TURKISH STANDARDS

TS500

FEBRUARY 2000

REQUIREMENTS FOR DESIGN AND CONSTRUCTION OF REINFORCED CONCRETE STRUCTURES

ANKARA TURKEY

2003

CONTENTS

REFERENCES

TS 22-1 Env 413/March 1998	“Masonry Cement Part 1: Specification”
TS 498/ November 1987	“Design Loads for Buildings”
TS 647/November 1979	“Building Code for Timber Structures”
TS 648/December 1980	“Building Code for Steel Structures”
TS 706/December 1980	“Aggregates for Concrete”
TS 708/March 1996	“Steel Bars for Concrete”
TS 802/January 1985	“Design Concrete Mixes”
TS 1247/March 1984	“Mixing, Placing and Curing of Concrete - Abnormal Weather Conditions”
TS 1248/April 1989	“Mixing, Placing and Curing of Concrete - Abnormal Weather Conditions”
TS 2940/January 1978	“Methods of Sampling Fresh Concrete”
TS 3068/March 1978	“Making and Curing Concrete Test Specimens in The Laboratory”
TS 3114/December 1990	“Determination of Compressive Strength of Concrete Test Specimens”
TS 3351/April 1979	“Making and Curing Concrete Test Specimens in The Field”
TS 3440/May 1982	“Rules for Making Concrete Exposed to Aggressive Effects of Liquids, Soils and Gases”
TS 3452	“Chemical Admixtures for Concrete”
TS ISO 9194/November 1997	“Bases For Design of Structures; Actions due to The Self-Weight of Structures, Non- structural Elements and Stored Materials; Density”
TS 11222/July 1994	“Concrete - Ready Mixed Concrete”

Ministry of Construction and Settlement, 1998. Specifications for structures to be built in disaster areas.

The Turkish Earthquake Code is available in English on the web at the following addresses:

http://www.metu.edu.tr/home/wwwdmc/code.html

http://www.koeri.boun.edu.tr/depremmuh/FINEND-999.pdf

SUBJECT, DEFINITIONS, SCOPE

0.1 - SUBJECT

This standard provides rules and minimum requirements for the safe design, detailing and construction of reinforced concrete structural elements and structures in accordance with their design purpose and service life.

0.2 - DEFINITIONS

0.2.1 - Compression Block

The actual stress distribution in the compression zone of a reinforced concrete member under bending is defined as the compression block.

0.2.2 - Concrete Cover

The concrete cover is the distance between the center of gravity of the longitudinal reinforcement and the extreme concrete fiber.

0.2.3 - Torsion

0.2.3.1 - Equilibrium Torsion

Equilibrium torsion is the torsion that must be included in the calculations to achieve equilibrium of the structural system at the ultimate and the serviceability limit states.

0.2.3.2 - Compatibility Torsion

Compatibility torsion is the torsion without the inclusion of which equilibrium of the structural system can still be achieved at the ultimate and the serviceability limit states.

0.2.4 - Strength

Strength is the value of the maximum stress that a material can resist under the conditions given in its definition.

0.2.4.1 - Concrete Compressive Strength

Concrete compressive strength is the maximum stress that a 28 day old standard cylindrical concrete specimen with 150mm diameter and 300mm height can resist. The specimen should be cured in accordance with the related standard and loaded under uniaxial compression at a speed in accordance with the related standard.

0.2.4.2 - Concrete Characteristic Strength

Concrete characteristic strength is the strength value obtained from statistical data by which the concrete grade is specified. The probability of obtaining a strength value lower than the characteristic strength is specified (generally 10 percent).

0.2.4.3 - Yield Strength of Reinforcement

The yield strength of reinforcement is the value of stress carried at the instant of yielding of the steel under axial tension.

0.2.4.4 - Tensile Strength of Reinforcement

The tensile strength of reinforcement is the maximum value of stress carried by the steel reinforcement tested under axial tension.

0.2.4.5 - Characteristic Yield Strength of Reinforcement

The characteristic yield strength of reinforcement is the yield strength value obtained from statistical data by which the steel grade is specified. The probability of obtaining a strength value lower than the characteristic yield strength of reinforcement is specified. In practice, the characteristic yield strength is the lowest value of yield strength envisaged in the specifications for that grade of steel.

0.2.4.6 - Average Strength

The average strength of a material is the average of strength values obtained from an adequate number of tests.

0.2.4.7 - Design Strength

Design strength is obtained by dividing the characteristic strength by a material factor, which can be equal to or greater than 1.0, to achieve a certain margin of safety. In design calculations, the design strength value should be used.

0.2.5 - Balanced Reinforcement Ratio

The reinforcement ratio which enables a reinforced concrete beam under bending to reach its ultimate strength by simultaneous yielding of the tension steel reinforcement and crushing of the extreme concrete fiber is defined as the balanced reinforcement ratio.

0.2.6 - Moment Redistribution
Moment redistribution is the transfer of moments from sections, which have reached their moment capacities, to other sections due to the ductile flexural behavior of under-reinforced members.

0.2.7 - Equivalent Compression Block
In the case of ultimate strength analysis, the resultant of the compression block and its location are important. In order to simplify the analysis, an equivalent compression block that has a simpler shape than the actual compression block can be used provided that its volume and the location of its centroid are more or less the same as those of the actual compression block.

0.2.8 - Effective Length
The effective length is the distance between two inflection points (zero moment points) of a member.

0.2.9 - Moment of Inertia
0.2.9.1 - Cracked Section Moment of Inertia
The cracked section moment of inertia is the moment of inertia of the cracked cross sectional area with respect to the neutral axis.

0.2.9.2 - Gross Section Moment of Inertia
The gross section moment of inertia is the moment of inertia of the gross cross sectional area with respect to the neutral axis.

0.2.10 - Effective Depth
The effective depth is the distance between the center of gravity of the tension reinforcement and the extreme concrete compression fiber.

0.2.11 - Development Length
The development length is the length of the embedded reinforcement required to develop the design yield strength of the reinforcing bar at the critical section.

0.2.12 - Cross Sectional Area
0.2.12.1 - Cracked Cross Sectional Area
The cracked cross sectional area is obtained by ignoring the concrete area in tension and by transforming the longitudinal reinforcement area into an equivalent concrete area.

0.2.12.2 - Gross Cross Sectional Area
The gross cross sectional area is obtained by neglecting the longitudinal reinforcement area and assuming an uncracked concrete section.

0.2.13 - Material Factor
Material factors are coefficients, equal to or bigger than 1.0, by which characteristic strength values are divided in order to obtain design values resulting in a certain level of safety.

0.2.14 - Slenderness Effect
The slenderness effect is the behavior that necessitates the inclusion of the second order moments in the design or analysis of a column.

0.2.15 - Clear Concrete Cover
The clear concrete cover is the distance between the extreme concrete fiber and the external surface of the outermost reinforcing bar or tie.

0.2.16 - Limit State
0.2.16.1 - Serviceability Limit State
In this limit state, ways of preventing problems such as excessive deformation, cracking, or vibration of structural members that would render their use difficult and impair their serviceability are evaluated.

0.2.16.2 - Ultimate Limit State
In this limit state, the ultimate capacities of structural members at the onset of the collapse stage are evaluated with respect to structural safety.

0.2.17 - Structural Analysis
0.2.17.1 - First Order Structural Analysis
First order structural analysis is based on the geometry of the undeformed structural system. In this type of analysis changes in the values of the internal forces due to deformation are ignored.

0.2.17.2 - Second Order Structural Analysis
Second order structural analysis is based on the geometry of the deformed structural system. Changes in the values of the internal forces due to deformation are included in this type of analysis.

0.2.18 - Loading

Loading comprises various physical actions that may affect a structure throughout its service life and which need to be considered in design such as vertical (gravity) loads, horizontal loads (earthquake loading, wind loading etc.), and the effects of differential settlement of foundations, temperature changes, deformation due to creep, shrinkage etc.

0.2.18.1 - Live Load

Loads which will not remain on structures for long periods during their service life are called live loads.

0.2.18.2 - Permanent Load

Loads which will remain on structures throughout their service life (e.g. the weight of load carrying members or other non-structural members) are called permanent loads.

0.2.18.3 - Sustained Load

Permanent loads and the live loads known to be acting on the structure for long periods are called sustained loads.

0.2.19 - Load Combinations

As the probability of various load types affecting a structure simultaneously is low, load combinations are arrangements of those load types for which the probability of acting together is considerably high.

0.2.20 - Load Effect

Load effects are internal force resultants (e.g. bending moment, shear force, torsional moment) that are produced during the service life of the structure by various physical effects such as vertical and horizontal loads, imposed deformations, temperature changes etc., which are assumed to possess a statistical distribution.

0.2.20.1 - Design Load Effect

The design load effect is the load effect considered in design calculations to achieve a certain level of safety. Design load effect is obtained by multiplying the characteristic load effect value by a factor that can be equal to or greater than 1.0 and by substituting the obtained value into a suitable load combination.

0.2.20.2 - Characteristic Load Effect

The characteristic load effect is established on the basis of statistical data and the probability of exceedance of this load effect during the lifetime of the structure is in general 10 percent.

0.2.21 - Load Factors

Factors by which characteristic load values are multiplied to obtain load values to be used in design in order to achieve a certain level of safety are called load factors.

0.3 - SCOPE

This standard provides minimum requirements for the safe design, detailing and construction of reinforced concrete structural elements and structures in accordance with their design purpose and service life.

This standard does not apply to reinforced concrete structures that are constructed by using high strength concrete (higher than 50 MPa). When high strength concrete is used, calculations should be proven to be correct by using relevant references.

In case of structures to be constructed in seismic regions, in addition to basic principles and requirements given in this standard, the requirements of “Specifications for Structures to be Built in Disaster Areas”, published by the Turkish Ministry of Public Works and Settlement, should be satisfied as well.

In case of special structures such as water tanks, nuclear structures, silos, in addition to requirements of this standard, requirements of standards for the special structure under consideration should be satisfied as well.

1 - NOTATION AND UNITS AND EMPLOYED

1.0 - NOTATION

a	Equivalent rectangular compression block depth, k_1c
	Spacing of hairpins
	Concrete cover measured from centre of gravity of tension reinforcement
a_v	Distance between loading point and support face in case of short cantilevers and brackets
A_s'	Cross sectional area of compression reinforcement
A_c	Concrete area of web section
	Concrete area of web section in case of beams
	Gross area of concrete section in case of columns
A_e	Area enclosed by boundary line connecting centroids of reinforcing bars located at the corners of the section
	Area enclosed by center lines of walls in case of hollow box sections
A_g	Gross area of reinforced concrete wall
A_n	Cross-sectional area of reinforcement necessary for resisting horizontal force in case of short cantilevers and brackets
a_n	Clear distance between parallel beams
A_o	Cross sectional area of stirrup bar or tie
A_{ot}	Cross sectional area of tie resisting torsion (single bar)
A_{ov}	Cross sectional area of tie resisting shear (single bar)
A_s	Cross sectional area of tension reinforcement
	Cross sectional area of flexural reinforcement
A_{sl}	Cross sectional area of longitudinal reinforcement necessary for resisting torsion
	Cross sectional area of skin reinforcement in beams
A_{sh}	Total cross sectional area of horizontal reinforcement in case of walls
A_{st}	Total cross sectional area of horizontal reinforcement resisting bending and horizontal forces in case of brackets and short cantilevers
	Total cross sectional area of longitudinal reinforcement in case of columns
A_{sv}	Cross sectional area of total horizontal reinforcement in the region between top face and 2/3 of the depth of section in case of brackets and short cantilevers
A_{sw}	Total cross sectional area of shear reinforcement
A_t	Effective concrete area for each tensile reinforcing bar in members, $A_t=2ab_w/n$
A_v	Cross sectional area of shear reinforcement perpendicular to tension reinforcement in deep beams
A_{vh}	Cross sectional area of shear reinforcement parallel to tension reinforcement in deep beams
A_{wf}	Cross sectional area of shear–friction reinforcement
b	Width of footing under wall
	Effective flange width for beam
	Cross sectional dimension of column
b_1, b_2	Dimensions of smallest possible rectangle enclosing critical punching perimeter u_p (dimension along eccentricity b_1)
b_w	Width of ribs in a ribbed slab
	Web width of beam
b_x,b_y	Dimensions of critical punching perimeter u_p in x and y directions
c	Concrete cover measured from centre of gravity of extreme reinforcing bar
	Distance from extreme compression fiber to neutral axis
c_c	Clear concrete cover
C_m	Moment coefficient in case of buckling
d	In slabs, average of effective depth in two directions
d	Effective depth in case of flexural members
d	In footings, average of effective depth values in two directions
d'	Concrete cover measured from centroid of compression reinforcement
d_m	Minimum bend diameter
d_o	Diameter of circular column or diameter of circular load area
E	Earthquake effect
e	Eccentricity
e	Eccentricity in plane of bending to be considered in calculations
e	Clear distance between two successive ribs
e_{min}	Minimum eccentricity

e_x, e_y	Eccentricity in x and y directions
E_c	Modulus of elasticity of concrete
E_{cb}	Modulus of elasticity of concrete in beams
E_{cj}	Modulus of elasticity of "j" days old concrete
E_{cs}	Modulus of elasticity of concrete in slabs
E_s	Modulus of elasticity of reinforcement ($=2\times10^5$ MPa)
EI	Flexural stiffness
E_cI_c	Flexural stiffness of gross concrete section of column
E_sI_s	Flexural stiffness of longitudinal reinforcement
f_{cd}	Design compressive strength of concrete
f_{ck}	Characteristic compressive strength of concrete
f_{ckj}	Characteristic compressive strength of "j" days old concrete
f_{cm}	Average compressive strength of concrete
f_{cmin}	Minimum concrete compressive strength
f_{ctd}	Design tensile strength of concrete
f_{ctk}	Characteristic tensile strength of concrete
f_{ctm}	Average axial tensile strength of concrete
f_{su}	Maximum strength of longitudinal reinforcement
f_{yd}	Design yield strength of longitudinal reinforcement
f_{yk}	Characteristic yield strength of longitudinal reinforcement
f_{ywd}	Design yield strength of transverse reinforcement
F	Force
F_a	Sum of slab loads within critical punching perimeter (u_p)
F_d	Design force
	Design load effect
F_k	Characteristic force
G	Permanent load effect
	Shear modulus
G_{cj}	Shear modulus of "j" days old concrete
h	Slab thickness
	Member depth
	Total depth of beam
	Cross sectional dimension of column on its bending plane
H	Horizontal force effect (e.g. earth pressure)
H_d	Horizontal design force in case of brackets and short cantilevers
I	Moment of inertia
I_b	Gross moment of inertia of beam
I_c	Gross moment of inertia of column
I_{cr}	Cracked moment of inertia about neutral axis
I_{ef}	Effective moment of inertia
I_s	Gross moment of inertia of slab section
i	Radius of gyration
k	Column effective length factor
k_1	Equivalent rectangular compression block depth factor
ℓ	Width of part of footing under wall that projects beyond the wall
	Span length to be used in calculations
	Column length (measured from axis to axis)
ℓ_o	Lap splice length
ℓ_1	Span length of slab between support axes in direction considered
ℓ_2	Span length of slab between support axes in the direction perpendicular to considered direction
ℓ_b	Development length
ℓ_{bk}	Development length with hook
ℓ_e	Equivalent thickness
ℓ_i	ith storey column length (measured from axis to axis)
ℓ_k	Column effective (buckling) length
ℓ_n	Clear span of slab in the direction under consideration
	Clear length of member
	Clear span measured from one support face to the other
ℓ_p	Clear distance between two inflection points of beam
ℓ_l	Span length between support axes in long direction of slab

ℓ_s	Span length between support axes in short direction of slab
ℓ_{sn}	Clear span length in short direction of slab
ℓ_t	Span length in case of members being tested In slabs, length of short side The bigger value of the sum of the distance between two support axes and the sum of the clear span and member depth (Twice the clear span in cantilevers)
m	Ratio of long side to short side of slab $m=\ell_\ell / \ell_s$
m_d	Design bending moment per unit width of slab
M_1, M_2	Column end moments
M_{cr}	Cracking moment of member under bending
M_d	Design bending moment
M_{d1}, M_{d2}	Plate design bending moments at column face
M_{max}	Maximum bending moment of member
M_o	Total static moment
n	Number of reinforcing bars in a bundle Number of tension bars. Number of transverse reinforcing bars in mesh reinforcement Number of stirrup arms at a section
N_1, N_2	Axial loads of top and bottom columns in case of punching shear
N_d	Design axial force
N_{gd}	Permanent load contribution to design axial force
N_k	Column buckling load
p	Distributed slab load
p_d	Uniformly distributed slab design load
p_g	Uniformly distributed slab dead load
p_q	Uniformly distributed slab live load
Q	Live load effect
q_{sp}	Soil reaction
r	Ratio of spliced reinforcement to total reinforcement in a section
R_d	Design strength
R_m	Creep coefficient
s	Stirrup spacing
s_h	Spacing of shear reinforcement parallel to tension reinforcement in a deep beam
S	Shape factor in torsion
t	Plate thickness of ribbed slab
t	Thickness of beam flange (deck)
t_o	Thickness of thickened slab region in flat plate slabs
T	Load effect due to changing temperature, shrinkage, differential settlement etc.
T_{cr}	Cracking strength of section under torsion
T_d	Design torsional moment
t_e	Wall thickness of a box section
u	Perimeter of section
u_e	Perimeter of area A_e
u_p	Critical perimeter for punching shear (with distance d/2 from loaded area)
V	Shear force
V_c	Contribution of concrete to shear strength
V_{cr}	Cracking strength of section under shear
V_d	Design shear force
V_{fi}	Total shear force at ith storey
V_{gd}	Permanent load contribution to design shear force
V_{pd}	Design value of punching shear force
V_{pr}	Punching shear strength
V_r	Shear strength
V_w	Contribution of shear reinforcement to shear strength
W	Wind effect
W_m	Section modulus of area within critical punching perimeter (u_p)
x,y	Short and long side lengths of rectangles making up a T-section in case of torsion calculations
y	Distance between extreme tension fiber and neutral axis
α	Slab moment coefficient
α_1, α_2	Ratio of $\Sigma(EI/\ell_c)$ of compression members to $\Sigma(EI/\ell)$ of flexural members in a plane at one end of a compression member ($\alpha_1 \leq \alpha_2$)
α_1	Lap splice length coefficient

α_f	Small angle that shear – friction reinforcement makes with the shear plane
α_m	Average of α_1 and α_2
α_s	Ratio of sum of continuous side lengths to sum of all side lengths for slabs
α_t	Thermal expansion coefficient
β	Ratio of beam flexural stiffness to flexural stiffness of plate having a width of ℓ_2, $\beta=E_{cb}I_b/E_{cs}I_s$
β	Moment magnification factor for a column
β_s	Moment magnification factor for all columns of a storey
δ_i	Instantaneous deflection
δ_t	Total deflection
δ_{ig}	instantaneous deflection due to permanent loads
Δ_i	Relative drift at ith storey
ΔM	Amount of support moment reduction
Δ_{m1}	Maximum deflection measured under load in first loading test
Δ_{m2}	Maximum deflection measured under load in second loading test
Δ_{mp1}	Maximum residual deflection measured in first loading test
Δ_{mp2}	Maximum residual deflection measured in second loading test
ε_s	Strain in the reinforcement
ε_{ce}	Creep strain
ε_{cs}	Shrinkage strain
ε_{cu}	Unit shortening at concrete crushing
ε_{sm}	Average reinforcement strain in the region between cracks
ε_{su}	Elongation of reinforcement at maximum stress
ϕ	Diameter of longitudinal reinforcement (the largest of different diameters)
ϕ	Diameter of circular section (diameter of largest circle that can be inscribed in a polygonal section)
ϕ_e	Equivalent reinforcement diameter in case of bundled bars
ϕ_{ce}	Creep Coefficient
ϕ_ℓ	Smallest diameter of longitudinal reinforcement
γ	Factor reflecting effect of axial force on cracking strength
γ_m	Factor reflecting effect of bending on punching shear
γ_m	Material factor
γ_{mc}	Material factor for concrete
γ_{ms}	Material factor for reinforcing steel
γ_t	Factor for permanent load
η	Factor to be used in punching shear calculations
φ	Stability index
λ	Permanent deflection multiplier
μ	Shear – friction coefficient
μ_c	Poisson ratio for concrete
ρ	Ratio of tension reinforcement in beams, A_s/b_wd
ρ'	Ratio of compression reinforcement in beams, A'_s/b_wd
ρ_b	Balanced reinforcement ratio
ρ_t	Total ratio of longitudinal reinforcement in columns
ρ_{max}	Maximum ratio of reinforcement in beams
ρ_{min}	Minimum reinforcement ratio
ρ_{tmax}	Maximum ratio of longitudinal reinforcement in columns
σ_s	Stress in reinforcement
	Reinforcement stress calculated by assuming cracked section
σ_{co}	Nominal stress in concrete under permanent loading
σ_{sr}	Reinforcement stress at first cracking calculated by assuming cracked section
ω	Crack width
ω_{max}	Maximum allowable crack width

1.1 - UNITS

SI units are used in this code.

2 - DOCUMENTS RELATED TO THE STRUCTURE

2.1 - GENERAL

Documents that are prepared for all aspects of design and inspection of RC structures both before construction and for application during construction should at least include all of the information required by this section. In addition to these documents, geotechnical site investigation reports and, if applicable, reports explaining special methods to be used in construction and maintenance should be preserved with care throughout the serviceability lifetime of the structure.

2.2 - STRUCTURAL ANALYSIS AND DESIGN CALCULATIONS

Hand or computer analysis and design calculations should start by providing the following information in summary form under the title "Design Principles":

- Sketches explaining the structural system
- Soil type, foundation level, soil characteristics and safe bearing capacity of soil
- Specified steel reinforcement grade
- Specified concrete grade
- Related load specification, information about special loading cases if applicable
- Information obtained from earthquake code (effective ground acceleration, ductility level, response factor, etc.)
- Environmental conditions affecting structure and corresponding maximum crack width values
- Allowable horizontal and vertical deflection limits
- Stability index of the structure and its independent parts
- Names of standards, specifications and sources used

The static and dynamic analysis results should be presented in a clear and easily understandable format. If equations or charts other than those given in related standards have been used in the analyses and design calculations, this should be stated and copies of sources used should be attached to calculations.

