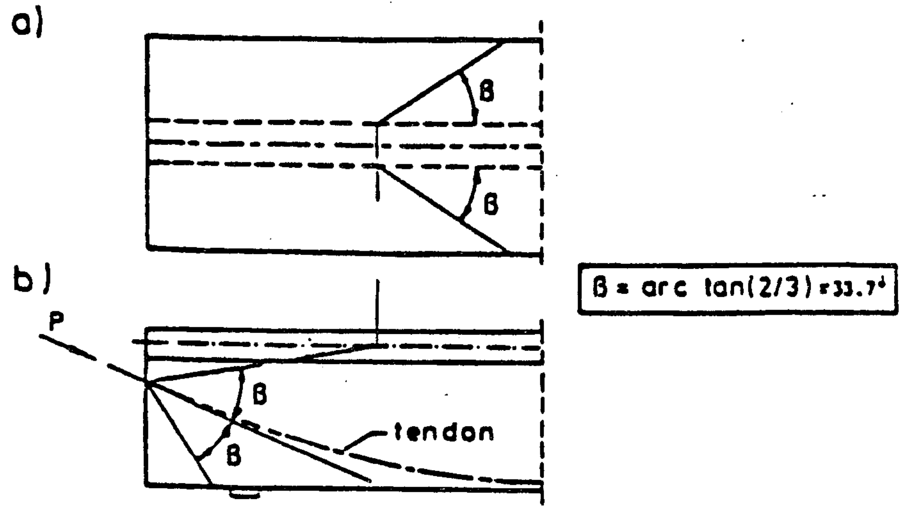

Figure B-6 Dispersion of Prestress

B.5.5.8 Design for Shear

B.5.5.8.1 ***Members with Inclined Prestressing Tendons***

(1) Taking into account the effect of inclined prestressing tendons, the design shear force is given by:

$$V_{sd} = V_{od} - V_{pd} \qquad \text{(B.19)}$$

where V_{pd} denotes the force component of the inclined prestressed tendons, parallel to V_{od}. V_{pd} is taken as positive in the same direction as V_{od}.

(2) Equation B.19 applies in combination with Eqs.4.29 and 4.30.

(3) Concerning the value V_{pd} in Eq.B.19, two cases should be distinguished :

Case 1: The stresses in the tendons do not exceed the characteristic strength $f_{p0.1k}$

The relevant prestressing force is the mean value P_{mt} allowing for losses (see Section B.4.3.2(1)) multiplied by the relevant safety coefficient (generally γ_p = 0.9).

Case 2: The steel stress in the tendons exceeds $f_{p0.1k}$

The prestressing force is calculated with $f_{p0.1k}/\gamma_s$.

(4) In shear analysis, the effective depth d is calculated ignoring the inclined tendons.

B.5.5.9 Limit State of Cracking

(1) All relevant provisions in Section 5.3 shall apply.

B.5.5.9.1 *General*

(1) The durability of prestressed members may, for humid and sea water of aggressive chemical environment types of exposure, be more critically affected by cracking. In the absence of more detailed requirements the design crack width w_k under the frequent load combination may be taken as:

(a) 0.2 mm for both post-tensioned and pre-tensioned members for dry exposure.
(b) 0.2 mm for post-tensioned and decompression for pre-tensioned members for humid exposure.
(c) Decompreesion or coating of tendons and 0.2 mm for post-tensioned members.
(d) The decompression limit requires that, under the frequent combination of loads, all parts of the tendons or duct lie at least 25 mm within concrete in compression.

B.5.5.9.2 *Minimum Reinforcement Areas*

(1) In prestressed members subject to compressive normal force the minimum reinforcement area may be reduced below that necessary for ordinary reinforced concrete due to the influence of:

(a) the increased flexural stiffness of the compression zone, and
(b) the contribution of the prestressing tendons.

(2) In prestressed members, the minimum reinforcement for crack control is not necessary in areas where, under the rare combination of actions and the relevant estimated characteristic value of prestress or normal force, the concrete remains in compression.

(3) If the conditions in (2) above are not fulfilled, the required minimum area should be calculated according to Section 5.3.2(3) with the following values for k_c.