In case of computer analyses, program inputs should be provided in a clear format so that program outputs can be checked by carrying out hand calculations or using other programs if required. If the computer program used has not been accepted as adequate for the structure type under consideration by the competent authorities, its manuals should also be submitted for verification. Equilibrium equations should always be shown to be satisfied in program outputs.

2.3 - DRAWINGS

For reinforced concrete structures, drawings should be provided at each design stage. These drawings should give required information in adequate detail and scale for the design stage under consideration. At least the following information should be provided for application drawings showing formwork plan and reinforcement detail:

- Specified concrete grade
- Cement type and the number of the related standard
- Specified reinforcement grade
- Nominal maximum size of coarse aggregate
- Concrete workability
- Warnings and recommendations about formwork and formwork removal
- Clear cover values for different faces of different members
- In case of foundation drawings, soil level, soil conditions, safe bearing capacity of soil, measures to be taken for soil underneath the foundation
- Information giving relationships between drawings
- Working loads considered in design and maximum loads that may develop during construction
- Position and dimension of holes, pipes and within concrete
- Schedules for steel bars showing dimensions and bending details.

Apart from the required information mentioned above, additional drawings and explanatory notes should be provided for illustrating arrangements at construction joints and expansion joints.

2.4 - INSPECTION OF CONSTRUCTION

The inspection of construction should be conducted by authorized technical personnel with care to ensure the realization of the conditions envisaged in the design project.

A journal or notebook showing the general progress of work with dates must be maintained at the construction site. In this journal, information about aggregates and how their characteristics alter with time, water/cement ratio, concrete quality, workability (slump) of fresh concrete, natural water content of aggregates, measured dilatation or swelling of sand specimens taken, frost (dates and duration) and daily temperature changes should be provided. This journal should be made available to the building owner or the inspector whenever requested.

2.5 - CHANGES IN THE PROJECT

Any project alterations, which are considered necessary due to problems arising during construction, should be approved by the design engineer and recorded in signed documents, after which the sanction of related authorities should be obtained.

3 - MATERIALS

3.0 - NOTATION USED

A_c	Web area of concrete section in case of beams
	Gross area of concrete section in case of columns
E_c	Modulus of elasticity of concrete (28 days old)
E_{cj}	Modulus of elasticity of j days old concrete
E_s	Modulus of elasticity of reinforcing steel (=2×10^5 MPa)
f_{ck}	Characteristic compressive strength of concrete (28 days old)
f_{ckj}	Characteristic compressive strength of j days old concrete
f_{cm}	Average compressive strength of concrete
f_{cmin}	Minimum concrete compressive strength
f_{ctk}	Characteristic tensile strength of concrete (28 days old)
f_{ctm}	Average tensile strength of concrete
f_{yk}	Characteristic yield strength of longitudinal reinforcement
f_{su}	Rupture strength of longitudinal reinforcement
G	Shear modulus (28 days old)
G_{cj}	Shear modulus of j days old concrete
l_e	Equivalent thickness
u	Perimeter of member cross-section
α_t	Thermal expansion coefficient
ε_{ce}	Creep strain
ε_{cs}	Shrinkage strain
ε_{su}	Rupture strain of the longitudinal reinforcement
ϕ_{ce}	Creep Coefficient
μ_c	Poisson ratio for concrete
σ_{co}	Nominal stress in concrete under sustained loading

3.1 - CONCRETE MATERIALS

3.1.2 - Cement

Cement, specified by the designer should be checked by means of tests to ensure strength, strength increase and durability characteristics of concrete. Cement must be transported to the construction site in accordance with the related standards and stored at the construction site properly so that its properties are preserved.

3.1.3 - Aggregates

Aggregates to be used in concrete should conform to TS 706. In the selection of the aggregates, the characteristics of the structure as well as environmental conditions should be taken into account.

The grading of aggregates plays an important role in determining concrete characteristics and therefore the most suitable granulometric grading of the aggregates to be used should be obtained by conducting tests. The nominal maximum size of coarse aggregate shall not be larger than any of the following: 1/5 of the formwork breadth, 1/3 of the slab depth, 3/4 of the spacing between two longitudinal reinforcing bars and the cover.

3.1.4 - Water

Water used in the concrete mix should be in accordance with related standards. Mixing water should not be acidic (pH value should be $\geq$7); it should be free from injurious amounts of carbonic acid, manganese compounds, ammonium salts, free chlorine, mineral oils, organic materials and industrial waste. Water may contain maximum 15g of dissolved and 2g of floating mineral salt and a maximum of 2g of SO_3 per liter. Seawater must not be used in mixing concrete containing high alumina cement.

3.1.5 - Chemical Admixtures

Concrete chemical admixtures to be used in concrete should satisfy the requirements of TS 3452.

3.2 - Steel Reinforcement

Reinforcement should satisfy the requirements of TS 708. Some of the important properties of reinforcing steel specified in TS 708 are shown in Table 3.1.

The modulus of elasticity of reinforcement steel is 2×10^5 MPa.

Welding should not be applied to cold worked reinforcing bars. For weldable reinforcing bars of normal hardness, welding may be used only if their carbon equivalents (as defined in TS 708) do not exceed 0.4.

Table 3.1 Mechanical Properties of Reinforcing Bars (Taken from TS 708)

Mechanical Characteristics		Reinforcing Bars					Mesh Reinforcement
		Normal Hardness			Cold worked		
		S220a	S420a	S500a	S420b	S500bs	S500bk
Minimum yield strength, f_{yk} (MPa)		220	420	500	420	500	500
Minimum rupture strength, f_{su} (MPa)		340	500	550	550	550	550
Minimum rupture elongation, ε_{su} (%)	$\phi \leq 32$	18	12	12	10	8	5
	$32 < \phi \leq 50$	18	10	10	10	8	5

3.3 - Concrete

For construction, concrete made on site or ready-mix concrete can be used. If concrete is prepared on site, materials to be used in the mix should be automatically weighed and mixed in a mixer to satisfy the concrete properties specified by the designer. If ready-mix concrete is used, the requirements of TS11222 should be satisfied.

3.3.1 - Concrete Grades and Concrete Compressive Strength

Concrete is classified according to its compressive strength. Compressive strength should be determined by testing 28 days old standard cylindrical concrete specimens with 150 mm diameter and 300 mm height in accordance with requirements of TS 3068. Concrete characteristic compressive strength f_{ck} is the value for which the probability of obtaining a lower compressive strength from tests on cylinders is a definite ratio (in general 10 percent). The compressive strength of concrete can be obtained from cube tests as well. Under such circumstances, the concrete characteristic compressive strength f_{ck} can be derived by making use of conversion coefficients validated by tests. Equivalent strength for 150 mm cubes are given in Table 3.2. For 200 mm cube specimens the cube strength given in the table should be reduced by 5 percent.

Table 3.2 Concrete Grades and Strengths

Concrete Grade	Characteristic Compressive Strength f_{ck} MPa	Equivalent Cube (200 mm) Compressive Strength MPa	Characteristic Axial Tension Strength f_{ctk} MPa	28 Day Elastic Modulus E_c MPa
C16	16	20	1.4	27 000
C18	18	22	1.5	27 500
C20	20	25	1.6	28 000
C25	25	30	1.8	30 000
C30	30	37	1.9	32 000
C35	35	45	2.1	33 000
C40	40	50	2.2	34 000
C45	45	55	2.3	36 000
C50	50	60	2.5	37 000

3.3.2 - Concrete Tensile Strength

Concrete tensile strength should be obtained from axial tension tests. The average tensile strength obtained from tests is f_{ctm} whereas f_{ctk} is the characteristic tensile strength. The concrete characteristic tensile strength f_{ctk} is the value for which the probability of obtaining a lower tensile strength from tests on axial tension specimens is a definite ratio (in general 10percent). The concrete characteristic tensile strength shall be calculated by using the equation given below:

$$f_{ctk} = 0.35\sqrt{f_{ck}} \quad (MPa) \tag{3.1}$$

For certain concrete grades, the tensile strength values obtained from this equation are given in Table 3.2. Concrete tensile strength can also be obtained from flexural tests and split cylinder tests. The tensile strength f_{ctk} can approximately be obtained by dividing the tensile strength values obtained from the split cylinder test by 1.5 and the tensile strength values from the flexural test result by 2.0.

3.3.3 - Modulus of Elasticity, Shear Modulus, Poisson Ratio and Thermal Expansion Coefficient of Concrete

3.3.3.1 - Modulus of Elasticity

For normal weight "j" days old concrete, the modulus of elasticity can be calculated from the following equation:

$$E_{cj} = 3250\sqrt{f_{ckj}} + 14000 \quad (MPa) \qquad (3.2)$$

The f_{ckj} value in this equation is the characteristic compressive strength of j days old concrete. For impact loading, modulus of elasticity values obtained from Equation 3.2 should be increased by 10 percent. E_c values for 28-day-old different concrete are given in Table 3.2 for different concrete grades. Results obtained from Equation 3.2 are secant moduli of elasticity corresponding to the $0.4f_{ck}$ stress level.

3.3.3.2 - Poisson Ratio

The Poisson ratio of concrete can be taken as μ_c=0.2.

3.3.3.3 - Shear Modulus

The shear modulus of concrete can be calculated approximately from the equation given below.

$$G_{cj} = 0.40 E_{cj} \qquad (3.3)$$

3.3.3.4 - Thermal Expansion Coefficient

The thermal expansion coefficient of concrete may be taken as $\alpha_t = 10^{-5}/^0C$

3.3.4 - Time Dependent Deformation of Concrete

In case of inadequate data, coefficients related to creep of concrete can be obtained from Table 3.3 and coefficients related to shrinkage of concrete can be obtained from table 3.4. Values given in these tables are reached within 2-3 years. For shorter periods, methods proven to be correct should be used. Linear interpolation should be employed to find intermediate values of relative humidity, age of concrete and fictitious thickness.

3.4 - QUALITY CONTROL and Acceptance of Concrete Quality

At the construction site, concrete compressive strength should be determined by conducting quality tests on specimens cured in accordance with TS 3351. The site engineer may require hardening tests for specimens that are to be stored under site conditions. As mentioned in section 3.3.1, these tests are conducted on 150mmX300mm standard cylinders. In compelling situations, cube specimens may be employed. For evaluation, groups each consisting of three cylindrical or cubic specimens should be considered.

Table 3.3 Creep Coefficient ϕ_{ce} (for long periods)

Concrete Age at Loading	Dry environment (relative humidity 50percent)			Moist environment (relative humidity (80percent)		
	Equivalent Thickness (mm), $\ell_e = 2A_c / u$					
	50	150	600	50	150	600
1 day	5.4	4.4	3.6	3.5	3.0	2.6
7 day	3.9	3.2	2.5	2.5	2.1	1.9
28 days	3.2	2.5	2.0	1.9	1.7	1.5
90 days	2.6	2.1	1.6	1.6	1.4	1.2
365 days	2.0	1.6	1.2	1.2	1.0	1.0

Note: $\varepsilon_{ce} = \dfrac{\sigma_{co}}{E_c}\phi_{ce}$

Table 3.4 Shrinkage Strains, $\varepsilon_{cs} \times 10^3$ (for long periods)

Concrete Maintenance	Environment (Relative humidity 50percent)		Moist environment (Relative humidity 80percent)	
	Equivalent Thickness (mm), $\ell_e = 2A_c / u$			
	150	600	150	600
Inadequate	0.60	0.50	0.40	0.30
Adequate	0.40	0.40	0.25	0.25

At least one group (3 specimens) of test specimens should be taken from each production unit for quality control. A production unit is composed of concrete having the same target design strength and the same ingredients in the same mix proportions. Furthermore, one unit can not exceed $100m^3$ (by volume) or 450 m^2 (by area) of concrete placed on the same day. From each batch, at least 3 groups (9 specimens) should be taken as samples. The specimens of each group shall be stored under standard conditions and tested under uniaxial compression. Each specimen shall be taken from a different transmixer or concrete mixer. If more than one specimen is taken from the same concrete mixer, they should be counted as one specimen and their average strength value should be taken into consideration. Specimens should be taken, cured and prepared in accordance with TS 2940, TS 3068 and TS 3351 and tests should be carried out in accordance with TS 3114.

When ready mixed concrete is used, in addition to the specimens taken at the plant, specimens, in manner and number a described above, should be taken at the construction site as well. The evaluation should primarily be based on the specimens taken at the construction site. Specimen, groups of three cylinders each, should successively be named as G_1, G_2, $G_3 \ldots G_n$ in the order in which they are taken and the average compressive strength of each group should be obtained.

If each successive party consisting of 3 groups named as P_1 (G_1, G_2, G_3), P_2 (G_2, G_3, G_4), P_3 (G_3, G_4, G_5) $\ldots P_{n-2}$ do not satisfy both of the requirements mentioned below, the concrete will be rejected.

Average of each party $f_{cm} \geq f_{ck} + 1.0\ MPa$

Minimum group average in each party $f_{cmin} \geq f_{ck} - 3.0\ MPa$

Specimens taken for hardening test should be stored at site conditions and tested at a suitable time as required. The purpose of conducting the hardening test, which should be carried out on at least 3 specimens, is to check whether the curing and storing of concrete are satisfactory or not, and to establish the time required for the removal of the formwork.

If the results obtained from the quality control tests do not satisfy the requirements mentioned above, the load bearing capacity of structure or of the structural members involved should be re-evaluated by using the low concrete strength obtained. In case significant strength reduction is obtained, necessary remedial measures should be taken.

The supervising engineer may require tests to determine the compressive strength of concrete in the structure whenever it is necessary. This can be done by taking cores from locations that will not result in any strength reduction in the structure or by making use of non-destructive testing methods (surface hardness, sound velocity tests, etc.). In case of inspection with non-destructive tests, correlation of results with the concrete used in that structure should be established.

4 - PREPARING CONCRETE AND REINFORCEMENT

4.1 - GENERAL

Before starting construction work, mix calculations should be carried out in accordance with TS 802 with the object of obtaining the required average concrete compressive strength (f_{cm}), which would provide the concrete characteristic compressive strength used in design (f_{ck}). By preparing trial mixes, it should be checked that the required concrete properties can be obtained.

After arriving at the mix proportions that would provide the required concrete characteristics, the mix materials at the site and the resulting concrete obtained should be tested at frequent intervals.

4.2 - MIXING, CONVEYING, DEPOSITING AND CURING CONCRETE

Mix proportions should be prepared according to weight, and calculations on a volume basis are not allowed. Mixing, conveying, depositing and curing of concrete should be in accordance with TS 1247, TS 1248 and/or TS 11222; if harmful underground water and the effects of gases are present, the requirements of TS3440 should be satisfied.

4.3 - PLACING REINFORCEMENT

Reinforcing steel should be cleaned free of dirt, oil and surface rust before use. Special care should be taken to place reinforcement in the location specified in the design drawings. Primary tension or compression reinforcement should be tied up properly with distribution reinforcement and ties.

In columns, longitudinal reinforcement should be confined with ties. Before casting concrete, the site engineer and supervising engineer should check whether the amount and placement of the reinforcement are in accordance with the design drawings and prepare a signed record to this effect.

While casting concrete, no change should occur in the position of the reinforcement. To achieve the required concrete cover around the reinforcing bars, the reinforcement should be lifted up and concrete spacing blocks (chairs) should be placed between the formwork and reinforcing bars, and bits of steel bar should be placed as spacers between two layers of reinforcing bars. Instead of concrete and steel spacers, small plastic blocks produced for this purpose can be used. Special care should be taken to cover ties with concrete completely on the sides and in the case of slabs and beams, to prevent the top reinforcement from being forced downwards. It must be ensured that reinforcement is fully covered by concrete.

In case of structural members which will be placed directly on soil (as in foundation slabs), a concrete layer at least 50 mm thick should be provided as cover between the soil and the structural member, depending on the type of soil.

5 - FORMWORKS AND SCAFFOLDING

5.1 - GENERAL

All formworks and scaffolding should have adequate stability and safe load carrying capacity. Formwork and scaffold members should be prepared, connected and set up in accordance with related codes so as to prevent excessive deformation and settlement. Timber formwork and scaffold members should be designated to conform to TS 647 and steel formwork and scaffold members should be designated to conform to TS 648.

Planks forming the timber formwork should be sufficiently tight to prevent leakage of grout during concrete compaction and they should have adequate strength against vibrator effects.

It should be ensured that all loads, which will act on the formworks during their use, should be carried safely. Special care should be taken in case of formworks and scaffolds that are supported by intermediate slabs or other structural members as during the construction of an additional storey or in strengthening and rehabilitation work.

Vertical loads from scaffolds should be transferred to the soil ensuring load distribution in an appropriate manner, and special precautions should be taken in case of loose soil and frozen soil conditions. Load transfer and transmission should be provided by inserting well-finished timber wedges, which have adequate strength and which would remain in position underneath the vertical struts (for this purpose brick or stone should not be used). When these supports cannot be constructed as a single piece and instead need to be arranged in layers, safety against overturning should be provided. In case of inclined struts, safety against sliding should be provided as well.

During the placement of the reinforcement and casting concrete, scaffolding for work safety should be provided.

Formwork and scaffolding should be organized so as to be dismantled easily and safely without shock or vibration. Use should be made of available dismantling procedures using wedges, sand boxes, bolts, jacks, etc.

In order to ensure that structural members with long spans will acquire the position envisaged in the design after removal of formworks and scaffolds the formworks and scaffolds should be provided with the appropriate amount of camber.

Before concrete placement, the interior of the formwork should be well cleaned; if required they should be wetted. For this purpose cleaning holes should be provided in the bottom regions of columns, at the free ends of cantilevers and at the soffits of deep beams.

Before and during casting concrete, all formwork and scaffolds should be checked carefully.

Without the permission of the supervising engineer, materials should not be placed or piled up on the formwork.

5.2 - FORMWORK AND SCAFFOLDING LOADS

While designing formworks and scaffolds, the self-weight of the formworks and scaffolds, the self-weight of freshly placed or piled-up concrete, the self-weight of vehicles being used to carry concrete, impact effects occurring while casting concrete and the self-weight of laborers should be considered as vertical loads.

As horizontal loads, other than the wind effect, tension in cables (if applicable) and the horizontal components of the support reactions of inclined members etc should be considered.

When compaction of concrete is carried out with a vibrator, especially in case of concrete in a plastic or fluid state, the hydrostatic lateral pressure effect transferred to the forms due to concrete possessing a density of 25 kN/m^3 should be considered.

5.3 - DRAWINGS

In case of multi-storey and free standing (self supporting) scaffolds, formwork and scaffold design drawings required for strength and stability checks should be provided.

5.4 - FORMWORK AND SCAFFOLD SUPPORTS

All formworks and scaffolds should be supported both horizontally and vertically so that they can transfer horizontal forces to the soil safely. Scaffold support bracing should generally be arranged to form a triangular truss pattern.

Bracing bars should be arranged in such a manner that as far as possible no bending moments are produced in the vertical struts.

Truss-like bracing does not need to be provided only if the struts are restrained from moment by being connected near the support regions and along the strut to fixed points or strong walls by encastering or similar measures.

During the setting up (assembly) stage, formworks and scaffolds should possess adequate rigidity.

Formwork struts should be constructed in accordance with the related codes. In case of timber formworks, second and third class timber should be used. In the case of single storey structures, if the scaffold height does not exceed 5m and if all loads are transferred to the soil with vertical struts, the adequacy of whose section properties is known beforehand, there is no need for a buckling check unless there is reason for doubt. In all other cases, strength and buckling calculations should be carried out for formworks and scaffolds.

5.5 - REMOVAL OF FORMWORKS

No formwork or strut supporting any part of the structure shall be removed before presenting the test results related to concrete strength to the supervising engineer and obtaining his approval. The time required after concrete placement, for formwork removal depends on the type of cement used, the rate of concrete strength gain, the water/cement ratio, the type of structural loading, the magnitudes of the load effects and on weather conditions.

Special attention should be given to parts of the structure that are envisaged to carry loads equal to the design loads right after formwork removal.

If frost occurs during hardening of concrete, the removal of formworks should be delayed by a period of time at least equal to the frost duration. A decrease of temperature to 0^0C in the shade during a 24-hour period should be considered as frost. After the end of the frost duration, the hardening stage of concrete should be investigated before removal of formwork to decide whether the concrete has adequately set or whether it looks hardened because it is frozen.

In case of adverse weather conditions and especially in case of frost, the removal time of formwork should be decided upon on the basis of results from compression tests on specimens that have hardened under the same conditions as the concrete in the structure.

Extra struts should be maintained in their original positions for a suitable period after formwork removal. The duration of this period depends on cement type. The days on which the temperature falls below 5^0C should not be included in this period.

Under special conditions the supervising engineer may reduce the duration of these periods. However tests should be conducted to prove that the concrete has reached an adequate strength.

6 - BASIC PRINCIPLES OF REINFORCED CONCRETE DESIGN

6.0 - NOTATION

a	Support width
a_n	Clear distance between parallel beams
b	Flange width of beam
b_w	Web width of beam
E	Earthquake loading
e	Eccentricity
e_{min}	Minimum eccentricity
F	Force
F_d	Design force Design load effect
F_k	Characteristic force
f_{cd}	Design compressive strength of concrete
f_{ck}	Characteristic compressive strength of concrete
f_{ctk}	Characteristic axial tensile strength of concrete
f_{ctd}	Design axial tensile strength of concrete
f_{yd}	Design yield strength of longitudinal reinforcement
f_{yk}	Characteristic yield strength of longitudinal reinforcement
G	Permanent load effect
H	Horizontal load (e.g. earth pressure) effect
h	Cross sectional dimension of column in the direction of bending
ℓ	Span length to be used in calculations
ℓ_p	Length between two inflection (zero moment) points of beam
ΔM	Amount of support moment reduction
Q	Live load effect
R_d	Design strength
T	Load effect due to temperature change, shrinkage, differential settlement, etc.
t	Thickness of beam flange (deck)
V	Shear force
W	Wind effect
γ_m	Material coefficient
γ_{mc}	Material coefficient for concrete
γ_{ms}	Material coefficient for reinforcement
ρ	Ratio of tension reinforcement in beams
ρ'	Ratio of compression reinforcement in beams
ρ_b	Balanced reinforcement ratio

6.1 - GENERAL

The purpose of structural analysis and section design of reinforced concrete structures is to ensure that the structure during its serviceability lifetime behaves in accordance with the envisaged purpose.

Structures and structural members should be designed for a definite safety level and for continuous serviceability under all possible loads and deformations that can affect the building during the construction and utilization stages.

Design calculations for the purpose described above should be based on scientific principles, experimental data and on past experience.

The design calculations performed for reinforced concrete structures are valid only if the envisaged material strengths are achieved on site.

In case the material strengths obtained are considerably different from the specified value, the results of the design calculations will lose their validity.

6.2 - STRUCTURAL SAFETY

6.2.1 - General

In design, the required safety against collapse during the service life of the structure should be provided and in addition cracking, deformation, vibration etc. should not exceed levels which would impair the serviceability and affect the strength of the structure. To achieve this, the load effects on the structure should be magnified by load factors and material strengths should be reduced by material factors during the design stage. These factors are based on statistical data.

6.2.2 - Limit States Method

To achieve the required structural safety defined above, the important limit state conditions which may be encountered during the lifetime of the structure: (i) ultimate limit state and (ii) serviceability limit state should be checked.

6.2.3 - Ultimate Limit State

The ultimate capacity of each structural member calculated by using reduced material strengths (design strengths) as defined in section 6.2.5 shall not be smaller than the internal forces that develop due to design loads magnified as defined in section 6.2.6.

$$R_d \geq F_d \qquad (6.1)$$

6.2.4 - Serviceability Limit State

Deflection, deformation, crack width values of each structural member, calculated by using methods explained in chapter 13, should not exceed the corresponding limiting values given in the same section.

6.2.5 - Material Factors

Considering the statistical distribution of material strengths, "design strength" values to be used in design calculations should be obtained by dividing the characteristic material strengths defined in Section 3 by the corresponding "material factors" which are equal to or greater than 1.0. For the ultimate limit state, concrete and steel design strengths are given below:

Concrete: $f_{cd} = f_{ck} / \gamma_{mc}$

$f_{ctd} = f_{ctk} / \gamma_{mc}$ (6.2)

Steel: $f_{yd} = f_{yk} / \gamma_{ms}$

γ_{mc}=1.5 shall be taken for concrete cast in situ. This factor can be taken as 1.4 for precast concrete. However, if concrete quality control is not satisfactory, then with the designer's approval this factor can be taken as 1.7. For all steel grades, the material factor shall be taken as 1.15 (γ_{ms}=1.15). For calculations dealing with the serviceability limit state, material factors should usually taken as equal to 1.0.

6.2.6 - Load Factors and Load Combinations

The probability of exceeding the characteristic load effect value, F_k, during the service lifetime of structure is equal to a certain value. Characteristic load effect values given in this standard are in accordance with TS 498 and TS ISO 9194 and "Specifications for Structures to be Built in Disaster Areas" published by the Ministry of Public Works and Settlement.

In design, all possible load combinations that can possibly act on the structure should be considered. Load combinations that are usually encountered in calculations are given below:

For vertical loads only

$$F_d = 1.4G + 1.6Q \qquad (6.3)$$

$$F_d = 1.0G + 1.2Q + 1.2T \qquad (6.4)$$

T in Equation 6.4 is the load effect created by temperature change, shrinkage, differential settlement and similar deformations and displacements. This load combination should be considered only when such effects are not negligible.

When wind loading is considered, in addition to Equations 6.3 and 6.4 the following load combinations shall be taken into account:

$$F_d = 1.0G + 1.3Q + 1.3W \qquad (6.5)$$

$$F_d = 0.9G + 1.3W \qquad (6.6)$$

When earthquake loading is considered, in addition to Equations 6.3 and 6.4 the following load combinations shall be taken into account:

$$F_d = 1.0G + 1.0Q + 1.0E \qquad (6.7)$$

$$F_d = 0.9G + 1.0E \qquad (6.8)$$

When horizontal soil pressure is present, in addition to Equations 6.3 and 6.4 the following load combinations shall be taken into account:

$$F_d = 1.4G + 1.6Q \tag{6.9}$$

$$F_d = 0.9G + 1.6H \tag{6.10}$$

When fluid pressure is present, this pressure should be multiplied by 1.4 and should be added to all load combinations in which there is live load.

In case of the serviceability limit state, described above in Section 6.2.4, all load coefficients should be taken equal to 1.0.

6.3 - STRUCTURAL ANALYSIS

6.3.1 - Methods of Analysis

Internal forces, which provide the basis for the dimensioning of the sections of structural members, should be obtained by an analysis in accordance with the principles of structural mechanics. This analysis based on the stress-strain relationships of concrete and steel may be a nonlinear method or a method that is based on the assumption of linear elastic structural behavior. Moments obtained for continuous beams and slabs by using linear elastic methods, can be modified as described in Section 6.3.8; provided that the real behavior is taken into consideration and equilibrium conditions are fully satisfied.

6.3.2 - Loads

Load effects due to temperature change, shrinkage and creep are given in this standard. Others should be obtained from TS 498, TS ISO 9194 and "Specifications for Structures to be Built in Disaster Areas" prepared by the Ministry of Public Works and Settlement. In case of special structures, loads should be chosen from specifications especially formulated for those structures.

6.3.3 - Arrangement of Live Loads

For all load combinations, except those including earthquake loading, the live load should be arranged on the structure so as to provide the most critical (unsuitable) internal force effects.

6.3.4 - Expansion, Shrinkage and Creep Effects

By considering temperature change and shrinkage effects, expansion joints should be provided in the superstructure of long span indeterminate structures. In case of structures, which are open to external environmental effects, the spacing of expansion joints should not exceed 40m. In the case of frames protected against temperature changes and in the case of frame type structures that do not have stiff shear walls at the edges, the spacing of the expansion joints can be increased up to 60m. These limits may be exceeded if calculations for time dependent behavior are carried out or if measures are taken to protect the structure against shrinkage. In the case of asymmetric systems and for symmetric systems with stiff vertical members at both ends, preventing horizontal displacement, care should be taken to keep the spacing of joints shorter. In the analysis of statistically indeterminate structures, the shrinkage effect can be introduced into the calculations as temperature drop. If detailed calculations are not conducted, the amount of shortening due to shrinkage can be obtained from Table 3.4. For statically indeterminate structures, the member bending stiffness (EI) may be reduced to calculate the long-term effect of temperature change and shrinkage. Coefficients required for creep calculations can be obtained from Table 3.3. The concrete modulus of elasticity should not be reduced when strains due to shrinkage are calculated.

6.3.5 - Design Span Lengths

Span length to be used in design of beams and slabs should be the distance from the centerline of one support to the centerline of the other support. This value can be reduced up to a value of 1.05 times the clear span length value when beams and slabs are monolithically cast with the supports. When beams and slabs are not cast monolithically with the supports, this value can be reduced up to the sum of clear span value plus the member depth.

6.3.6 - Effective Flange Width

The flange width of T beams required for the cross sectional area and moment of inertia calculations can be obtained by using equations given below.

Symmetric cross-sections (T section): $$b = b_w + 0.2\ell_p \tag{6.11}$$

Non-symmetric cross-sections (L section, etc): $$b = b_1 + 0.1\ell_p \tag{6.12}$$

However the overhanging flange on each side beyond the web cannot exceed 6 times the flange thickness or half of the distance to the next beam face. ℓ_p used above is the distance between the two inflection (zero moment) points of beam. If precise calculations are not conducted, ℓ_p can be taken as:

$\ell_p = 1.0\ell$ (single span, simply supported beam)

$= 0.8\ell$ (end span of continuous beam)

$= 0.6\ell$ (internal span of continuous beam)

$= 1.5\ell$ (cantilever beam) *(6.13)*

Here, ℓ is the design span length of beam.

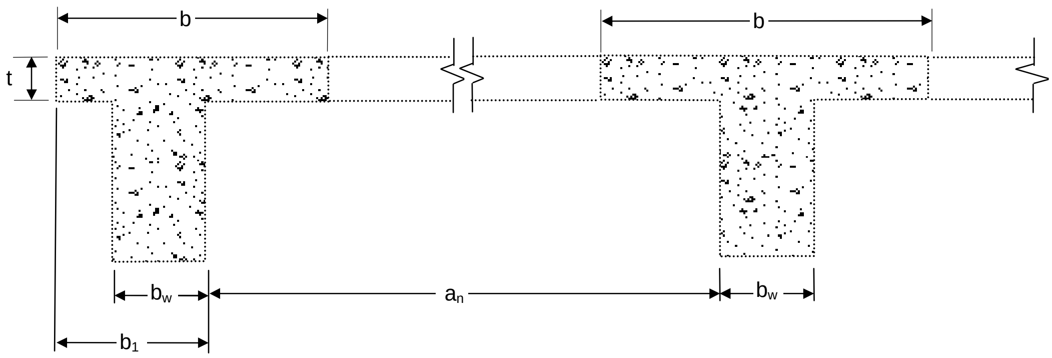


Figure 6.1 Dimensions of flanged section

6.3.7 - Stiffnesses

In case of structural analysis based on linear elastic behavior, unless otherwise stated in this standard, the gross sectional moment of inertia and the value of the modulus of elasticity of concrete given in table 3.2 should be the basis of calculations.

6.3.8 - Redistribution

In case of frame beams, continuous beams and slabs, it is possible to change support moment values, obtained from linear elastic structural analysis, by the ratios given below. However when this is done, the span moments should be recalculated to satisfy the requirements of equilibrium.

If $(\rho-\rho') \leq 0.4\rho_b$; then the maximum change can be15 percent

If $0.4\rho_b < (\rho-\rho') \leq 0.6\rho_b$; then the maximum change can be10 percent *(6.14)*

ρ_b in this equation is the balanced reinforcement ratio as defined in section 0.2.1.4.

6.3.9 - Support Moment Correction

In case of continuous flexural members, the moment calculated at the support centerline may be reduced by up to,

$$\Delta M = V \times \frac{a}{3} \quad (6.15)$$

In this equation “V” is the shear force at the support and “a” is the support width.

6.3.10 - Minimum Eccentricity Requirement

Eccentricity, based on column end moments obtained from the structural analysis, should not be less than the minimum eccentricity value obtained from Equation 6.16.

$e_{min} = 15\ mm + 0.02h$ *(6.16)*

Here “h” is the cross sectional dimension of the column in the direction of bending.

7 - SECTION DESIGN (ULTIMATE STRENGTH) – BENDING AND COMBINED BENDING

7.0 - NOTATION

A_c	Gross area of concrete section
A_s	Area of tension reinforcement
A_s'	Area of compression reinforcement
A_{sl}	Cross sectional area of intermediate reinforcement in beams
A_{st}	Total cross sectional area of longitudinal reinforcement in columns
a	Support width
	Equivalent rectangular compression block depth, k_1c
b_w	Web width of beam
C_m	Moment coefficient in case of buckling
c	Distance from extreme compression fiber to neutral axis
c_c	Clear concrete cover
d	Effective depth in beams
d'	Concrete cover measured from centroid of compression reinforcement
E	Seismic load effect
E_c	Modulus of elasticity of concrete
EI	Flexural rigidity
E_cI_c	Flexural rigidity of the gross column section
E_sI_s	Flexural rigidity of the column longitudinal reinforcement
E_s	Modulus of elasticity of reinforcement ($=2\times10^5$ MPa)
F_d	Design load effect
f_{cd}	Design compressive strength of concrete
f_{ck}	Characteristic compressive strength of concrete
f_{ctd}	Design tensile strength of concrete
f_{yd}	Design yield strength of longitudinal reinforcement
G	Permanent load effect
h	Total depth of beam
I	Moment of inertia
I_c	Gross moment of inertia of column
i	Radius of gyration
k	Column effective length coefficient
k_1	Equivalent rectangular compression block depth coefficient
l	Column length, measured from axis to axis
ℓ_i	i^{th} story column length, measured from axis to axis
ℓ_k	Column effective (buckling) length
ℓ_n	Clear span from support face to support face
	Clear length of column
M_d	Design bending moment
M_1,M_2	Column end moments (multiplied by load factors in the analysis)
N_d	Axial design load
N_{gd}	Permanent load contribution to axial design force
N_k	Column buckling load
Q	Live load effect
R_m	Creep coefficient
t	Thickness of beam flange (deck)
V_d	Design shear force
V_{fi}	Total shear force at ith story
V_{gd}	Permanent load contribution to design shear force
W	Wind effect
α_m	Average of α_1 and α_2 ratios
α_1, α_2	Column end restraint coefficients ($\alpha_1 \leq \alpha_2$)
β	Moment magnification factor for the column
β_s	Moment magnification factor for all columns of a story
Δ_i	Drift at ith story
ε_{cu}	Concrete crushing strain
ε_s	Strain in reinforcing steel
ε_{su}	Strain in reinforcing steel at maximum strength

φ	Stability index
ρ	Ratio of tension reinforcement in beams, $A_s/(b_w d)$
ρ'	Ratio of compression reinforcement in beams, $A'_s/(b_w d)$
ρ_b	Ratio of tension reinforcement for balanced case in beams
ρ_{max}	Maximum ratio of tension reinforcement in beams
ρ_{min}	Minimum ratio of tension reinforcement in beams
ρ_t	Total ratio of longitudinal reinforcement in columns
ρ_{tmax}	Maximum ratio of longitudinal reinforcement in columns
σ_s	Stress in reinforcing steel

7.1 - ASSUMPTIONS

The ultimate strength design of sections is based on the following assumptions:

- The tensile strength of concrete is neglected
- Perfect bond between reinforcing bar and surrounding concrete exists, so that the strain in the reinforcement is taken as equal to the strain of the concrete fiber at the same level.
- Plane sections remain plane after bending
- When the ultimate capacity is reached, the compressive strain in the extreme concrete fiber in compression is $\varepsilon_{cu} = 0.003$.
- Steel reinforcement behaves elasto-plastically

$$\sigma_s = E_s\,\varepsilon_s \leq f_{yd} \qquad (7.1)$$

For all reinforcing steels, the modulus of elasticity is $E_s = 2\times10^5$ MPa and the strain at the maximum strength shall be taken as 0.1.

For the concrete compressive stress distribution at the ultimate stage, any distribution providing substantial agreement with experimental results can be used. However, for simplicity in the calculations, instead of the actual compressive stress distribution, an equivalent rectangular concrete stress block with the following characteristics can be used. The width of the rectangular stress block is taken equal to the equivalent compressive strength $0.85\times f_{cd}$. The depth of the stress block can be obtained by multiplying the distance from the extreme compression fiber to the neutral axis by the coefficient k_1 i.e. $a=k_1c$. k_1 values for various concrete grades are given in Table 7.1.

Table 7.1 k_1 values for different concrete grades

Concrete Grade	C16	C18	C20	C25	C30	C35	C40	C45	C50
k_1	0.85	0.85	0.85	0.85	0.82	0.79	0.76	0.73	0.70

7.2 - BASIC PRINCIPLES

Structural analysis should be carried out using the load factors and load combinations given in section 6.2.6 and principles given in section 6.3. Proportioning of the section and calculation of the required reinforcement should be based on the most unfavorable internal force values obtained from structural analysis and using the design strength values given in Clause 6.2.5.

7.3 - REQUIREMENTS FOR FLEXURAL MEMBERS (BEAMS)

If the axial compressive force does not exceed the limit given below, such members are defined as flexural members, i.e. beams.

$$N_d \leq 0.1 f_{ck}\,A_c \qquad (7.2)$$

Of flexural members, requirements related to beams are given in this section and requirements related to slabs are given in Section 11.

The total depth of a beam shall neither be less than 300 mm nor less than three times the slab thickness. The beam web width should neither be less than 200 mm nor be more than the sum of the beam height and column width (see figure 7.1).

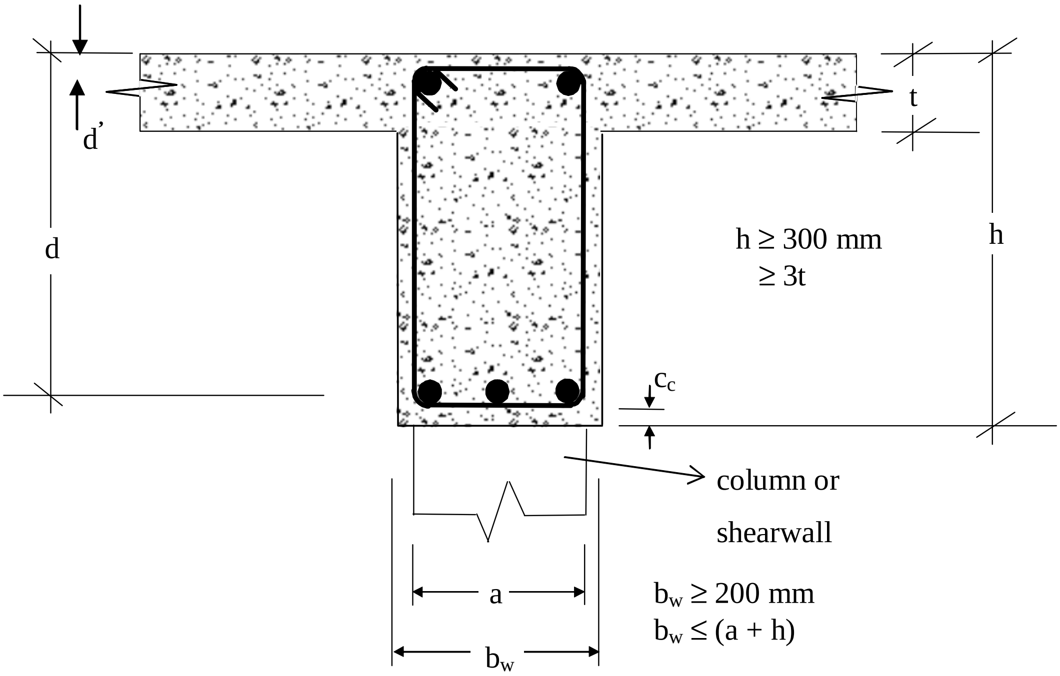


Figure 7.1 Dimensions of beam section

The ribs of ribbed slabs, secondary beams that are not frame beams, pre-tensioned and precast beams do not necessarily need to satisfy the above requirements related to beam dimensions.

The clear concrete cover of beams, c_c, except in special structures, shall not be less than 25mm for exterior members and 20mm for interior members (see figure 7.1). These values must be increased in case of unfavorable environmental conditions and when greater fire safety is needed.

The clear spacing between two reinforcing bars in a single layer and between two layers of reinforcement shall not be less than 20mm nor less than both the bar diameter and 4/3 times the nominal maximum coarse aggregate dimension. When reinforcing bars are bundled together, the equivalent diameter ϕ_e should be used (Fig. 9.4). When more than one reinforcement layer exists, reinforcing bars should be aligned exactly on top of each other.

Bars in beams should be bent beyond the theoretical bar cut-off points. This distance should not be smaller than one-third the effective depth nor less than 8 bar diameters. The distance between the theoretical and actual cut-off points for secondary, bars should not be less than the effective depth nor less than 20 bar diameters for deformed bars or 40 bars diameters for plain bars.

In beams the ratio of tension reinforcement, ρ, should not be less than the minimum values given below.

$$\rho = \frac{A_s}{b_w d} \geq \rho_{\min} = 0.8 f_{ctd} / f_{yd} \tag{7.3}$$

The difference between the tension and compression reinforcement ratios in beams should not exceed 0.85 of the balanced reinforcement.

$$\rho - \rho' \leq \rho_{\max} = 0.85\rho_b \tag{7.4}$$

In addition, the ratio of tension reinforcement should not exceed the limit given below.

$$\rho \leq 0.02 \tag{7.5}$$

In case of beams, bars with diameters less than 12 mm shall not be used as longitudinal reinforcement.

In case of beams with webs deeper than 600 mm, intermediate reinforcement should be used. The amount of the intermediate reinforcement shall be at least equal to that obtained from Equation 7.6. This reinforcement should be arranged in equal amounts at the faces of the web. The diameter of these reinforcing bars should be least 10 mm and the vertical spacing should not exceed 300 mm.

$$A_{s\ell} = 0.001 b_w d \tag{7.6}$$

At least one third of the tension reinforcement at the mid-span should extend up to the supports and be properly anchored there.
Continuous beams with clear span less than 2.5 times the total beam depth and simple beams with a clear span less than 1.5 times the total beam depth should be designed as deep beams. Non-linear strain distribution and lateral buckling should be taken into account in the design of such members.

The ratio of tension reinforcement, ρ, for deep beams, calculated using the effective depth, must satisfy the requirement given in Equation 7.3. The shear capacity design requirements for these beams are given in Section 8.5.

7.4 - REQUIREMENTS FOR MEMBERS SUBJECTED TO AXIAL FORCE AND BENDING (COLUMNS)

7.4.1 - Axial Compression and Bending

The width of rectangular column sections shall not be less than 250 mm. However in the case of I, T and L column sections, the smallest thickness can be 200 mm and in case of hollow box column sections, the smallest wall thickness can be 120 mm. In case of circular column sections, the diameter of the section shall not be less than 300 mm. In addition, the column axial load shall satisfy the following requirement:

$$N_d \leq 0.9 f_{cd} A_c \tag{7.7}$$

The clear concrete cover, c_c, of columns shall not be less than 25 mm for exterior members and 20 mm for interior members.

The total longitudinal reinforcement ratio for columns shall not be less than the value given below.

$$\rho_t = \frac{A_{st}}{A_c} \geq 0.01 \tag{7.8}$$

However if at least 1.3 times the required reinforcement is provided, this limit may be reduced to 0.005.

For columns ties, there should be at least one longitudinal reinforcing bar at each corner. For columns with spiral reinforcement, there should be at least 6 longitudinal reinforcing bars. In columns, the longitudinal reinforcing bar diameters shall not be less than 14 mm. The total longitudinal reinforcement ratio for columns shall not exceed the values given below:

Outside the lapped splice region $\rho \leq 0.04$ (7.9)

Within the lapped splice region $\rho_t \leq 0.06$ (7.10)

Longitudinal column reinforcement should be confined by transverse reinforcement over the column height. The diameter of the transverse reinforcement shall not be less than 1/3 of the largest longitudinal reinforcement diameter. The spacing of transverse reinforcement shall neither exceed 12 times the smallest longitudinal reinforcement diameter nor 200 mm. In the case of rectangular columns, the spacing of ties or cross ties should not exceed 300 mm.