(a) For box sections
k_c = 0.4 for webs
= 0.8 for the tension chord
(b) For rectangular sections, the value of k_c may be interpolated between 0.4 for pure bending without normal force and zero when
  (i) the condition just satisfy (2) above, or
  (ii) where, under the action of the relevant estimated value of prestress, the depth of the tension zone, calculated on the basis of a cracked section under the loading conditions leading to formation of the first crack, does not exceed the lesser of $h/2$ or 0.5 m.

(4) Prestressing tendons may be taken into account as minimum reinforcement within a 300 mm square surrounding the tendon, provided that the difference bond behaviour of the tendons and reinforcement are taken into account. In the absence of better information, this may be done by assuming prestressing tendons to be 50% effective.

B.5.9.9.3 *Control of Cracking without Direct Calculation*

(1) For prestressed slabs in buildings subjected to bending without significant axial tension, measures specifically to control cracking are not necessary where the overall depth does not exceed 200 mm and the provisions Section 7.2.2 have been applied.

(2) For prestressed concrete sections, Section 5.3.4.2 and Table 5.2 may be applied with w_k = .2 mm. The stresses in the reinforcement should be calculated regarding the prestress as an external force without allowing for stress increase in the tendons due to loading.

B.6 DETAILING PROVISIONS

B.6.1 Arrangement of the Prestressing Units

(1) In the case of pre-tensioning, the tendons shall be spaced apart.

(2) In the case of post-tensioned members, bundled ducts are not normally permitted.

(3) A pair of ducts, placed vertically on above the other, may be used if adequate precautions are taken for tensioning and grouting. Particular care is necessary if the tendons are doubly curved.

B.6.2 Concrete Cover

(1) The concrete cover between the inner surface of the formwork and either a pre-tensioned tendon or a duct shall be fixed with due regard to the size of the tendons of the duct. Minimum covers shall be in accordance with Section 7.1.3, in addition to the following:

(a) For pre-tensioned members, the minimum cover shall not be less then 2ϕ, where ϕ is the diameter of a tendon. Where ribbed wires are used, the minimum cover shall not be less then 3ϕ.

(b) For post-tensioned members, the minimum cover is to the duct. The cover shall not be less than the diameter of the duct. For rectangular ducts, the cover shall not be less than the lesser dimension of the duct cross-section nor half the greater dimension of the duct.

B.6.3 Horizontal and Vertical Spacing

(1) The spacing of ducts or of pre-tensioned tendons shall be such as to ensure that placing and compacting of the concrete can be carried out satisfactorily and that good bond can be attained between the concrete and the tendons.

B.6.3.1 *Pre-tensioning*

(1) The minimum clear horizontal and vertical spacing of individual tendons is given in Fig. B-7.

B.6.3.2 *Pos-tensioning*

(1) Except for paired ducts (see Section B.5.1.1(3)), the minimum clear spacing between individual ducts should be:

(a) Horizontal: $\phi \geq 40$ mm
(b) Vertical: $\phi \geq 50$ mm
where ϕ denotes the diameter of the duct.

B.6.4 Anchorages and Couplers for Prestressing Tendons

(1) The anchorage devices used for post-tensioned tendons and the anchorage lengths in the case of pre-tensioned tendons shall be such as to enable the full design strength of the tendons to be developed, taking account of any repeated, rapidly changing action effects.

(2) Where couplers are used, these shall be so placed by taking account of the interference caused by theses devices, i.e. that they do not affect the bearing capacity of the member and that any temporary anchorage which may be needed during construction can be introduced in satisfactory manner.

B.6 DETAILING PROVISIONS

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(3) A pair of ducts, placed vertically on above the other, may be used if adequate precautions are taken for tensioning and grouting. Particular care is necessary if the tendons are doubly curved.

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(a) For pre-tensioned members, the minimum cover shall not be less then 2ϕ, where ϕ is the diameter of a tendon. Where ribbed wires are used, the minimum cover shall not be less then 3ϕ.

(b) For post-tensioned members, the minimum cover is to the duct. The cover shall not be less than the diameter of the duct. For rectangular ducts, the cover shall not be less than the lesser dimension of the duct cross-section nor half the greater dimension of the duct.