7.4.2 - Axial Tension and Bending

In the design calculations of members subjected to axial tension in addition to bending, the effect of axial tension must not be ignored.

The ultimate capacity of a member, the whole section of which is subjected to tension, should be designed by ignoring the concrete contribution and considering only the contribution of the longitudinal reinforcement as given in the equation below:

$$N_d = A_{st} f_{yd} \tag{7.11}$$

The longitudinal reinforcement of such members should be arranged as symmetrically as possible. In the case of members for which the whole section is in tension, the ratio of longitudinal reinforcement must not be less than the value given below:

$$\rho_t = 1.5 \frac{f_{ctd}}{f_{yd}} \tag{7.12}$$

7.5 - MEMBERS UNDER AXIAL COMPRESSION AND BIAXIAL BENDING

The ultimate strength calculations of members under axial compression and biaxial bending should be based on the assumptions given in section 7.1 and should be conducted by using a suitable method of proven validity.

7.6 - SLENDERNESS EFFECT

7.6.1 - General Method

The design and dimensioning of members under axial compression and bending under the load combinations given in Section 6.2 should be based on axial force and moment values obtained from a second-order analysis considering material nonlinearity and cracking as well as shrinkage and creep of concrete. However, if the slenderness ratio does not exceed $\ell_k/i \leq 100$, the approximate method of section 7.6.2 can be used for the design of the section.

7.6.2 - Approximate Method (Moment Magnification Method)

This approximate method can be applied to columns whose section dimensions and axial load do not vary over their height. The design moment for the column should be obtained by magnifying the larger end moment, which is obtained from an analysis based on linear elastic behavior and satisfying the minimum eccentricity requirement of section 6.3.10, by an appropriate magnification factor.

7.6.2.1 - The Sway Criterion

If shear walls or similar members exist in a structural system providing adequate stiffness against horizontal forces, sway can be assumed to be prevented. If the column end moments obtained from a second-order analysis, which is based on linear material behavior assumption, differ at the most by 5 percent from column end moments obtained from a first order analysis, sway can also be assumed to be prevented.

If a second order analysis is not carried out, for cases where the stability index of any story, calculated by considering the whole structural system, does not exceed the limit given below, it can be assumed that adequate stiffness exists in that story and that sway is prevented.

$$\phi = 1.5\Delta_i \frac{\sum \frac{N_{di}}{\ell_i}}{V_{fi}} \leq 0.05 \tag{7.13}$$

The value to be used in Eq. (7.13) should be calculated using uncracked sections and the most critical of the following load combinations:

$F_d = 1.0G + 1.0Q + 1.0E$

and

$F_d = 1.0G + 1.3Q + 1.3W$

7.6.2.2 - Effective Column Length

The column clear length is the distance between slabs, beams or other members providing lateral support to the columns. If a drop panel or capital exists, the column clear length should be measured from the bottom face of the drop panel or capital. In cases where more accurate analysis is not carried out, the column effective length can be obtained by multiplying the column clear length with the "k" factor given below, which is related to column end restraints. The column effective length factor "k" is defined for columns in nonsway frames and for columns in sway frames, in Equations 7.14 and 7.15, respectively.

For columns in nonsway frames:

$$k = 0.7 + 0.05\,(\alpha_1 + \alpha_2) \leq (0.85 + 0.05\alpha_1) \leq 1{,}0 \tag{7.14}$$

If no calculations are carried out for such columns, k should be taken as k=1.0.

For columns in sway frames:

$$\text{If } \alpha_m < 2, \quad k = \frac{20-\alpha_m}{20}\sqrt{1+\alpha_m} \tag{7.15}$$

$$\text{If } \alpha_m \geq 2, \quad k = 0.9\sqrt{1+\alpha_m}$$

In sway frames, $k=2+0.3\alpha$ shall be used for one end hinged columns. Here, α should be calculated at the end which is not hinged.

"α" that appears in equations 7.14 and 7.15 is defined below.

$$\alpha_{1,2} = \frac{\sum\left(\frac{I}{\ell}\right)_{column}}{\sum\left(\frac{I}{\ell}\right)_{beam}}; \quad \alpha_m = 0.5(\alpha_1 + \alpha_2) \tag{7.16}$$

In calculation of the α ratios, only beams in the direction of bending shall be considered. For beams the cracked sectional moment of inertia and for columns the gross sectional moment of inertia should be used. If a more reliable calculation cannot be carried out, the cracked sectional moment of inertia can be assumed as half of the uncracked sectional moment of inertia. In calculation of the moment of inertia of T sections, the flange should be considered as well.

When equation 7.16 is applied to flat slabs, column strips, which are depicted in Figure 11.2, should be considered as beams.

7.6.2.3 - Cases where Slenderness Effect Can Be Ignored

When the slenderness ratio (ℓ_k/i) does not exceed the limits given below, the slenderness effect can be ignored and the design moment can be taken to be equal to the largest end moment that is obtained from structural analysis. In slenderness ratio calculations, the radius of gyration of a rectangular column "i" can be approximately taken as 30 percent of its dimension in the bending direction and the radius of gyration of a circular column can be taken as 25 percent of its diameter. For the calculation of the radius of gyration for other sections, the whole concrete section should be considered.

a) For columns in nonsway frames, if the condition below is satisfied,

$$(\ell_k / i) \leq 34 - 12 \ (M_1 / M_2) \leq 40 \tag{7.17}$$

the slenderness effect can be ignored. M_1 and M_2 are the column end moments that are obtained from structural analyses for each load combination ($M_1 \leq M_2$). If M_1 and M_2 lead to compression on the same column face (single curvature column) the M_1/M_2 ratio should be positive, otherwise (double curvature column) the M_1/M_2 ratio should be negative.

b) For columns in sway frames, if the condition below is satisfied,

$$(\ell_k / i) \leq 22 \tag{7.18}$$

the slenderness effect can be ignored.

7.6.2.4 - Buckling Load

The buckling load for columns is calculated by using the Euler equation.

$$N_k = \frac{\pi^2 EI}{\ell_k^2} \tag{7.19}$$

The effective flexural stiffness of the column can be obtained by using equations given below, unless more reliable calculations are carried out.

$$(EI) = \frac{0.2 E_c I_c + E_s I_s}{1 + R_m} \tag{7.20}$$

or

$$(EI) = \frac{0.4 E_c I_c}{1 + R_m} \tag{7.21}$$

Here $E_c I_c$ is the flexural stiffness of the gross concrete section, and $E_s I_s$ is the flexural stiffness of the longitudinal reinforcement relative to the center of gravity of the member.

In case of nonsway frames, the creep ratio R_m is the ratio of the portion of the axial load caused by sustained loads to the total load.

$$R_m = \frac{N_{gd}}{N_d} \tag{7.22}$$

For sway frames, the creep ratio R_m is the ratio of the sum of the portion of the shear caused by sustained loads to the story shear.

$$R_m = \frac{\sum V_{gd}}{\sum V_d} \quad \textbf{(for the whole story)} \tag{7.23}$$

7.6.2.5 - Moment Magnification Coefficient

a) Columns in nonsway frames:

Moment magnification factor β for the column shall be calculated using Eq. (7.24)

$$\beta = \frac{C_m}{1 - 1.3\dfrac{N_d}{N_k}} \geq 1.0 \tag{7.24}$$

In case of columns bent in single curvature, the (M_1/M_2) ratio is positive, and in case of columns bent in double curvature the (M_1/M_2) ratio is negative.

Here, $$C_m = 0.6 + 0.4\left(\frac{M_1}{M_2}\right) \geq 0.4 \; ; M_1 \leq M_2 \tag{7.25}$$

If any horizontal load is applied in between the column ends, C_m should be taken equal to 1.

The design moment can be obtained by using equation 7.26. The M_2 value in this equation cannot be less than the moment value that is defined in equation 6.16 considering minimum eccentricity.

$$M_d = \beta M_2 \tag{7.26}$$

b) Columns in a sway frame:

In sway frames, the moment magnification factor for the story, β_s, can be calculated from Eq. (7.27).

$$\beta_s = \frac{1}{1 - \Sigma N_d / N_k} \geq 1.0 \tag{7.27}$$

ΣN_d and ΣN_k in this equation are the sum of the axial design loads that compression members in that story carry and the sum of the critical column loads respectively. If the condition given below is not satisfied, the column dimensions should be increased.

$$\Sigma N_d \leq 0.45 \Sigma N_k \tag{7.28}$$

For each column in sway frames, the individual β value should be calculated as well. In calculating β, C_m should be taken equal to 1.0. The larger of the β and β_s values should be used in calculating the design moment (the larger of $M_d = \beta M_2$ and $\beta_s M_2$). However in case of columns whose slenderness ratios are,

$$\left(\frac{\ell_k}{i}\right) > \frac{35}{\sqrt{\dfrac{N_d}{f_{ck} A_c}}} \tag{7.29}$$

the product of the β and β_s should be used in calculating the moment ($M_d = \beta\beta_s M_2$).

7.6.2.6 - Biaxial Bending

In case of compression members that are subjected to biaxial bending, moment values in both directions should be magnified by β factors separately and the calculations should be based on these magnified moment values.

8 - SHEAR AND TORSION

8.0 - NOTATION

A_c	Web area of concrete section
A_e	Area enclosed by lines connecting the centroids of the reinforcing bars at the corners of the section Area enclosed by centre lines of walls in case of hollow box sections
A_n	Area of reinforcement required for resisting horizontal force in short cantilevers and brackets
A_o	Cross sectional area of stirrup bar
A_{ov}	Cross sectional area of stirrups required for resisting shear (single bar)
A_{ot}	Cross sectional area of stirrup required for resisting torsion (single bar)
A_s	Cross sectional area of reinforcement resisting bending
A_{sl}	Cross sectional area of longitudinal reinforcement required for resisting torsion
A_{st}	Total cross sectional area of transverse reinforcement resisting bending and Horizontal forces in brackets and short cantilevers
A_{sv}	Cross sectional area of transverse reinforcement in the region between the top face and 2/3 of the depth of the section in brackets and short cantilevers
A_{sw}	Total cross sectional area of shear reinforcement
A_v	Cross sectional area of shear reinforcement perpendicular to tension Reinforcement in deep beams
A_{vh}	Cross sectional area of shear reinforcement parallel to tension reinforcement in deep beams
A_{wf}	Cross sectional area of shear–friction reinforcement
a_v	Distance between loading point and support face in case of short cantilevers and brackets
b	Dimension of column cross section
b_w	Web width of beam
b_1,b_2	Dimensions of smallest possible rectangle enclosing critical punching perimeter (u_p)
b_x,b_y	Dimensions of critical punching perimeter (u_p) in x and y directions
d	In two way slabs average of effective depth Effective height in case of bending members
d_o	Diameter of column or circular loading
e	Eccentricity in the direction of bending
e_x,e_y	Eccentricity in x and y directions
F_a	Sum of slab loads within critical punching perimeter (u_p)
f_{cd}	Design compressive strength of concrete
f_{ctd}	Design axial tensile strength of concrete
f_{yd}	Design yield strength of longitudinal reinforcement
f_{ywd}	Design yield strength of transverse reinforcement
H_d	Horizontal design force in brackets and short cantilevers
h	Total depth of beam Cross sectional dimension of column in the direction of bending
ℓ_n	Clear span length measured from one support face to the other
N_d	Axial design force
N_1,N_2	Axial loads of top and bottom columns in punching shear calculations
M_{d1},M_{d2}	Plate design bending moments at column face
n	Number of stirrup arms at a section
p	Distributed slab load
q_{sp}	Base soil reaction
S	Shape factor in torsion
s_h	Spacing of shear reinforcement parallel to tension reinforcement in a deep beam
s	Stirrup spacing
T_{cr}	Cracking strength of section under pure torsion
T_d	Design torsional moment
t_e	Wall thickness of a hollow box section
u_e	Perimeter of area A_e
u_p	critical punching perimeter (at a distance d/2 from loaded area)
V_c	Contribution of concrete to shear strength
V_{cr}	Cracking shear strength of the section
V_d	Design shear force
V_{pd}	Design punching shear force

V_{fi} Total shear force at ith storey
V_{pr} Punching shear strength
V_r Shear strength
V_w Contribution of shear reinforcement to shear strength
W_m Section modulus of area within critical punching perimeter (u_p)
x,y Short and long side lengths of rectangles making up a flanged section in case of torsion calculations
α_f Smaller angle made by shear – friction reinforcement with the shear plane
γ Coefficient showing effect of axial force on cracking shear strength
Coefficient showing effect of bending on punching shear
A coefficient used in punching shear calculations
μ Shear – friction coefficient
μ_c Poisson ratio for concrete
ϕ Diameter of circular section (Diameter of largest circle inscribable in a polygonal section)
ϕ_ℓ Diameter of smallest longitudinal reinforcement

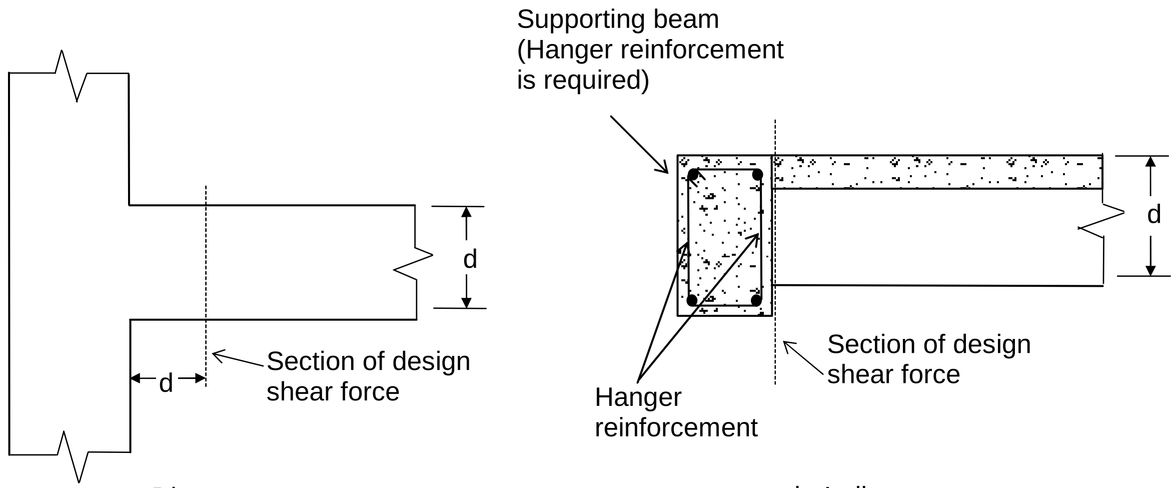
8.1 - DESIGN FOR SHEAR

8.1.1 - General

The principal tensile stresses developed when flexural moment and shear forces act together on reinforced concrete structural members, will be resisted by concrete and suitable shear reinforcement, and the principal compressive stresses developed will be maintained at such a level that no crushing will occur.

8.1.2 - Location of Critical Section for Shear

The design shear force V_d shall be calculated at a distance "d" away from the face of the support. However, when the beam support is provided by another flexural member (indirect support), the shear force at the face of the support should be considered in the design calculations.



a. Direct support b. Indirect support

Figure 8.1 Different Supports Types

When there is a possibility of a point load acting at a distance "d" away from the support face or at a nearer distance, the shear force at the support face should be used in the design calculations.

8.1.3 - Diagonal Cracking Strength

The shear cracking strength of a reinforced concrete section can be obtained from the equation below unless more detailed calculations are used. The design axial load N_d is positive in this equation whether it is a compressive or a tensile force.

$$V_{cr} = 0.65 f_{ctd}\ b_w d\left(1+\gamma\frac{N_d}{A_c}\right) \quad (8.1)$$

In the case of axial compression, γ should be taken as γ=0.07. In the case of axial tension, γ should be taken as γ=-0.3. When the axial tensile stress calculated is lower than 0.5 MPa, γ can be taken as zero.

8.1.4 - Shear Strength

For the safe shear design of sections, the following condition must be satisfied:

$$V_r \geq V_d \tag{8.2}$$

The design shear force V_d that appears in the equation above is calculated according to Clause 8.1.2. V_r is the shear strength of section. The shear strength of the section under consideration is the sum of the shear strength provided by the concrete (V_c) and the shear strength provided by the shear reinforcement (V_w):

$$V_r = V_c + V_w \tag{8.3}$$

In the general contribution of concrete to shear strength is calculated from Equation 8.4:

$$V_c = 0.8V_{cr} \tag{8.4}$$

Under seismic load combinations, for confined regions at the ends of the member, V_c values given in "Specifications for Structures to be Built in Disaster Areas" should be used.

The contribution of stirrups to shear strength can be calculated by using equation 8.5:

$$V_w = \frac{A_{sw}}{s} f_{ywd}\, d \tag{8.5}$$

If the design shear force is equal to or less than diagonal cracking strength ($V_d \leq V_{cr}$), there is no need to calculate the shear reinforcement. However, in such cases, the minimum stirrups, specified in Clause 8.1.5a, shall still be provided.

8.1.5 - Prevention of Brittle Failure

a) Minimum Shear Reinforcement

Stirrups must be provided along the whole length of reinforced concrete beams. Stirrups provided should satisfy Equation 8.6.

$$\frac{A_{sw}}{s} \geq 0.3 \frac{f_{ctd}}{f_{ywd}} b_w \tag{8.6}$$

b) Upper Limit for the Design Shear

To prevent crushing of web concrete due to high principal compressive stresses, the design shear force should be limited as below. If this requirement is not satisfied, the beam dimensions should be increased.

$$V_d \leq 0.22 f_{cd}\, b_w\, d \tag{8.7}$$

8.1.6 - Shear Reinforcement Details

To provide the required shear strength, individual reinforcing bars (vertical and horizontal stirrups, hair pins,, cross-ties etc.) and mesh reinforcement can be used. The contribution of bent bars to shear strength should be neglected. The stirrup spacing should not exceed half of the effective beam depth ($s \leq d/2$). Also when $V_d > 3V_{cr}$ the stirrup should not exceed half of the value given above ($s \leq d/4$). At the ends of frame beams over a length equal to twice the beam depth, the spacing of stirrups should satisfy the following requirements:

$s \leq d/4$

$s \leq 8\phi_\ell$

$s \leq 150\ mm$

Requirements for columns are given in Section 7.

When supports are arranged at a level higher than the bottom face of beams or when a beam is supported by another beam, reinforcement should be provided and detailed to transfer the shear force to the top of the beam.

8.1.7 - Shear-Friction

The principles and provisions of this section are to be applied in the shear strength calculations and reinforcement detailing for an interface between dissimilar materials, or between the surfaces two concretes cast at different times. For shear-friction, a crack should be assumed to exist along the shear plane under consideration. For shear-fiction, Equation 8.2 should also be satisfied. V_r in this equation should be calculated as shown below:

$$V_r = A_{wf}\, f_{yd}\, \mu \tag{8.8}$$

In this equation, shear-friction reinforcement area (A_{wf}) is equal to the sum of the cross sectional areas of reinforcing bars placed perpendicular to the shear plane only. Values of μ, the shear-friction coefficient, for different cases are given in Table 8.1.

Table 8.1 Shear-Friction Coefficients for Different Cases

Monolithically cast concrete	$\mu = 1.4$
Roughened surface at interface between hardened concrete and new concrete (roughness amplitude ≥ 5 mm)	$\mu = 1.0$
Unroughened surface	$\mu = 0.6$
Interface between steel section and concrete	$\mu = 0.7$

When the shear-friction reinforcement is inclined to the shear plane, such that the component of shear force parallel to the reinforcement tends to produce tension in the reinforcement, V_r should be calculated using the equation given below.

$$V_r = A_{wf}\, f_{yd}\left(\mu \sin\alpha_f + \cos\alpha_f\right) \tag{8.9}$$

When compression develops in the reinforcement due to the shear force, this reinforcement should be considered to be ineffective. For earthquake loading, reinforcement should be arranged so that it is perpendicular to the shear plane. The angle α_f in Equation 8.9 is the smaller angle made by the shear-friction reinforcement with the shear plane.

Shear-friction shall not be allowed to exceed the limit given below, and for the calculation of this limit, the design compressive strength of concrete should not be taken greater than 25 MPa.

$$V_d \leq 0.2\, f_{cd}\, A_c.$$

If tension forces exist across the shear plane, they should be resisted by additional reinforcement which is sufficiently anchored on both sides of shear plane. The total cross sectional area of shear-friction reinforcement may be reduced in accordance with the minimum value of any permanent compressive force acting across the shear plane.

8.2 - SHEAR AND TORSION

8.2.1 - General

In case of structural members subjected to torsion, shear and flexure, the resulting principal tensile stresses should be resisted by providing adequate reinforcement, and compressive stresses should be kept at such a level that crushing of concrete will not occur.

In structural systems, torsion is divided into two classes; (a) equilibrium torsion and (b) compatibility torsion (Section 0.2).

8.2.2 - Diagonal Cracking Limit

When shear force exists together with a torsional moment, the diagonal cracking limit should be calculated by using the equation given below.

$$\left(\frac{V_d}{V_{cr}}\right)^2 + \left(\frac{T_d}{T_{cr}}\right)^2 \leq 1 \tag{8.10}$$

V_{cr} should be obtained from equation 8.1. T_{cr} should be calculated from the equation given below:

$$T_{cr} = 1.35\, f_{ctd}\, S \tag{8.11}$$

S in equation 8.11 is the shape factor, and when precise calculations are not carried out, S can be taken from Table 8.2.

Table 8.2 Nominal torsional moment strength values for different cross sections

Rectangular sections	$S = b_w^2 h/3$
Flanged Sections	$S = \Sigma x^2 y/3$
Circular or convex polygonal sections	$S = \pi\phi^3/12$.
Thin walled (if t_e<x/5) hollow box sections	$S = 2A_e t_e$

“x” and “y” in Table 8.2 are the short and long sides of rectangles making up the flanged section. In these calculations, the width of the overhanging flange extending from the web face cannot be taken greater than three times the flange thickness.