B.6.3 Horizontal and Vertical Spacing

(1) The spacing of ducts or of pre-tensioned tendons shall be such as to ensure that placing and compacting of the concrete can be carried out satisfactorily and that good bond can be attained between the concrete and the tendons.

B.6.3.1 *Pre-tensioning*

(1) The minimum clear horizontal and vertical spacing of individual tendons is given in Fig. B-7.

B.6.3.2 *Pos-tensioning*

(1) Except for paired ducts (see Section B.5.1.1(3)), the minimum clear spacing between individual ducts should be:

(a) Horizontal: $\phi \geq 40$ mm
(b) Vertical: $\phi \geq 50$ mm
where ϕ denotes the diameter of the duct.

B.6.4 Anchorages and Couplers for Prestressing Tendons

(1) The anchorage devices used for post-tensioned tendons and the anchorage lengths in the case of pre-tensioned tendons shall be such as to enable the full design strength of the tendons to be developed, taking account of any repeated, rapidly changing action effects.

(2) Where couplers are used, these shall be so placed by taking account of the interference caused by theses devices, i.e. that they do not affect the bearing capacity of the member and that any temporary anchorage which may be needed during construction can be introduced in satisfactory manner.

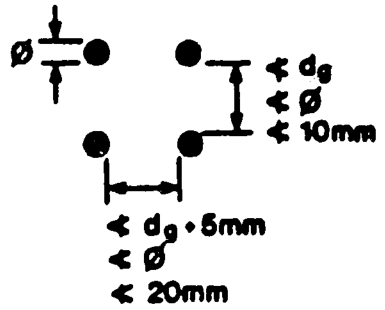

Figure B-7 Minimum Clear Spacing for Pretensioned Tendons

(3) Calculations for local effects in the concrete and for the transverse reinforcements should be made in accordance with Section B.4.2.

(4) In general, couplers should be located away from intermediate supports.

(5) The placing of couplers on 50% or more of the tendons at one cross-section should be avoided.

B.6.5 Anchorage Zones for Post-Tensioning Forces

(1) Anchorage zones should always be provided with distributed reinforcement near all surfaces in the form of an orthogonal mesh.

(2) Where groups of post-tensioned cables are located at a certain distance from each other, suitable links should be arranged at the ends of the members, as a protection against splitting.

(3) At any part of the zone, the reinforcement ratio on either side of the block should be at least 0.15% in both directions.

(4) All reinforcement should be fully anchored.

(5) Where a strut and tie model has been used to determine the transverse tensile force, the following detailing rules should be followed:

- (a) The steel area actually required to provide the tie force, acting at its design strength, should be distributed in accordance with the actual tensile stress distribution, i.e. over a length of the block approximately equal to its greatest lateral dimension.
- (b) Closed stirrups should be used for anchorage purposes.
- (c) All the anchorage reinforcement should preferably be formed into a 3-dimensional orthogonal grid.

(6) Special attention should be given to anchorage zones having cross sections different in shape from that of the general cross-section of the beam.

B.7 CONSTRUCTION AND WORKMANSHIP

B.7.1 Objectives

(1) This Section provides, in addition to those given in Chapter 8 of this Code, minimum specification requirements for prestressing steel and for the standard of workmanship that must be achieved on site in order to ensure that the design assumptions in this Code are valid and hence that the intended levels of safety and of durability will be attained.

B.7.2 Basic Requirements

(1) Prestressing steel shall comply with the requirements of Section B.2.1 of this Code.

(2) The prestressing devices (anchorages, couplers, sheaths and ducts) shall comply with the requirements of Section B.2.2 of this Code.

(3) The tendons (wire, bars, cables), anchorage devices, couplers and sheaths used shall be those in the project design documents. They shall be capable of being identified as such.

B.7.3 Transport and Storage of the Tendons

(1) Tendons, sheaths, anchorage devices and couplers shall be protected from harmful influences during transport and storage and also when placed in the structure, until after concreting has taken place.

(2) During transport and storage of the tendons, the following should be avoided:

(a) any type of chemical, electro-chemical or biological attack liable to cause corrosion;
(b) any damage to the tendons;
(c) any contamination liable to affect the durability or bond properties of the tendons;
(d) any deformation of the tendons, not provided for the design;
(e) any unprotected storage, exposure to rain or contact with the ground;
(f) the use of water transport without suitable packaging;
(g) welding in the vicinity of prestressing tendons without the provision of special protection (from splashes).

(3) For sheaths, the following should be taken into consideration:

(a) local damage and corrosion inside should be avoided;
(b) water-tightness should be ensured;
(c) it should be resistant to mechanical and chemical attack.

B.7.4 Fabrication of Tendons

(1) The devices used in jointing the tendons, for their anchorage and coupling shall be as specified in relevant Standards. The prestressing members shall be assembled and placed in position in accordance with the relevant Standards. The sheaths and their connections shall be as specified in the project design documents.

(2) Particular consideration should be given to:
(a) maintaining the identification marks on all materials;
(b) the appropriate methods for cutting;
(c) the straight entry into the anchorage and couplers as required by the manufacturer;
(d) assembly;
(e) transportation; when lifting by crane, any local crushing or bending of the tendons should be avoided.

B.7.5 Placing of the Tendons

(1) Placing of the tendons shall be carried out in compliance with the criteria relating to:
(a) the concrete cover and the spacing of the tendons;

(b) the permissible tolerances in respect of the position of the tendons, couplers and anchorages;
(c) the ease with which the concrete can be cast.

(2) The tolerances required for the placing of the prestressing tendons shall be those given in Section 8.2. alternatively they shall be stated in the contract documents.

(3) The sheaths should be fixed carefully according to the designer's specification of dimensions, spacers and supports.

(4) After placing the sheaths in position, vents should be provided at both ends and at their high points, as well as at all points where air or water may accumulate; in the case of sheaths of considerable length, vents are also needed at intermediate positions.

(5) The sheaths should be protected from penetration of extraneous materials until the completion of grouting.

B.7.6 Tensioning of the Tendons

(1) Prestressing shall be in accordance with a pre-arranged stressing program.

(2) Written instructions shall be provided at the site or in the works on the prestressing procedure to followed.

(3) Workmen and staff engaged in stressing shall be skilled and have had special training.

(4) During prestressing, suitable safety measures should be taken and be recorded by an engineer.

B.7.6.1 Pre-tensioning

(1) In the case of pre-tensioning the instructions for prestressing shall specify:

(a) the prestressing tendons and the prestressing devices;
(b) any special sequence in which the prestressing tendons are to be tensioned;
(c) the jack pressure or the forces at the jacks which must not be exceeded;
(d) the final pressure which must be attained after stressing has been completed or the corresponding forces at the jack;
(e) the maximum permissible extension of the tendons and slip in the anchorage;
(f) the manner and sequence in which the tendons are to be released;
(g) the required concrete strength at the time of release, which should be checked;
(h) operational suitability of re-useable anchorage components.

(2) the necessity for temporary protection of the tendons after tensioning and before casting should be checked. Where necessary, the protective material should not affect bond and should have no detrimental effect on the steel or the concrete.

B.7.6.2 Post-tensioning

(1) The following shall be specified by the designer:

(a) the prestressing process to be employed;
(b) the type and grade of the prestressing steel;
(c) the number of bars or wires in the individual tendons;

(d) the required concrete strength prior to tensioning;
(e) the order in which successive tendons should be tensioned, specifying the location where the tension is to be applied;
(f) where appropriate, the time of the removal of the falsework during tensioning;
(g) the force required to be developed at the jack;
(h) the design elongation required;
(i) the maximum slip;
(j) the number, type and location of couplers.

(2) The following should be recorded by the supervising engineer during the tensioning process:

(a) the type of prestressing devices used which should be calibrated;
(b) the elongation measured on site;
(c) the measured pressure in the jacks;
(d) the observed value of slip;
(e) the deviation of the measured values from the design values;
(f) the actual concrete strength;
(g) the actual order in which successive tendons are tensioned;
(h) where appropriate, the time at which the formwork has been removed.

B.7.7 Grouting and other Protective Measures

(1) Tendons placed in sheaths or ducts in the concrete, couplers and anchorage devices shall be protected against corrosion.

(2) should the delay between stressing and grouting exceed the time permitted, then protection of the tendons shall continue until grouting takes place.

(3) Where temporary protection is provided, the material used shall have an approval document and shall not have a deleterious effect on the prestressing steel or on the cement grout.