8.2.3 - Design Forces

Design shear forces should be calculated in accordance with section 8.1.2. In cases where equilibrium torsion exists, the design torsional moments should be calculated by using elastic structural analysis. The design torsional moment calculated should not be reduced in calculating the cross-sectional dimensions. In the case of compatibility torsion, there is no need to calculate the torsional moment, as it can be assumed to be equal to the cracking moment. In case of compatibility torsion, providing the minimum amount of stirrups, calculated by using equation 8.17, is adequate.

$$T_d = T_{cr} \text{ (compatibility torsion)} \qquad (8.12)$$

When T_d is less than given by $T_d \leq 0.65\, f_{ctd}\, S$, torsion can be neglected. When this is the case, the minimum amount of stirrups for shear only calculated using Equation 8.6 is adequate.

8.2.4 - Strength

If the design shear force and the torsional moment satisfy Equation 8.10, there is no need to calculate the stirrups. However, the minimum amount of stirrups and longitudinal reinforcements given in section 8.2.5 must be provided. If the design shear force and torsional moment do not satisfy Equation 8.10, the required reinforcement should be calculated by using the equations given below:

$$\frac{A_o}{s} = \frac{A_{ov}}{s} + \frac{A_{ot}}{s} \qquad (8.13)$$

$$\frac{A_{ov}}{s} = \frac{(V_d - V_c)}{d\; n\, f_{ywd}} \qquad (8.14)$$

$$\frac{A_{ot}}{s} = \frac{T_d}{2\, A_e\, f_{ywd}} \qquad (8.15)$$

When there is more than one stirrup in a given section, the contribution of the inner stirrup legs should not be considered in the torsional resistance. When torsion exists, the cross sectional area of stirrups surrounding an area A_e cannot be less than A_{ot}. Longitudinal bars having the same volume of stirrups should be provided as given in Equation 8.16.

$$A_{s\ell} = \frac{A_{ot}}{s}\; u_e\; \frac{f_{ywd}}{f_{yd}} \qquad (8.16)$$

Longitudinal reinforcement that is required to resist bending and axial forces should be calculated separately and should be provided in addition to the longitudinal reinforcement for resisting torsion mentioned above. The calculated stirrup and longitudinal reinforcement cross sectional areas cannot be less than the minimum values given in Clause 8.2.5.a.

In case of compatibility torsion, providing the minimum amount of reinforcement given in Clause 8.2.5.a is adequate. However this reinforcement cannot be less than the amount of reinforcement required for shear resistance.

8.2.5 - Prevention of Brittle Failure

a) Minimum Reinforcement

The minimum amount of stirrup and longitudinal reinforcement as given below must be provided to prevent brittle failure due to principal tensile stresses.

Minimum Stirrups:

$$\frac{A_o}{s} \geq 0.15 \frac{f_{ctd}}{f_{ywd}} \left(1 + 1.3 \frac{T_d}{V_d b_w} \right) b_w \qquad (8.17)$$

In Equation 8.17, $T_d/V_d b_w$ should not be taken greater than 1.0. For compatibility torsion, T_d can be taken be equal to T_{cr}.

Minimum Longitudinal Bars:

$$A_{s\ell} = \frac{T_d u_e}{2 f_{yd} A_e} \qquad (8.18)$$

b) Upper Limit

To prevent crushing of concrete due to high principal compressive stresses, the load effects have been limited as given below. If this condition is not satisfied, the beam dimensions should be increased.

$$\left(\frac{T_d}{S} + \frac{V_d}{b_w d} \right) \leq 0.22 f_{cd} \qquad (8.19)$$

8.2.6 - Reinforcement Details

In cases where shear forces and torsional moments act together, the stirrups and longitudinal reinforcements to be provided should satisfy the requirements given below.

When torsion cannot be ignored, stirrups having lapped 90^0 hooks at the ends shall not be used. Instead, closed stirrups with 135^o hooks should be used and stirrup ends should be anchored within the core. The stirrup spacing should not exceed the limits given below.

$s \leq d/2$

$s \leq u_e/2$

$s \leq 300$ mm

Longitudinal reinforcement required to resist torsion should be distributed around the circumference of the section and the diameter of bars to be placed at each corner of the section should not be less than 12mm. The distance between longitudinal reinforcing bars should not exceed 300mm.

8.3 - PUNCHING SHEAR

8.3.1 - Punching Shear Strength

The punching shear strength of slabs loaded over a limited area or by means of columns should be calculated and the punching shear strength should be shown to be equal to or higher than the calculated design value.

$$V_{pr} \geq V_{pd} \qquad (8.20)$$

In punching shear strength calculations, the cross sectional area enclosed by the critical perimeter, located at a distance d/2 away from the loaded area should be considered (see Figure 8.2). The design punching shear force is the algebraic sum of forces perpendicular to slab, Figure 8.2. F_a shown in the figure is the sum of slab loads acting on the area enclosed by the critical perimeter (slab loading for slabs and soil pressure for footings).

The punching shear strength V_{pr} shall be calculated by using the equation given below.

$$V_{pr} = \gamma f_{ctd} u_p d \qquad (8.21)$$

γ in this equation is a coefficient reflecting the bending effect. Unless the effect of unbalanced column moments transferred to the slab has been obtained by carrying out more reliable calculations, the shear due to bending moments should be considered in calculations by using the coefficients γ given below.

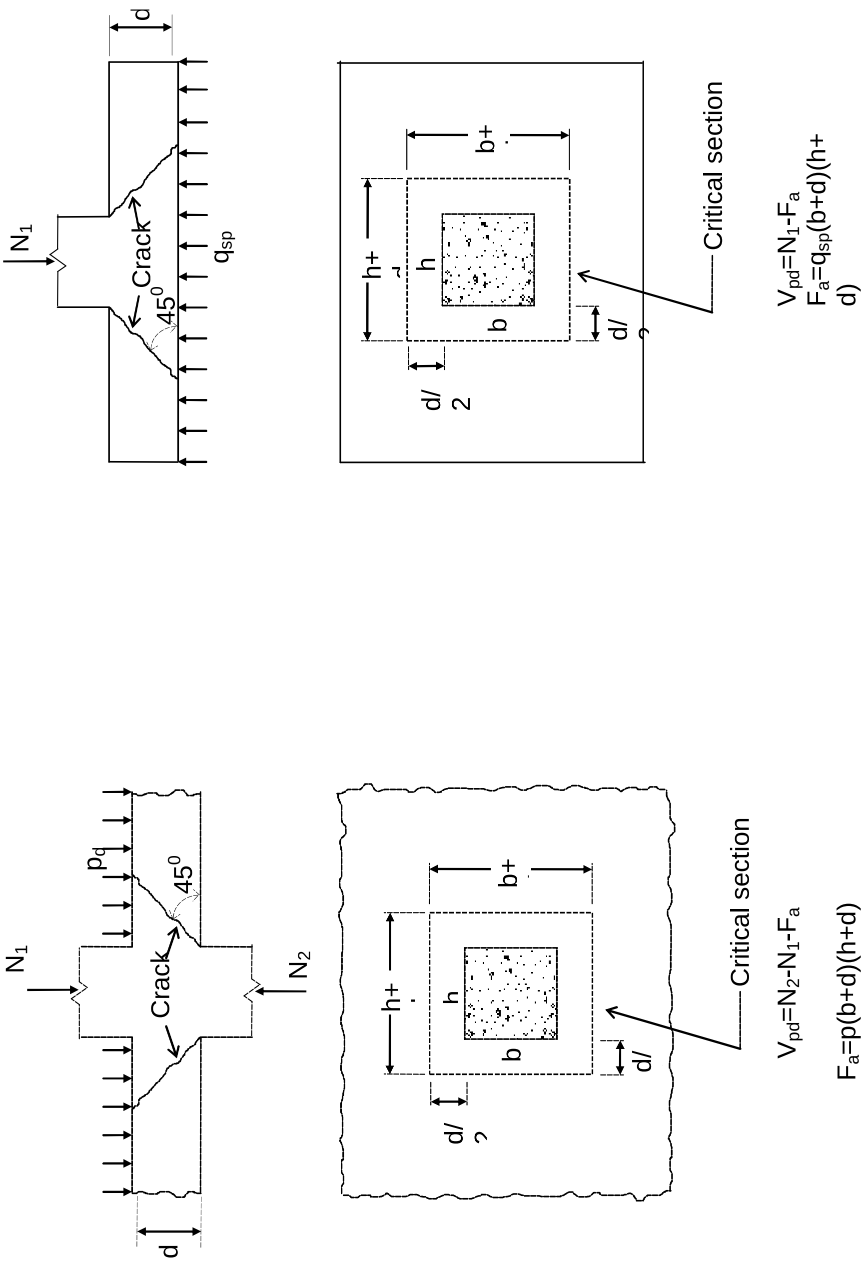


Figure 8.2 Punching area properties and design punching shear

In case of axial loading, $\gamma = 1.0$

In case of eccentric loading, $\gamma = \dfrac{1}{1 + \eta \dfrac{e}{W_m} u_p d}$ *(8.22)*

where $\eta = \dfrac{1}{1 + \sqrt{b_2 / b_1}}$ *(8.23)*

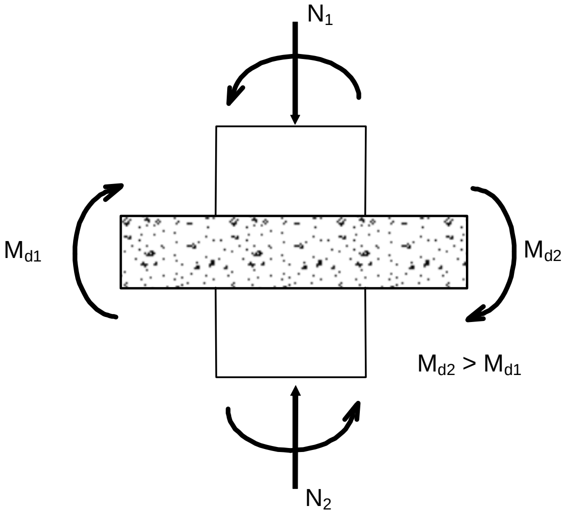
Equation 8.23 is only applicable when $b_2 \geq 0.7 b_1$.

When rectangular or circular loading areas (or columns) are not near edges or corners of the slabs, γ can be expressed by means of simpler equations.

For rectangular loading areas or rectangular columns $\gamma = \dfrac{1}{1 + 1.5\dfrac{e_x + e_y}{\sqrt{b_x b_y}}}$ *(8.24)*

For circular loading areas or circular columns $\gamma = \dfrac{1}{1 + \dfrac{2e}{d + d_0}}$ *(8.25)*

Eccentricities in Equations 8.22, 8.24 and 8.25 should be based on 40 percent of the algebraic sum of the slab moments acting in the direction of bending, together with the difference of axial loads in the columns below and above the plate. The eccentricity calculation is depicted in Figure 8.3.



$$e = \frac{0.4(M_{d1} + M_{d2})}{N_2 - N_1}$$

Figure 8.3 Eccentricity to be used in the calculations

Holes that are located at distances of 5d or less from an edge of the loading area should be considered in critical perimeter calculations. The critical perimeter u_p should be reduced due to these holes by subtracting that portion of the critical perimeter lying within radial lines connecting the center of gravity of the load area to the hole edges as shown in Figure 8.4.a.

When the aspect ratio of the loaded area is greater than 3, the critical perimeter should be calculated assuming h=3b as shown in Figure 8.4b.

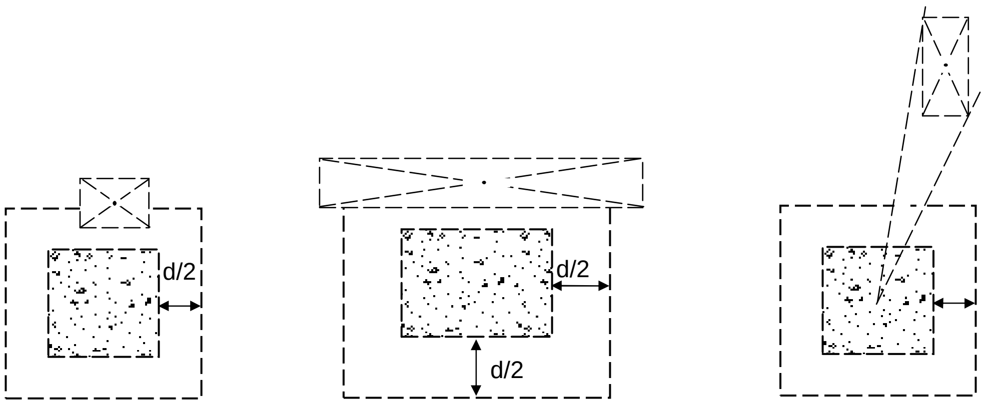
When the loading area has re-entrant corners (a T shape), it is possible to ignore the resulting concavity by making use of suitable tangents as shown in Figure 8.4c and using the resulting circumscribing line as the critical perimeter. When columns or loaded areas are near slab edges, the critical perimeter should be the smaller of the two options shown in Fig.8.5.a.

When columns or loaded areas are near slab corners, the critical perimeter should be the smaller of the two options shown in Fig.8.5b.

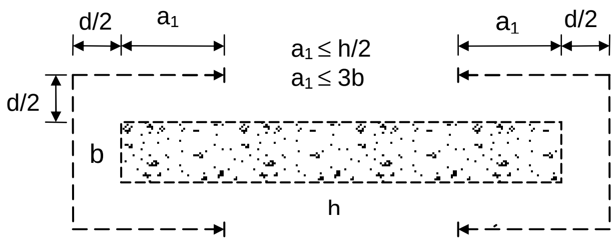
If more than one critical section exists, (e.g. if the column has a drop-panel or column capital) each one of them should be considered separately and the most unfavorable case should be selected from among them as the most critical perimeter.

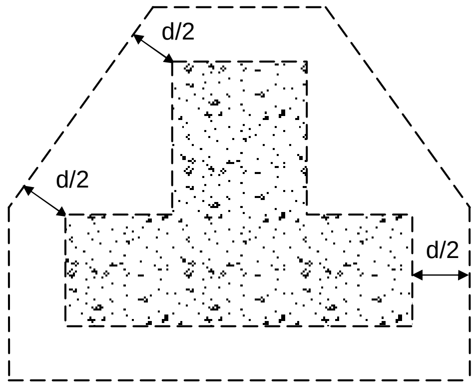
8.3.2 - Punching Shear Reinforcement

The punching shear strength obtained from Equation 8.21 can be increased by using suitable reinforcement or arrangements of steel sections or special steel elements if such increase has been validated by experiments. However for punching shear reinforcement to be effective, the slab thickness must be at least 250 mm. Also the punching shear strength augmented in this manner cannot exceed 1.5 times the value obtained by using Equation 8.21.



a. Openings in slabs

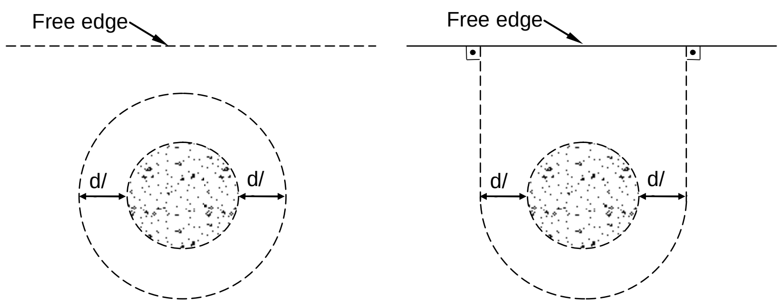




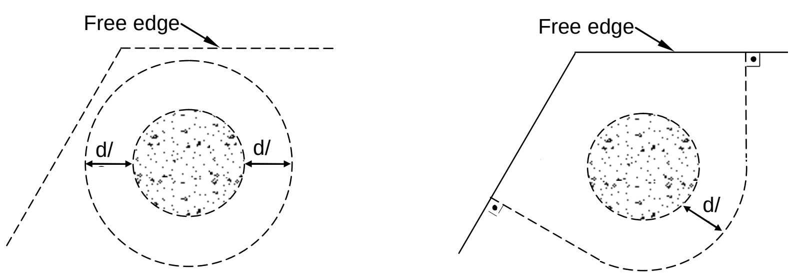
b. Rectangular section with large aspect ratio

c. Column with concave section

Figure 8.4 Critical section for special conditions



(a) Two options for edge columns



(b) Two options for corner columns

Figure 8.5 Critical perimeters for edge and corner columns

8.4 - BRACKETS AND SHORT CANTILEVERS

The ultimate capacity and reinforcement for brackets and short cantilevers with shear span to depth ratio (ratio of the distance between the support and the loaded point to the effective section depth) equal to or less than unity ($a_v \leq d$) should be calculated in accordance with the principles of this Section.

When special measures are not taken, the elongations and contractions produced by temperature changes and shrinkage results in the development of horizontal forces in brackets and short cantilevers. For this horizontal force the load factor should be taken as 1.6. This force is always tensile and cannot be taken as less than 0.2 V_d. The design shear strength of brackets and short cantilevers should not exceed the value obtained by using Equation 8.26. A friction-shear calculation should be carried out for brackets and short cantilevers and the required friction shear reinforcement (A_{wf}) should be calculated according to Clause 8.1.7.

$$V_d \leq 0.22 f_{cd} b_w d \tag{8.26}$$

The total tension reinforcement area (A_{st}), is the sum of the cross sectional areas of reinforcing bars resisting both bending and the horizontal force (H_d), Figure 8.6.

$$\begin{aligned} A_{st} &= (A_s + A_n) \geq \left(\frac{2}{3} A_{wf} + A_n\right) \\ &\geq 0.05 \frac{f_{cd}}{f_{yd}} b_w d \end{aligned} \tag{8.27}$$

$$A_s = \frac{V_d a_v + H_d (h-d)}{0.8 f_{yd} d} \tag{8.28}$$

$$A_n = \frac{H_d}{f_{yd}} \tag{8.29}$$

The cross sectional area A_{sv} of closed and open transverse reinforcement arranged in the region between the top face and 2/3rd of the effective depth "d" of the section in case of brackets and short cantilevers cannot be less than the value given below.

$$A_{sv} \geq 0.5 (A_{st} - A_n) \tag{8.30}$$

Tension reinforcement should have sufficient anchorage to enable yielding to occur. To achieve this, the primary tension reinforcement should either be suitably welded to an anchoring bar of diameter not less than that of the smallest tensile reinforcing bar (Figure 8.6) or the tensile reinforcement must comprise U shaped hairpin bars. The anchor bar or the bottom part of the hairpin should project beyond the loaded area.

8.5 - SPECIAL PROVISIONS FOR DEEP BEAMS

The provisions and limits of this section apply to the shear design of beams with clear span (between support and loaded tip) to depth ratio less than 5. Members that are loaded and supported on the same face should be designed according to Section 8.1. The design shear force shall be calculated at a distance of 0.15 ℓ_n from the support for uniformly loaded beams and at a distance of 0.5a from the support for beams with concentrated loads but in no case should this distance exceed the beam effective depth. Here, "a" is the distance of the concentrated load from the support face.

The contribution of concrete to shear strength should be calculated using Equations 8.4 and 8.1.

The design shear force calculated as described above cannot exceed the limits given below. If these requirements are not satisfied, the dimensions of the section should be increased.

If $(\ell_n / d) < 2$; then $V_d \leq 0.20 f_{cd} b_w d$ (8.31)

If $2 \leq (\ell_n / d) \leq 5$; then $V_d \leq 0.017 f_{cd} b_w d (10 + \ell_n / d)$ (8.32)

When the design shear force exceeds the design cracking strength obtained by using Equation 8.1, the amount of shear reinforcement to be placed perpendicular and parallel to the axis of the beam should be calculated by using the Equation 8.33. The concrete contribution V_c should be calculated by using Equation 8.4.

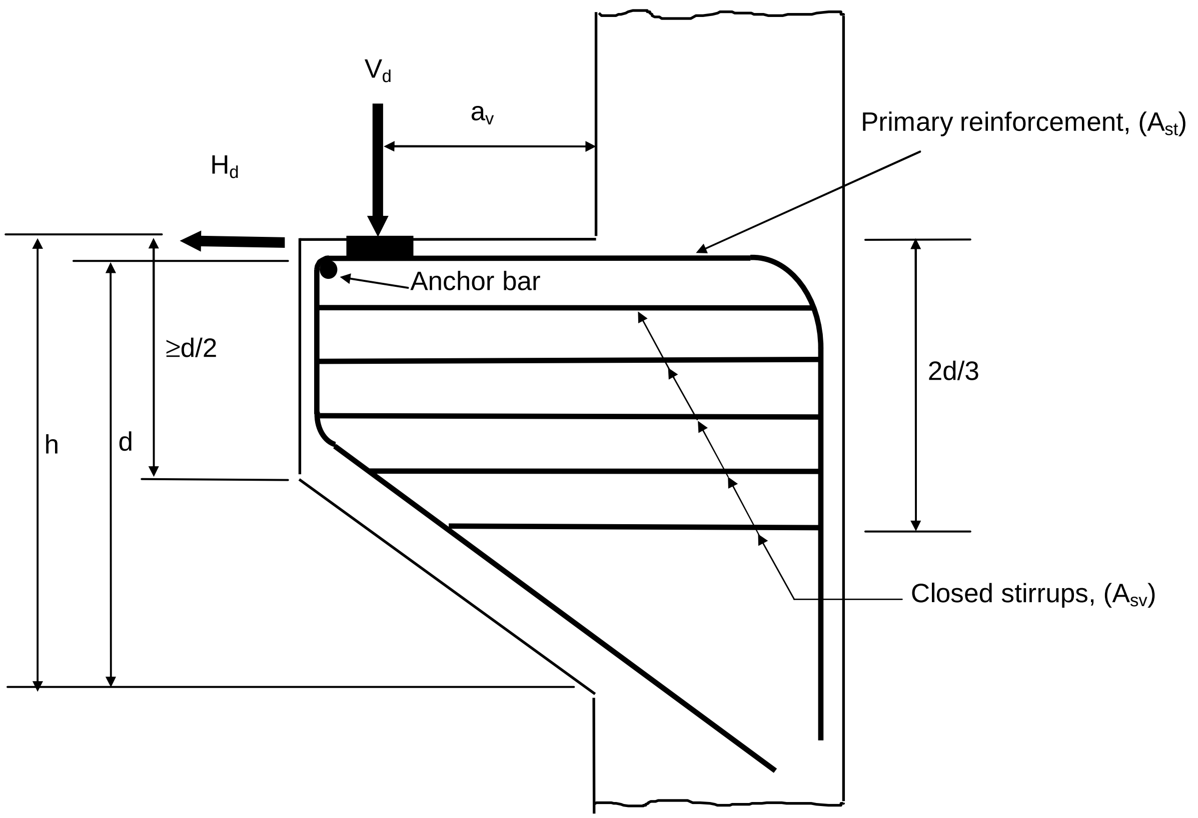


Figure 8.6 Notation for short cantilever

$$V_w = \frac{d}{12}\left[\left(1+\frac{\ell_n}{d}\right)\frac{A_v f_{ywd}}{s} + \frac{A_{vh} f_{yd}}{s_h}\left(11-\frac{\ell_n}{d}\right)\right] \tag{8.33}$$

A_v in this equation is the area of transverse shear reinforcement placed perpendicular to the beam axis with spacing "s". A_{vh} is the area of longitudinal shear reinforcement placed parallel to the beam axis with spacing "s_h" over the beam depth.