(4) Written instructions shall be provided for the site or the works for the preparation and execution of the grouting.

(5) Corrosion protection of the tendons is ensured by filling all voids with a suitable grouting material (usually cement mortar); as a rule, the anchorage should be enveloped in concrete or mortar. This objective is met by:

(a) using approved grout materials (must remain alkaline, no harmful components) and by covering the tendons completely;
(b) filling the ducts completely (including voids between tendons) with a grout which after hardening fulfills the structural requirements (strength, bond, modulus of elasticity, shrinkage).

B.7.7.2 Cement Grout

(1) The cement grout used shall have adequate properties, for example:

(a) high fluidity and cohesion when plastic;
(b) low shrinkage deformation when hardening;
(c) no loss of fines ("bleeding").

(2) Appropriate materials (types of cement, admixtures) shall be used and the mixing process (batching, water-cement ratio. procedure, time) shall ensure the required properties.

(c) Chlorides (as % by mass cement) from all sources shall not exceed the values given in the specified Standards.

B.7.7.3 Instructions to the Site

(1) Before grouting starts, the following preconditions shall be fulfilled:

- (a) equipment operational (including "stand by" grout pump to avoid interruptions in the event of malfunction)
- (b) permanent supplies of water under pressure and of compressed air;
- (c) materials batched (excess to allow for overflow)
- (d) ducts free of harmful material (e.g. water);
- (e) vents prepared and identified;
- (f) preparation of control tests for grout;
- (g) in case of doubt, grouting trail on representative ducts;
- (h) grout flow not affected.

(2) The grouting program shall specify:

- (a) the characteristics of the equipment and the grout;
- (b) order of blowing and washing operations;
- (c) order of grouting operations and fresh grout tests (fluidity, segregation);
- (d) grout volume to be prepared for each stage of injection;
- (e) precautions to keep ducts clear;
- (f) instructions in the event of incidents and harmful climatic conditions;
- (h) where necessary, additional grouting.

B.7.7.4 Grouting Operations

(1) Before injecting, it should be checked that the grouting program can be fulfilled.

(2) The injecting process should be carried out at a continuous and steady rate. In some circumstances (large diameter, vertical or inclined ducts) post-injection may be necessary to replace bleed water by grout.

(3) After completion of grouting, loss of grout from the duct should be prevented. To allow expansion of grout during hardening and to displace bleed water, appropriate vents may be opened.

(4) After injecting, if large voids are suspected, the effectiveness of grouting should be checked with appropriate equipment.

B.7.7.5 Sealing

(1) Where necessary, all openings, grouting tubes and vents shall be sealed hermetically to prevent penetration of water and harmful products.

B.7.7.6 Other Protections

(1) Tendons may be protected by materials based on bitumen, epoxy resins, rubber, etc, provided that there are no detrimental effects on bond, fire resistance, and other essential properties.

B.8 QUALITY CONTROL

B.8.1 Objectives

(1) This Section provides, in addition to those given in Chapter 9 of this Code, the minimum control measures for design and construction of prestressed concrete members. They comprise essential actions and decisions, as well as check to be made, in compliance with specifications, standards and the general state-of-the-art, to ensure that all specified requirements are met.

B.8.2 Compliance Controls

(1) For prestressing steels and prestressing devices, Section B.7 shall apply.

B.8.2 Control prior to Concreting and during Prestressing

(1) Before being placed in position, the tendons should for any damage that might have occurred since arrival on site or at the factory.

(3) Before tensioning it is advisable to check that the prestressing operation can be carried out correctly. Checks should be made that the requirements of Section B.7.6 are being met, at the time of transfer of the prestressing force.

(4) A prestressing record should be kept of the measurement made at each stage of stressing (pressure in the jacks, elongations, slippage at the anchorages, etc.).

(5) The time elapsed between prestressing and the completion of the protective measures for the steel (grouting) should be controlled and noted.

Before grouting, it should be ensured that the provisions of Sections B.7.7.3 and B.7.7.4 are applied and checked.

(6) During grouting it is necessary to check the injection pressure, the free flow of the grout from the vents, to look for grout leaks, to check the quantity of injected grout as well as to take samples for checking viscosity and loss of water. Where necessary, the strength of the grout should be checked.

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