The calculated shear reinforcement areas shall not be less than the values given below.

$$\frac{A_v}{s} \geq 0.8\frac{f_{ctd}}{f_{ywd}} b_w \tag{8.34}$$

$$\frac{A_{sh}}{s_h} \geq 0.8\frac{f_{ctd}}{f_{yd}} b_w \tag{8.35}$$

The spacing of reinforcement being placed perpendicular and parallel to the beam axis shall exceed neither d/5 nor 400 mm.

9 - RULES / SPECIFICATIONS FOR ANCHORAGE AND PLACEMENT OF REINFORCEMENT

9.0 - NOTATION

a	Spacing of hairpin bars
c	Concrete cover measured from center of gravity of outermost reinforcing bar
c_c	Clear concrete cover
f_{ctd}	Design tensile strength of concrete
f_{yd}	Design yield strength of longitudinal reinforcement
d_m	Diameter of roller around which the bar is bent
f_{yk}	Characteristic yield strength of longitudinal reinforcement
ℓ_b	Development length
ℓ_{bk}	Development length with hooked end
ℓ_o	Lap splice length
n	Number of reinforcing bars in a bundle
	Number of transverse reinforcing bars in mesh reinforcement
r	Ratio of spliced reinforcement to total reinforcement at a given section
α_1	Lap splice length coefficient
ϕ	Diameter of longitudinal reinforcement (if different bar sizes used, the largest bar diameter)
ϕ_e	Equivalent diameter in case of bundled bars

9.1 - DEVELOPMENT OF REINFORCEMENT

9.1.2 - General

For a reinforced concrete structural member to behave as required, it is necessary that the reinforcement be fully anchored into concrete. The development length required to achieve complete bond depends on the position of the reinforcing bars during concrete casting.

CASE I: General situation (All bars not in Case II)

CASE II: Reinforcing bars making an angle of 45^o-90^o with the horizontal during casting as well as reinforcing bars in the lower half of the section or at least 300 mm away from the upper face of the section which are horizontal or which make an angle less than 45^o with the horizontal during casting.

For a reinforced concrete member at any section to be able to resist the required tensile or compressive stresses safely, the reinforcement should have adequate development length in both directions. Anchorage can be achieved by providing development length, sleeves or other similar mechanical devices or by providing hooks.

9.1.3 - Development of Reinforcing Bars in Tension

a) Straight Embedment

Anchorage can be achieved by extending the reinforcing bars a distance of ℓ_b beyond the point where the stress in the bar is maximum. This length is defined as the development length, and for deformed bars the development length shall be calculated using Equation 9.1.

$$\ell_b = \left(0.12\frac{f_{yd}}{f_{ctd}}\phi\right) \geq 20\phi \qquad (9.1)$$

In case of plain bars, twice this value should be taken as the development length.

When the diameter ϕ of the reinforcement is 32 mm < $\phi \leq$ 40 mm, the development length value obtained from equation 9.1 should be increased by multiplying it by 100/(132-ϕ).

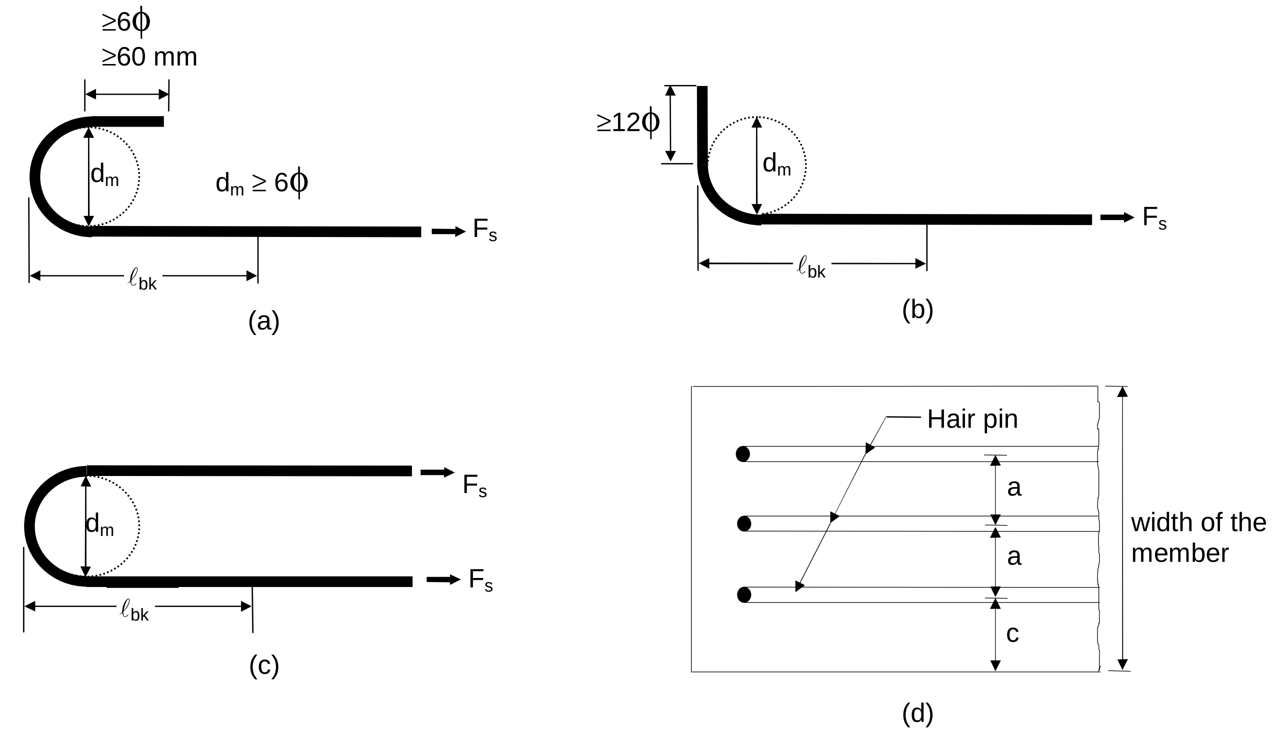
Development length values, obtained from equation 9.1, should be multiplied by 1.4 when bars conform to Case 1.

When the amount of reinforcement provided in a section is greater than the required amount, the development length calculated using Equation 9.1 can be reduced by multiplying it with the ratio of the amount of required reinforcement to the amount of available reinforcement at the section. However the development length reduced by this method cannot be less either than half of the value calculated using Equation 9.1 or 20ϕ. In case of earthquake resistant high ductility frame members and for the critical heights in shear walls as described in “Specifications for Structures to be Built in Disaster Areas”, the development length shall not be reduced. When the concrete cover is less than the diameter of the reinforcing bars or the clear spacing between reinforcing bars in a layer is smaller than one and a half times the diameter of the reinforcing bars, the development lengths calculated by using Equation 9.1 should be increased by multiplying them by 1.2.

Straight embedment is only allowed in case of deformed reinforcing bars.

b) Development with Hooks or Hairpin Reinforcement

The development length can be reduced for reinforcing bars which have hooked ends and for hairpins. If the standard hooks shown in Figure 9.1 are used, the development length can be reduced to ¾ of the value calculated from Equation 9.1.



Longitudinal bar, tie and cross-tie hooks: $d_m \geq 6\phi$

Hair pins: $d_m \geq 12\phi$, $a \geq 2\phi$, $c \geq 3\phi$

Figure 9.1 Hooked bar details

c) Development with Welded Transverse Bars

The required development length can be achieved by welding transverse bars to the reinforcement. This type of development is used widely in case of spot-welded mesh reinforcement.

The number of transverse bars and their minimum dimensions, which are required for adequate development, are given in Table 9.1 for spot welded mesh reinforcement. In case of dynamic or frequently changing loading, the number of bars should be increased by one and 100mm should be added to the length given in Table 9.1.

Table 9.1 Anchorage Requirements for Welded Transverse Bars

Bar Type	ϕ (mm)	Case I		Case II	
		n	ℓ_b (mm)	n	ℓ_b (mm)
Plain	$\phi < 8.5$	3	450	3	350
	$\phi \geq 8.5$	4	500	4	400
Deformed	$\phi < 8.5$	3	350	3	300
	$\phi \geq 8.5$	3	450	3	350

d) Mechanical Anchorage

Under special conditions, reinforcement can develop the required anchorage strength by the welding or bolting of steel plates to the end of the reinforcing bar. Under such circumstances, the envisaged mechanical anchorage should be tested in a laboratory and the design reinforcement force should not exceed 70 percent of the ultimate fracture load value obtained from the tests. Before using such anchorage, special permission needs to be obtained.

Mechanical anchorage can be achieved by using special bolts. Bolts to be used should be proved to be adequate with tests.

e) Development of Bundled Bars

In calculating the development length for bundled bars, the equivalent reinforcement diameter ϕ_e as defined in Equation 9.3 should be used.

9.1.4 - Development of Reinforcing Bars in Compression

No hooks are permitted at the ends of reinforcing bars in compression. If the reinforcing bar is in compression under all load combinations, the development length obtained from Equation 9.1, can be reduced to up to 75 percent of the calculated length.

9.1.5 - Development of Stirrups

Hooks, straight overlaps or welded transverse bars can be used to provide development length in case of stirrups.

a) Development with Hooks

Such development should be with 90^o or 135^o hooks as shown in Figure 9.2. The hook type shown in Fig.9.2a can be used in rectangular sections, columns and members subjected to torsion. The hook type shown in Fig.9.2b can only be used in ribbed slabs.

b) Development with Straight Laps

Straight overlaps such as shown in Figure 9.2c cannot be used in structural members which may be subjected to earthquake or torsion effects.

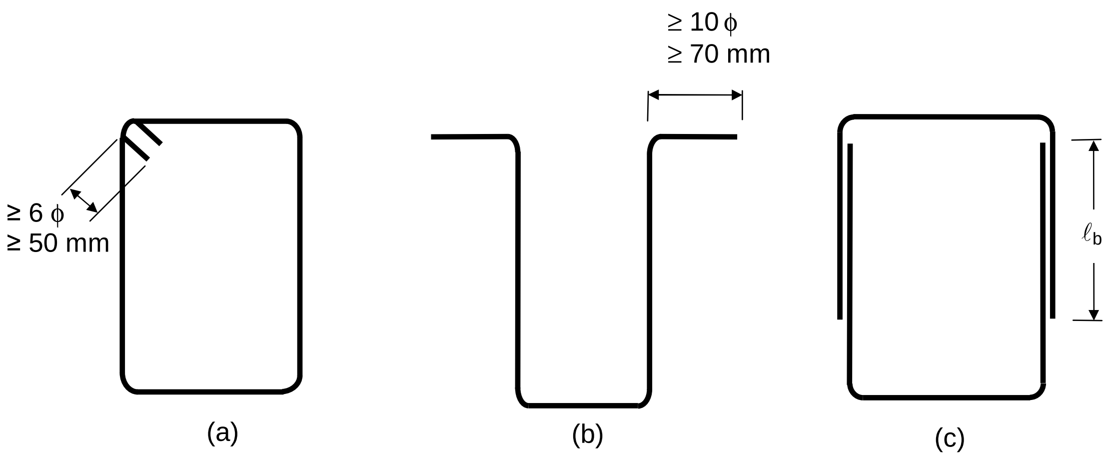


Figure 9.2 Different anchorage types for stirrups

c) Development with Welded Transverse Bars

For stirrups, this type of development can only be achieved with mesh reinforcement. Different anchorage techniques are shown in Figure 9.3.

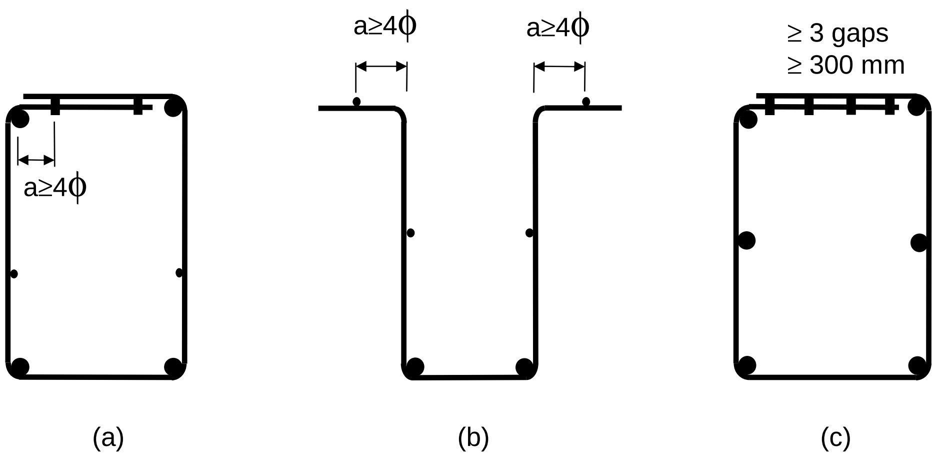


Figure 9.3 Welded splice types for stirrups

9.2 - SPLICING OF REINFORCEMENT

9.2.1 - General

Design drawings should illustrate locations of reinforcement splices and the manner in which they should be produced. Any alterations to be made must be approved by the design engineer.

The following procedures for splicing the reinforcing bars are permitted.

9.2.2 - Lap Splices

In lap splices it is preferable to have the spliced bars in contact. If it is necessary to leave a gap between the two spliced bars, this gap should not exceed neither 1/6 of the required lap splice length, nor 100 mm.

When bundled reinforcing bars are spliced, all the splices should not be located at the same section. The lap splice lengths of bars in a bundle should be obtained by increasing the lap splice lengths required for individual bars within the bundle, (obtained from Clause 9.2.5), by 20 percent.

9.2.3 - Mechanical Connections (Sleeves)

Sleeve connections to be used for splicing in reinforced concrete structural members should be tested both in tension and compression and the bar spliced with the sleeve should be proven to have a strength of 1.25 times the minimum characteristic yield strength envisaged in the standard.

9.2.4 - Welded Splices

The metallurgical analysis of reinforcing bars to be welded should be conducted and the steel carbon content should especially be checked to prove that it is suitable for welding.

Tension tests should be carried out on one out of every fifty welded splices (at least five specimens). These tests should prove that welded splices can reach a strength of 1.25 times the minimum characteristic yield stress f_{yk}.

9.2.5 - Splices of Reinforcing Bars in Tension

a) Lap Splices

The lap splice length ℓ_o in the case of lap spliced bars should be calculated using Equation 9.2.

$$\ell_o = \alpha_1 \ell_b \qquad \alpha_1 = 1 + 0.5r \tag{9.2}$$

Here "r" is the ratio of spliced reinforcement to total reinforcement at that section. For members where the whole section is in tension, α_1 is taken as $\alpha_1 = 1.8$. In case of bars that are classified as Case 1, the length ℓ_o should be magnified by multiplying it with 1.4.

If the ends of bars being lap spliced are hooked, the lap splice length can be reduced to three-quarters of the calculated length.

Confining hoop reinforcement should be provided along the lap splice length. The diameter of hoop reinforcement should be at least 1/3 of the diameter of the reinforcing bar being lap spliced and should not be less than 8 mm. At least six hoops should be present along the lap splice length. The spacing of the confinement reinforcement cannot be more either than ¼ of the member depth or 200 mm.

If more than one bar has to be spliced, splices should be staggered. When the distance between the midpoints of two individual bar splices is at least 1.5 ℓ_o, those splices are considered to be staggered.

Table 9.2, which is recommended for the lap splicing of mesh reinforcement, is only applicable for cases where at the section at least 50 percent more reinforcement has been provided than is required. When the ratio of area of reinforcement provided in the section to the area of reinforcement required is less than 1.5, the lap splice lengths and number of transverse bars given in Table 9.2 should be increased in the proportion (1.5×required reinforcement cross-sectional area ÷ cross-sectional area of reinforcement provided at the section).

Table 9.2 Lap splicing requirements for spot-welded mesh reinforcements

Bar Type	ϕ (mm)	Case I		Case II	
		n	ℓ_o (mm)	n	ℓ_o (mm)
Plain	$\phi < 8.5$	4	500	4	400
	$\phi \geq 8.5$	5	600	5	500
Deformed	$\phi < 8.5$	4	400	4	350
	$\phi \geq 8.5$	4	450	4	400

b) Mechanical Connections

Mechanical connections (sleeves) for splicing of reinforcing bars in tension should be designed according to Clause 9.2.3.

c) Welded Splices

Welded splices of reinforcing bars in tension should be designed according to Clause 9.2.4.

9.2.6 - Splices of Reinforcing Bars in Compression

a) Lap Splices

The lap splice length for bars in compression shall not be less than the development length that is defined in Clause 9.1.2a nor less than 300 mm. No hooks shall be provided at the ends of lap spliced reinforcing bars resisting compression. The spacing of confining hoop reinforcement along the lap splice length, which is defined in Clause 9.2.5a, should not be less than d/4.

Reinforcing bars with diameters larger than 30mm shall not be lap spliced. These reinforcing bars should be spliced with special mechanical connectors (sleeves) whose adequacy has been proven by tests.

b) Mechanical connectors for reinforcing bars in compression should be designed according to Clause 9.2.3.

c) Welded Splices

Welded splices of reinforcing bars in compression should be designed according to Clause 9.2.4.

9.2.7 - Lap Spliced Longitudinal Column Bars

c) If the longitudinal reinforcing bars are being lap spliced in the middle region of the column, the relation $\ell_0 \geq \ell_b$ should be satisfied.

d) If under all load combinations no tension develops in the column longitudinal bars the lap splices can be designed according to Clause 9.2.6.

e) If tension develops under the same load combination in the column longitudinal bars and if such bars are lap spliced at the bottom end of the column, then the conditions given below should be satisfied.

If half or less than half of column longitudinal bars are being lap spliced, $\ell_o \geq 1.25\ \ell_b$.

If more than half of column longitudinal bars are being lap spliced, $\ell_o \geq 1.5\ \ell_b$.

9.3 - STANDARD HOOK DETAILS

Standard hook types defined in this standard are shown in Figure 9.1.

9.3.1 - Hooks in Longitudinal Reinforcement

a) 180^{o} hook - As shown in Figure 9.1a, this type of standard hook makes an angle of 180 degrees with the reinforcing bar axis. At the free end of the hook, there should be a straight portion which is not shorter either than 4 bar diameters or 60 mm. The inner diameter of the hook, d_m, cannot be less than 6 bar diameters.

b) 90^{o} hook - As shown in Figure 9.1b, this type of standard hook makes an angle of 90 degrees with the reinforcing bar axis. At the free end of the hook, there should be a straight portion which is not shorter than 12 bar diameters. The inner diameter of the hook, $d_{m,}$ cannot be less than 6 bar diameters.

c) Hairpin hook - This type of standard hook is shown in Figure 9.1c. The inner diameter, d_m , of hairpin hooks should not be less than 12 bar diameters.

9.3.2 - Stirrup Hooks

a) 135^{o} hook - This type of standard stirrup hook is shown in Figure 9.2a.

b) 90^{o} hook - This type of standard stirrup hook is shown in Figure 9.2b (can not be used in members resisting seismic action).

c) 90^{o} straight lap hook - This type of standard stirrup hook is shown in Figure 9.2c. In case of stirrups with straight laps, lap lengths should be taken to be equal to development lengths calculated by using Equation 9.1. This stirrup type cannot be used in members resisting seismic action.

The inner diameters of stirrup hooks cannot be less than 4 bar diameters.

9.3.3 - Stirrup Hooks made up of Mesh Reinforcement

Stirrup hooks of mesh reinforcement are made as shown in Figures 9.3a, b and c.

9.4 - REGULATIONS FOR BENDING REINFORCING BARS

Longitudinal reinforcement should be bent without being heated around a roller that has a diameter of at least 6 bar diameters. By carrying out bending tests in accordance with TS708 it should be proved that the reinforcing bars under consideration are suitable for bending.

Bent reinforcing bars should not be straightened after concrete has been placed. If straightening is needed unavoidably, it should be carried out by using a roller of at least 6ϕ diameter with the approval of the engineer in charge.

9.5 - REGULATIONS FOR PLACEMENT OF REINFORCING BARS

9.5.1 - Clear Concrete Cover

Values for clear concrete cover required to provide necessary bond to the reinforcement and to protect it from external effects are given in Table 9.3 (measured from the external surface of the outermost reinforcing bar).

When the possibility of fire, rust (corrosion) and other harmful external effects exists, concrete cover should be increased by the amount considered necessary.

Table 9.3 Required Concrete Cover from External Surface of Outermost Reinforcing Bar

In members which are in direct contact with the soil	$c_c \geq 50$ mm
Exterior columns and beams open to the atmosphere	$c_c \geq 25$ mm
Interior columns and beams not open to external effects	$c_c \geq 20$ mm
In shear walls and slabs	$c_c \geq 15$ mm
In shells and folded plates	$c_c \geq 15$ mm

9.5.2 - Spacing of Reinforcing Bars

The clear spacing of reinforcing bars in the same layer should not be less either than the diameter of the reinforcing bar or 4/3 times the nominal maximum coarse aggregate size or less than 25 mm. These limits are also applicable in locations where lap splices exist.

When there is more than one layer of reinforcement, the reinforcing bars in the upper layer should be aligned on top of the reinforcing bars in the lower layer. The clear spacing between the two layers should be at least 25 mm or one bar diameter.

In columns, the clear spacing between two longitudinal reinforcing bars should not be less either than 1.5 times the reinforcing bar diameter or 4/3 times nominal the maximum coarse aggregate size or less than 40 mm.

9.5.3 - Bundled Bars

Only deformed bars can be used as bundled bars (as shown in Figure 9.4). The maximum number of reinforcing bars in a bundle can be three.

The equivalent diameter ϕ_e for bundled bars is given in Equation 9.3. ϕ is the diameter of the reinforcing bars in the bundle and n is number of reinforcing bars in the bundle.

$$\phi_e = 1.2\phi\sqrt{n} \tag{9.3}$$

Regulations given in section 9.1 and 9.2 are also applicable for bundled bars with the provision that ϕ_e be used instead of ϕ.

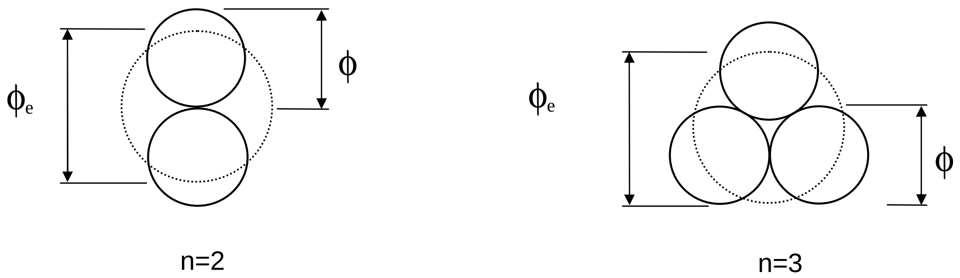


Figure 9.4 Arrangement of bundled bars

9.5.4 - Placement of Reinforcing Bars

Before placing, reinforcing bars should be cleaned and rendered free of dirt, oil and surface rust. Care should be taken to ensure that reinforcement is placed in accordance with the design drawings. Primary reinforcement (tension and compression) should be tied up properly with distribution reinforcement and stirrups.

While casting concrete, precautions should be taken so that reinforcement will not change its position. To achieve the required concrete layer around the reinforcing bars, concrete spacers should be placed between the formwork and reinforcing bars, and small steel rods should be placed as spacers between two layers of reinforcing bars. Instead of concrete spacers and steel rods, plastic blocks manufactured for this purpose may be used. Special care should be taken that stirrups are laterally covered completely by concrete. For slabs and beams, measures should be taken to prevent the top reinforcing bars from being forced downwards.

In case of structural members which have bottom reinforcement and will be placed directly on soil (as in foundation slabs), at least 50 mm of concrete cover or similar insulating layer should be provided between the soil and the structural member, considering the soil conditions.

10 - REINFORCED CONCRETE FOOTINGS

10.0 - NOTATION

b Width of footing supporting the wall
d Average of effective depth
ℓ Projecting length of the footing measured from the face of a wall or column
V_{cr} Cracking strength of section under shear
V_d Design shear force

10.1 - GENERAL

The provisions of this section apply to the design and construction of wall footings, isolated footings and continuous footings, which transfer the superstructure load to the earth. Special footings are out of the scope of this section. The footing level and type should be determined in accordance with the principles of soil mechanics after evaluating local conditions. In calculating the size of the footing that will limit soil deformations under loads and will also ensure footing stability, all the load factors defined in the load combinations in Clause 6.2.6 will be taken as 1.0. In checking the section dimensions and in reinforcement calculations, the base reactions resulting from design loads found using the load factors defined in Clause 6.2.6 will be used.

The concrete cover of footings designed according to be provisions of this section should not be less than 50mm.

10.2 - FOOTINGS SUPPORTING WALLS

10.2.1 - General Principles

Reinforced concrete wall footings should be designed to transfer the loads of structural walls safely to the soil.

Wall footings are proportioned in such a way that no reinforcement is required. However in order to be able to take care of differential settlements, these footings should have the minimum amount of reinforcement specified in Clause 10.2.3.

10.2.2 - Design Principles

The width of footings supporting walls should be determined by considering the permissible soil strength. The shear force to be used in dimensioning the section should be calculated at the wall face, and moment value to be used should be calculated at a distance of 1/4 of the wall thickness from the wall face.

The footing thickness should be chosen such that the design moment is less than the cracking moment calculated assuming a homogenous uncracked section. Also the design shear force should be less than the cracking shear strength.

The width of the footing supporting a wall should exceed that of the wall by at least 100mm on each side. The thickness of the footing supporting a wall should not be less either than half the projecting width of the footing beyond the wall or 200mm.

10.2.3 - Regulations relating to Reinforcement

Along the length of the wall at least four $\phi 10$ longitudinal reinforcing bars should be placed in the footing so that one reinforcing bar is placed at each corner of the section. These longitudinal reinforcing bars should be confined by stirrups of at least 8 mm diameter that have a spacing of not more than 300mm (Figure 10.1).

10.3 - INDIVIDUAL COLUMN FOOTINGS (TWO-WAY FOOTINGS)

10.3.1 - General Principles

When more detailed calculations are not needed, the footings can be assumed to be rigid and the soil pressure distribution underneath these footings can be taken as linear.

Column footings should be connected to each other in both directions with connecting beams or slabs. Such connecting beams should be constructed in accordance with the “Specifications for Structures to be Built in Disaster Areas”.

10.3.2 - Design Principles

Column footing base area dimensions are calculated on the basis of soil strength and settlement. For the calculation of the footing thickness, the soil pressure distribution obtained in accordance with the principles described above should be considered. For the dimensioning and provision of reinforcement, separate calculations should be carried out for bending, shear force and punching shear, and the reinforcement should be proven to have adequate development length.

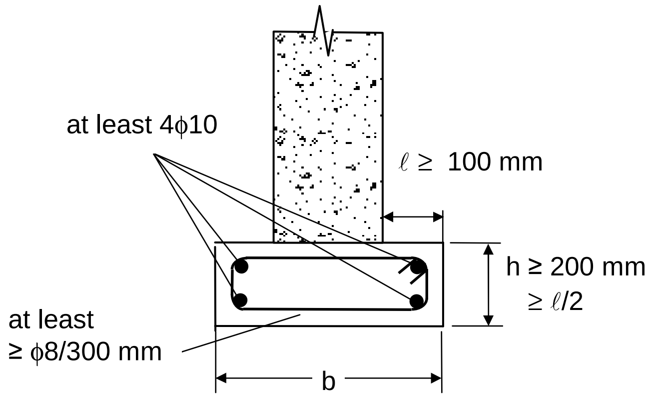


Figure 10.1 Wall footing

If more precise calculations are not required, the parts of the footing extending from the column faces can be designed as cantilever beams. The critical section for moment and shear can be assumed to be at the column face and the punching perimeter should be assumed to be d/2 away from the column face.

The smallest dimension of an isolated footing should not be less than 0.7 meters, its area should not be less than 1 m^2 and its thickness should neither be less than 250 mm nor less than ¼ of the cantilever span length.

10.3.3 - Regulations Relating to Reinforcement

The required reinforcement calculated in both directions should be placed at the base of the footing so as to produce a grid.

The reinforcing bars should be placed with equal spacing.

The ratio of tension reinforcement in each direction cannot be less than 0.002 of the associated cross-section and the spacing of reinforcement cannot exceed 250mm.

10.4 - CONTINUOUS FOOTINGS

10.4.1 - General Principles

Footings supporting several vertical load bearing members such as more than one column and/or structural walls and which are capable of transferring these loads to the soil with adequate stiffness are defined as continuous footings. Continuous footings under vertical load carrying members lined up in one direction are called strip footings, and continuous footings supporting vertical load carrying members placed in more than one direction are called raft footings. Strip footings and raft footings can be designed as plates with beams or as flat plates.

10.4.2 - Design Principles

For the evaluation of soil pressure under the footing due to the design loads, the interaction relations between the structure, footing and the semi-elastic (or inelastic) soil medium need to be considered. Satisfying compatibility between the footing deformations and soil surface deformations without considering special stiffness distributions in the structure above is generally found to be adequate.

For this purpose, the soil can be represented by a semi-elastic medium or more simply by a suitable number of individual springs with adequate stiffnesses. When the ratio between the soil and footing stiffness values is greater than certain known limit values, the soil pressure may be assumed to have a linear distribution as in Clause 10.3.1.

In case of continuous footings with beams, the beam depth including the plate (slab) should not be less than 1/10 of the clear span length and the plate (slab) thickness should not be less than 200mm. For footings of this type, the shear cracking strength V_{cr} of the beam section as defined in Clause 8.1.3 should not be less than the design force V_d calculated at the column face. If this cannot be achieved, the difference between these two values should be kept as small as possible.

In the case of continuous footings arranged as flat plates without beams, the plate (slab) thickness cannot be less than 300mm. For such combined footings, checks should be made for the shear at the column face and the punching shear in accordance with Clause 8.1.4 and Clause 8.3.1. The contribution of reinforcement should not be considered in case of punching shear.

10.4.3 - Regulations Relating to Reinforcement

The minimum longitudinal and transverse reinforcement ratios in all members of combined footings should be in accordance with corresponding ratios envisaged in this standard for beams and plates (slabs). In the compression regions of all sections under bending, compression reinforcement equal to at least 1/3 of the amount of tension reinforcement must be provided.

In the case of deep beams and thick slabs where concrete has to be cast in more than one layer, vertical reinforcement shall be placed at the resulting horizontal joints so that the required friction shear strength can be obtained and the design shear forces can be resisted.

11 - REINFORCED CONCRETE SLAB SYSTEMS

11.0 - NOTATION

a	Support width
b_w	Width of the rib in a joist slab
d	Effective depth
E_{cb}	Modulus of elasticity of concrete in beams
E_{cs}	Modulus of elasticity of concrete in slabs
e	Clear distance between two successive ribs in joist floors
h	Slab thickness
I_b	Gross moment of inertia of beam
I_s	Gross moment of inertia of slab
ℓ	Span length used in calculations
ℓ_s	Span length between support axes in short direction of slab
ℓ_{sn}	Clear span in the short direction of the slab
ℓ_1	Span length of slab between support axes in the direction under consideration
ℓ_n	Clear span of slab in the direction under consideration
ℓ_l	Span length between support axes in the long direction of the slab
ℓ_2	Span length of slab between support axes in the direction perpendicular to the direction under consideration
m	Ratio of long side to short side of the slab (Aspect ratio) m = ℓ_ℓ/ℓ_s
m_d	Design bending moment per unit width of slab
M_d	Design bending moment
M_o	Total static moment
ΔM	Amount of support moment reduction
p_d	Uniformly distributed slab design load
t	Plate (slab) thickness of joist slab
t_o	Thickness of drop panel (thickened slab region) in flat slabs
p_g	Uniformly distributed slab dead load
p_q	Uniformly distributed slab live load
V	Shear force at the center of the support
V_{cr}	Cracking strength of section under shear
V_d	Design shear force
α	Slab moment coefficient
α_s	In two-way slabs, the ratio of sum of continuous edges to the perimeter of the slab
β	Ratio of beam bending stiffness to bending stiffness of slab having a width of ℓ_2, $\beta = E_{cb}I_b / E_{cs}I_s$

11.1 - GENERAL

Provisions of this section apply to the design of one-way and two way slab systems with one or more panels, with or without beams, and for the design of one-way and two way joist slab systems.

11.2 - ONE WAY SOLID SLABS

11.2.1 - General Principles

Reinforced concrete solid slabs under uniformly distributed loading with an aspect ratio (long side to short side ratio) larger than two (ℓ_ℓ / ℓ_s > 2) are classified as one-way slabs.

In case of one way solid slabs, flexure reinforcement should only be provided in the short direction and distribution reinforcement should be provided in the long direction. Reinforcement in one-way slabs should satisfy the requirements given in Clause 11.2.3.

Span and support reinforcements of one-way solid slabs at the edge supports should be anchored into beams, columns or reinforced concrete walls in accordance with the requirements of Section 9. Either straight or hooked anchorages can be used, but the development length from the support face cannot be less than 150 mm.

11.2.2 - Design Principles

In one way continuous flat slabs, the moments due to distributed loads are calculated by using continuous beam theory that assumes beams to be capable of undergoing rotation freely at the supports.

When the solid one-way slab is supported by beams, the design support moment is calculated by subtracting $\Delta M = Va/3$ from the moment value at the center of the support. Here, V is the shear force at the support for the span under consideration and a is the width of the support. The support width cannot be greater than 0.175 of the span length and the reduced moment cannot be less than the value $p_d \ell^2/14$. In case of solid one-way slabs resting freely on supports, the value of the support moments should not be reduced.

In the case of continuous flat slabs spanning between reinforced concrete beams, the negative span moments, which may develop due to live loads, can be reduced by taking the torsional stiffness of these beams into consideration.

If the span positive moment of the of continuous slab is less than the positive moment obtained by assuming both ends of the slabs to be fixed, then the value for the fixed ended case should be used in designing the section.

At the edge supports of continuous solid slabs at least half of the span reinforcement must be provided on the top.

In case of continuous solid slabs under uniformly distributed loading, if the ratio of any two adjacent spans is not less than 0.8 and the ratio of live to dead load is less than two, the moments can be calculated approximately by using the coefficients given below.

Span moments

Exterior spans $M_d = p_d \ell^2 / 11$

Interior spans $M_d = p_d \ell^2 / 15$

Support moments

Slabs with 2 spans only

Exterior supports $M_d = - p_d \ell^2 / 24$

Interior supports $M_d = - p_d \ell^2 / 8$

Slabs with more than 2 spans

Exterior supports $M_d = - p_d \ell^2 / 24$

Interior supports of exterior $M_d = - p_d \ell^2 / 9$

Other interior supports $M_d = - p_d \ell^2 / 10$

In the above equations ℓ is the span length under consideration. For a slab resting freely on walls, the span length, ℓ, can be obtained by adding the slab depth to the clear span. However, ℓ can never exceed the distance between the support axes and can never be less than 1.05 times the corresponding clear span length. When calculating the support moments, the average of the span lengths and loads in adjacent spans should be used.

The smallest permissible thickness for one-way solid slabs is 80 mm. However in the case of solid slabs to be used as ceilings or as a cover for some area or which would be trod upon only for cleaning or similar purposes, the slab thickness may be reduced up to 60 mm. For slabs over which vehicles might pass, the thickness cannot be less than 120 mm.

In addition to the requirements above, the ratio of slab thickness to the clear span length cannot be less than the values given below:

- For simply supported, single span slabs 1/25
- For continuous slabs 1/30
- For cantilever slabs 1/12

For one-way slabs, the clear concrete cover protecting the reinforcement should be at least 15 mm.

11.2.3 - Regulations Related to Reinforcement

In case of one way solid slabs, the ratio of flexural reinforcement cannot be less than 0.003 for steel grade S220. For steel grades S420 and S500, the ratio of flexural reinforcement cannot be less than 0.002. The spacing of principal reinforcement cannot exceed 1.5 times of the slab thickness or 200mm. In case of single span slabs, at least half of the bottom span reinforcement and in the case of continuous slabs at least 1/3 of the bottom span reinforcement should be continuous from support to support.

In addition to the principal reinforcement that is provided in the short direction, distribution reinforcement should be placed perpendicular to the main reinforcement at the bottom face of the slab in the long direction. The amount of distribution reinforcement should not be less than 1/5 of the main reinforcement. The spacing of distribution reinforcement should not exceed 300mm. On the top of the beams, along the short direction of the slab, support reinforcement perpendicular to the principal reinforcement of the slab must be provided. This support reinforcement, which cannot be less than 60 percent of the principal reinforcement, will be placed on top and extended from both sides to a length of 1/4 of the short span length. In addition, for S220 grade steel, at least ϕ8/200 mm, for S420 at least ϕ8/300 mm and for S500 at least ϕ5/150mm should be used as distribution reinforcement.

11.3 - ONE WAY JOIST SLABS

11.3.1 - General Principles

Slabs made up of a thin deck and beams with clear spacing between each other not exceeding 700 mm, are defined as joist slabs. In case of such slabs, it is possible to leave the spaces between the joist vacant as well as to fill them with non-load-carrying filler material. As filling material, light materials such as hollow concrete briquettes or hollow clay tile can be used (Figure 11.1).

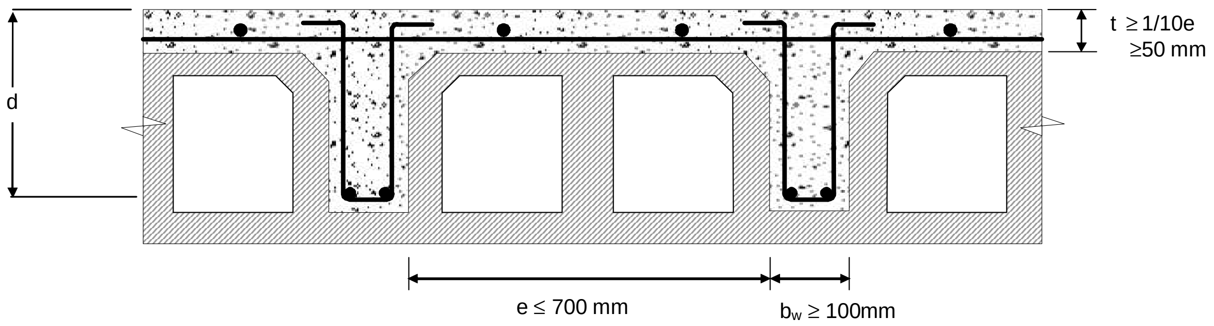


Figure 11.1 Joist floor

When the joists and deck are cast monolithically, these two members act together, and it is thus possible to dimension the span sections of joist slabs as flanged sections. The distance between two beam axes should be taken as the effective flange width. Line loads (due to partition walls etc.) placed perpendicular to the direction of the joists should be taken as point loads acting on the joists. When the magnitude of such line loading is high, a transverse rib should be constructed below it.

11.3.2 - Design Principles

In the case of one way continuous joist slabs, the design moments due to distributed loads are calculated by using continuous beam theory that assumes beams to be capable of undergoing rotation freely at the supports.

The design support moment is found by reducing the moment at the centre of the support by ΔM (Clause 11.2.2). The support width "a" cannot be more than twice the total slab depth. The joist slab section should be dimensioned without taking compression reinforcement into account..

At the end span of joist floors, if the torsional stiffness of the beam providing the exterior support of the joist is ignored, the minimum amount of reinforcement required to resist torsion as defined in Clause 8.2.5 should be provided in the support beam. At the exterior supports of the joists, top reinforcement having an area of at least one half of the span reinforcement should be provided.

In case of one way continuous joist slabs under uniformly distributed loading if the ratio of any two adjacent span lengths is not less than 0.8 and if the ratio of live to dead load is less than 2, the moments can be calculated approximately by using equations given in Clause 11.2.2.

For joist slabs, the maximum design shear force V_d should not exceed the shear cracking strength V_{cr} of the section. If this condition is not satisfied, all requirements of Chapter 8 dealing with the shear design of beams must be fulfilled.

For one-way joist slabs, the clear spacing between joists cannot exceed 700 mm. The deck depth should neither be less than 1/10 of the clear joist spacing nor less than 50 mm, and the joist width should not be less than 100 mm. The ratio of total joist depth (including the deck) to the clear span should not be less than

1/20 in the case of simply supported single span slabs, 1/25 in the case of continuous slabs and 1/10 in the case of cantilevers.

If the span of joists is more than 4 meters, transverse joists (distribution ribs) of at least the same size as the longitudinal ones should be provided. When the span is between 4 and 7 meters, one transverse joist and when the spacing is more than 7 meters, two transverse joists should be provided. These transverse joists should divide the span as equally as possible.

11.3.3 - Regulations Relating to Reinforcement

Flexure and shear reinforcement for joists should be calculated just as in the case of beams. However when the design shear force is less than the shear cracking strength, it is possible not to conform to the minimum stirrup requirement for beams and to use open stirrups. In this case, the stirrup spacing should not exceed 250 mm.

Distribution reinforcement should be provided in both directions in the deck, Figure 11.1. The area of this reinforcement in each direction should not be less than 0.0015 of the total cross sectional area of the slab (deck) and the spacing should not exceed 250 mm.

11.4 - TWO WAY SOLID SLABS

11.4.1 - General Principles

Reinforced concrete solid slabs under uniformly distributed loading, supported along all four edges and having an aspects ratio (long side to short side ratio) equal to or less than two ($\ell_\ell / \ell_s \leq 2$) are called two way solid slabs. However all slabs without beams (Clause 11.4.4) are designed as two way slabs regardless of their support conditions and ℓ_ℓ / ℓ_s ratio.

Two way flat slabs may rest on beams or walls as well as directly on columns (flat plates).

Reinforcement to be provided at critical sections must satisfy the conditions given in Clause 11.4.5.

Support and span reinforcements of flat slabs at the edge supports should be anchored into beams, columns or reinforced concrete walls in accordance with the requirements of Chapter 9. Either straight or hooked anchorages types can be used, but the development length of the anchorage from the support face cannot be less than 150 mm.

11.4.2 - Design Principles

Calculations for two-way solid slabs should be carried out by using a reliable method. Internal design forces which are calculated at critical sections with such methods, should be checked against and found to satisfy the strength and serviceability requirements of Chapters 7, 8 and 13.

The analysis of solid slabs can be carried out by using methods like the “Equivalent Frame Method", and the "Yield line method" etc. For two-way slabs with or without beams, whose span lengths are almost equal to each other or when very precise calculations are not required, the approximate methods given in this section may be used.

For the purpose of reinforcement calculation and placement, slabs without beams are divided into two regions, the column strip and the middle strip. The column strip is a design strip whose width on each side of a column or structural wall centre line is equal to $\ell_1/4$ or $\ell_2/4$, whichever is narrower, Figure11.2a. The column strip, includes the beams, if they exist, in the direction in which the moment is being calculated, Figure 11.2b. The middle strip is a design strip bounded by two column strips.

The depth of a two-way slab with beams cannot be less than the value given by using Equation 11.1.

$$h \geq \frac{\ell_{sn}}{15 + \frac{20}{m}} \left(1 - \frac{\alpha_s}{4}\right) \qquad (11.1)$$

$$h \geq 80mm$$

"α_s" in this equation is the ratio of the sum of the length of the slab continuous edges to the sum of the lengths of all edges.

The depth of a two way slab without beams (flat plate or flat slab) cannot be less than the values given below:

Slabs without beams and drop panels	$h \geq \ell_n/30$ and $h \geq 180$ mm
Slabs with drop panels but without beams	$h \geq \ell_n/35$ and $h \geq 140$ mm

If the design is carried out by using any one of the approximate methods given in Clause 11.4.4, the depth of the slab without beams cannot be less than the value given by using Equation 11.2.

$$h \geq \ell_1/30 \text{ and } h \geq 200 \text{ mm} \quad (11.2)$$

As far as possible, the depth of a flat slab without beams should be chosen so that punching shear reinforcement is not required. In the case of flat slabs without beams, to achieve the moment transfer between columns and slabs, the dimension of the columns in the span direction should not be less than 1/20 of the span length in that direction nor less than 300 mm.

Requirements to be satisfied about drop panel and column capital dimensions are shown in Figure 11.3.

The clear concrete cover protecting reinforcement in two-way slabs should be at least 15 mm.

11.4.3 - Approximate Method for Two-Way Slabs with Beams

In the case of slab systems with beams, if the span lengths are not considerably different from each other or if more precise calculations are not required, the method below can be used.

For flexural design, the theoretical section for the negative moment, the section at the support face, and for the positive moment the section at mid span should be considered.

The slab moment per unit width should be calculated by using α coefficients in accordance with the boundary conditions and the aspect ratio of the long side to the short side. The α coefficients can be obtained from Table 11.1, and by using Equation 11.3. In this equation ℓ_{sn} is the clear span length in the short direction.

$$m_d = \alpha p_d \ell_{sn}^2 \quad (11.3)$$

Moment values obtained by using Equation 11.3 are valid for middle strips as defined in Clause 11.4.2. Moment values for column strips can be obtained by taking 2/3 of these values. The moment value calculated for slabs with very short spans can be assumed to be constant over the slab width.

If the negative moment on one side of the common support of adjacent spans is less than 0.8 times the negative moment on the other side of the common support, 2/3 of the difference of these two values should be distributed to the neighboring slabs in proportion to the slab strip stiffnesses and the larger value should be used in reinforcement calculations. If the difference between the two negative moments is smaller, the greater negative moment value should be used in design. In this method redistribution of support moments cannot be carried out.

The moment at the end support (slab assumed to be resting freely) should be taken as a certain fraction of the span moment value given in Table 11.1, if the rotation of the slab is partially restrained at the end support region by a structural member. If rotation is fully restrained, this ratio should be taken as 1.0, otherwise it should be taken as 0.5.

When slab calculations are carried out with this method, the distribution of the loads to the supports should be according to tributary areas bounded by 45^o lines drawn from the corners.

11.4.4 - Approximate Methods for Two Way Slabs without Beams (Flat Slabs)

Slabs without beams can be flat plates as well as slabs with drop panels or capitals, Figure 11.4a, b and c. If the capital makes a slope of less than 45^o with the horizontal axis, then for calculations the region bounded by column surface and the line making 45^o with the column axis should be considered as the capital, Figure 11.4d.

In case of slabs with drop panels, the depth of the drop panel t_o cannot be less than half of the slab thickness and the length of the drop panel in each direction cannot be less than 1/6 of the slab span in that direction nor less than 4 times the drop panel depth, Figure 11.4a.

The punching shear strength of two way flat slabs without beams should be checked according to the principles given in Clause 8.3.1. The punching shear perimeter for slabs with and without drop panels is shown in Figure 11.4.

Openings may be permitted in flat slab systems without beams provided that strength and serviceability are proven to have been satisfied. If the punching shear strength calculated by considering openings in the slab satisfies the safety requirements envisaged in Clause 8.3.1, there is no need to carry out separate bending checks for the cases given below:

a) In cases where at the intersection region of two perpendicular middle strips all the required reinforcement can be placed.

b) In cases where at the intersection region of two perpendicular column strips, if the largest dimension of the opening is less than 1/8 of the width of the column strip in either direction and if the amount of reinforcement interrupted by the opening can be placed on the sides of the opening.

Calculations for flat slabs without beams can be carried out according to either of the two approximate methods below.

11.4.4.1 - Equivalent Frame Method

Slabs without beams can be analyzed as equivalent frames in two mutually perpendicular directions. In this analysis method, the width of the horizontal bending member of the frame should be taken as the distance between the center lines of spans of two adjacent slabs, which are perpendicular to the frame.

The total slab load should be considered in the analysis in each direction. In calculating the column and slab stiffnesses, the increase in the moment of inertia values due to column capitals and drop panels should be considered.

For gravity load analysis, a single storey can be considered by assuming the ends of columns as fixed.

The moments calculated using the equivalent frame method can be distributed in accordance with Table 11.2 to the column and middle strips, which are defined in Clause 11.4.2. These moments can be modified by up to ±10 percent if the sum of the support and span moments is kept constant for each span.

Table 11.1 Distribution Coefficients for Equivalent Frame Method

Strip		Interior Support Moment	Span Moment	Exterior Support Moment	
				Edge without beam	Edge with beam
Column strip		0.75	0.60	0.80	0.60
Middle strip		0.25	0.40	0.20	0.40
Half column strip parallel to edge beam or wall	Edge without beam	0.40	0.30	0.40	0.30
	Edge with beam	0.20	0.15	0.20	0.15

11.4.4.2 - Moment Coefficient Method

The calculations for slabs without beams satisfying all the conditions given below can be carried out with this approximate method. This method is only applicable for vertical load analysis.

a) There should be a minimum of 3 continuous spans in each direction.

b) The ratio of the longer to the shorter span in a panel should not be greater than 2.

c) Adjacent span lengths in any direction should not differ by more than one-third of the longer span.

d) The offset (eccentricity) of a column from the centerline of the axis of the frame should not be more than 10 percent of the span length in the direction of the frame.

e) The ratio of live load to dead load should not exceed 2.0.

In any slab, the absolute sum of the span moment and the average of the two support moments in each direction of any panel should not be less than

$$M_o = \frac{p_d \ell_2 \ell_n^2}{8} \tag{11.4}$$

ℓ_n in this equation is the clear span length in the direction under consideration and is the face to face distance between columns. ℓ_n can never be taken as less than 0.65 ℓ_1. Columns with circular or regular polygonal cross sections can be treated as equivalent square columns having the same area.

The support moment to be considered in design should be calculated at the support face. Moment re-distribution mentioned in Clause 6.3.8 cannot be applied to flat slabs when calculations are carried out by simplified methods.

The moment M_0, which is obtained from equation 11.4, should be distributed to the span and supports according to the principles given below:

For interior spans

Span moment	$= 0.35M_0$
Support moment	$= 0.65M_0$

For end spans

Exterior support moment	$= 0.30M_0$
Interior support moment	$= 0.70M_0$
Span moment	$= 0.50M_0$

If the exterior support of a slab acts as a fixed support due to the presence of structural members like structural walls oriented perpendicular to the direction of analysis, the support and span moment values corresponding to that particular span should both be taken as $0.65M_0$ and $0.35M_0$ respectively.

If support moments differ on the two sides at a particular support, the larger moment value should be used in the calculations.

The distribution of calculated moments to the column strip should be made as follows:

a) At interior supports, 75 percent of the total support moment calculated above should be given to the column strip.

b) At edge supports, if there is no edge beam perpendicular to the direction in which the analysis is being carried out, the total edge support moment calculated above should be given to the column strip.

c) If there are edge beams, 75 percent of total edge support moment calculated above should be given to the column strip.

d) In spans, 60 percent of the total span moment calculated above should be given to the column strip.

The distribution of the calculated moments to the middle strip should be made as follows:

a) The middle strip moment should be taken as the difference between the total moment and the column strip moment.

b) A maximum modification of ±10 percent can be made in the values of the support and span moments calculated for the column strip and middle strip with the method described above. However, the total statical moment for a flat slab in the direction considered shall not be allowed to differ from the value calculated using Equation 11.4.

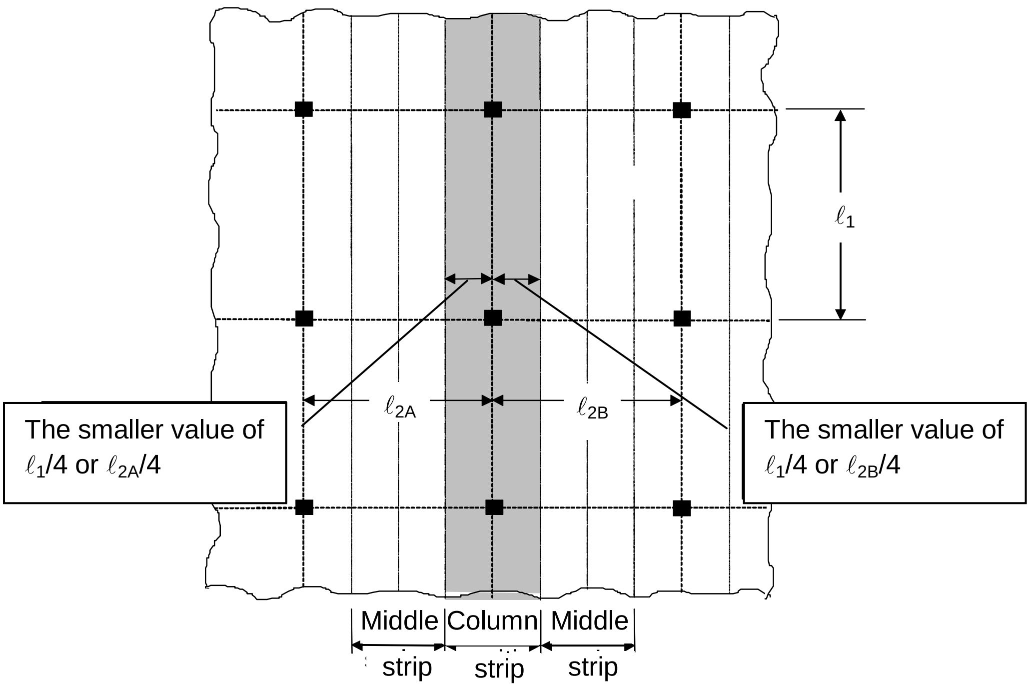
c) In monolithic systems, columns and walls forming supports for a slab should possess enough strength to be capable of resisting moments caused by design loads on the slab system. These moments should be considered in the punching shear calculations given in Clause 8.3.1. When more accurate calculations are not carried out at interior supports, the moment obtained by using Equation 11.5 should be distributed between top and bottom columns at a given support in accordance with their flexural stiffnesses.

$$M = 0.07\left[\left(p_g + 0.5p_q\right)\ell_2 \ell_n^{\,2} - p'_g \ell'_2 \left(\ell'_n\right)^2\right] \tag{11.5}$$

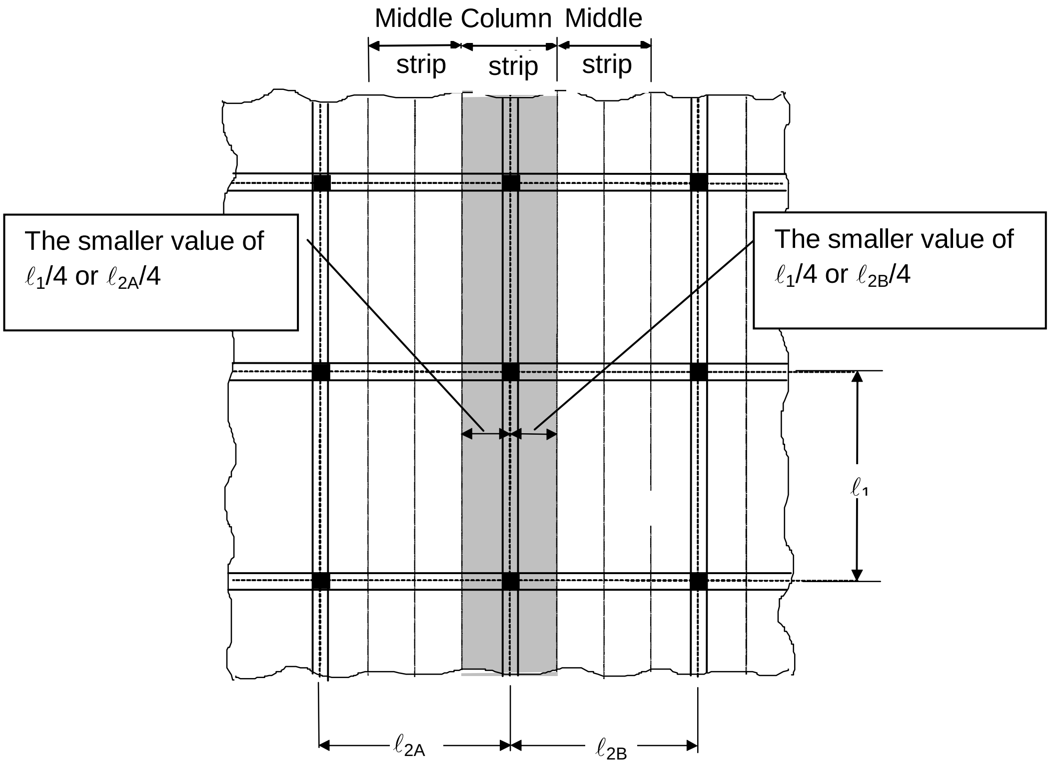
In this equation p'_g, ℓ'_2 and ℓ'_n are values corresponding to the shorter span of the adjacent spans under consideration.

Table 11.2 Moment Coefficients for Two Way Rectangular Slabs (α)

EDGE CONDITIONS OF SLAB	Moment coefficients in short direction								In long direction
	ℓ_ℓ/ℓ_s=1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	(for all ℓ_ℓ/ℓ_s values)
FOUR SIDES CONTINUOUS									
Negative moment at continuous support	0.033	0.040	0.045	0.050	0.054	0.059	0.071	0.083	0.033
Positive moment at midspan	0.025	0.030	0.034	0.038	0.041	0.045	0.053	0.062	0.025
ONE SIDE DISCONTINUOUS									
Negative moment at continuous support	0.042	0.047	0.053	0.057	0.061	0.065	0.075	0.085	0.041
Positive moment at midspan	0.031	0.035	0.040	0.043	0.046	0.049	0.056	0.064	0.031
TWO ADJACENT SIDES DISCONTINUOUS									
Negative moment at continuous support	0.049	0.056	0.062	0.066	0.070	0.073	0.082	0.090	0.049
Positive moment at midspan	0.037	0.042	0.047	0.050	0.053	0.055	0.062	0.068	0.037
TWO SHORT SIDES DISCONTINUOUS									
Negative moment at continuous support	0.056	0.061	0.065	0.069	0.071	0.073	0.077	0.080	-
Positive moment at midspan	0.044	0.046	0.049	0.051	0.053	0.055	0.058	0.060	0.044
TWO LONG SIDES DISCONTINUOUS									
Negative moment at continuous support	-	-	-	-	-	-	-	-	0.056
Positive moment at midspan	0.044	0.053	0.060	0.065	0.068	0.071	0.077	0.080	0.044
THREE SIDES DISCONTINUOUS									
Negative moment at continuous support	0.058	0.065	0.071	0.077	0.081	0.085	0.092	0.098	0.058
Positive moment at midspan	0.044	0.049	0.054	0.058	0.061	0.064	0.069	0.074	0.044
FOUR SIDES DISCONTINUOUS									
Positive moment at midspan	0.050	0.057	0.062	0.067	0.071	0.075	0.081	0.083	0.050



a. Slab without beams



b. Slab with beams

Figure 11.2 Slab Strips

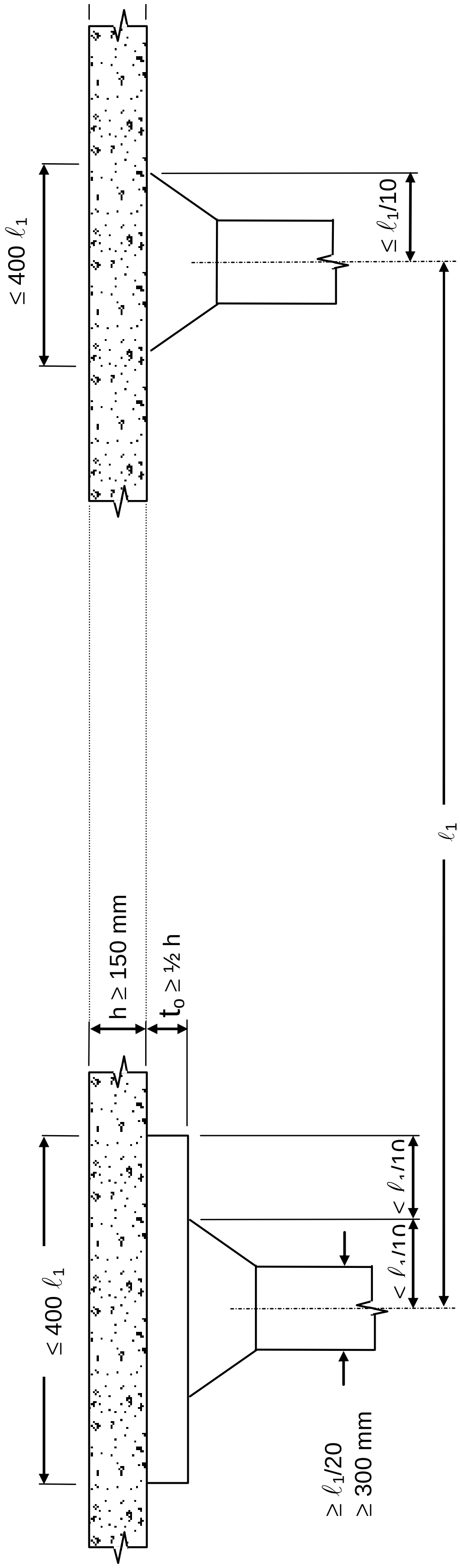


Figure 11.3 Drop Panel and Column Capital Dimensions for Flat Slabs

11.4.5 - Regulations Related to Reinforcement

In the case of two-way slabs, reinforcement to be placed in both directions at critical sections should be based on moments calculated in accordance with the principles of Clauses 11.4.2, 11.4.3 and 11.4.4 and the regulations given in Chapter 7. For two-way slabs with or without beams, the amount of reinforcement in either direction cannot be less than 0.0015 and the sum of the reinforcement in both directions shall not be less than 0.004 for steel grade S220, 0.0035 for S420 and S500. The spacing of reinforcement should not exceed 1.5 times the slab thickness for slabs without a drop panel or 200 mm in the short direction or 250 mm in the long direction.

11.5 - WAFFLE SLABS

Two-way waffle slabs can be designed in accordance with the general principles described in Section 11.d. For such slabs, structural analysis should be conducted in accordance with the principles of structural mechanics and the dimensioning and placement of reinforcement should be according to the requirements given in Clause 11.3.3.

12 - REINFORCED CONCRETE WALLS

12.0 - NOTATION

A_g gross area of reinforced concrete wall
A_{sh} total cross sectional area of transverse (horizontal) reinforcement in reinforced concrete wall

12.1 - GENERAL

Structural walls, walls of special structures such as silos, reservoirs, etc. and retaining walls are examples of reinforced concrete walls whose design and construction require special rules and regulations and, in addition, such walls must also satisfy the requirements given in this section.

Reinforced concrete walls should be dimensioned and reinforced in such a way that they can safely carry all types of loading including forces that would develop due to shrinkage and temperature changes.

The design and dimensioning of reinforced concrete walls should be in accordance with principles, given in Sections 6 and 7. The reinforcement of these walls should satisfy the requirements given in Section 12.3.

If it is necessary to have openings in reinforced concrete walls due to the presence of doors, windows, etc., then in addition to the minimum reinforcement specified in Section 12.3, at least two 16 mm bars should be placed on each side of the opening. The development lengths of these reinforcing bars from the edges of the opening should not be less than 40 bar diameters.

Unless a more detailed analysis has been conducted, the effective length of the wall to be considered for each concentrated load can neither exceed the center-to-center distance between the concentrated loads, nor the bearing length under the load plus four times the wall thickness. To resist tensile stresses in the region below a concentrated load, principal tension reinforcement should be provided perpendicular to the load axis.

12.2 - REGULATIONS RELATED TO SECTION DIMENSIONS

Reinforced concrete walls are vertical load carrying members having the ratio of long side to short side in plan of at least 7. The thickness of reinforced concrete walls cannot be less than 150 mm.

12.3 - REGULATIONS RELATED TO REINFORCEMENT

On each face of reinforced concrete walls, orthogonal reinforcement comprising transverse and longitudinal reinforcing bars should be placed. If calculations do not necessitate the provision of greater reinforcement, the amount of transverse and longitudinal reinforcement to be placed in the reinforced concrete walls should not be less than that given in this section.

The total area of longitudinal reinforcement (sum of both faces) for reinforced concrete walls cannot be less than 0.0015 times the total cross-sectional area (A_g) of the wall. The total area of the transverse reinforcement on both faces of reinforced concrete walls cannot be less than this amount.

The spacing of transverse and longitudinal reinforcement can neither exceed 1.5 times the wall thickness nor 300mm.

The reinforcement on two faces of the reinforced concrete wall should be connected to each other by at least four ties per 1 m^2 of wall.

13 - SERVICEABILITY REQUIREMENTS FOR REINFORCED CONCRETE MEMBERS

13.0 - NOTATION

A_t	Effective concrete area of tension reinforcement in members, $A_t = \dfrac{2ab_w}{n}$
a	Concrete cover measured from the center of gravity of all reinforcing bars in tension
c	Concrete cover measured from the center of gravity of the outermost reinforcing bar
d	Effective depth in case of bending members
E_c	Modulus of elasticity of concrete
f_{ctd}	Design tensile strength of concrete
f_{yd}	Design yield strength of longitudinal reinforcement
h	Total depth of beam
I_c	Gross sectional moment of inertia
I_{cr}	Cracked sectional moment of inertia relative to neutral axis
I_{ef}	Effective moment of inertia
ℓ_n	Clear span of member
M_{cr}	Cracking moment of member under bending
M_{max}	Maximum bending moment for member
n	Number of reinforcing bars in tension zone
y	Distance between extreme tension fiber and neutral axis
δ_i	Instantaneous deflection
δ_{ig}	Instantaneous deflection due to permanent loads
δ_t	Total deflection
ε_{sm}	Average strain in reinforcing bars in the region between cracks
γ_t	Time-dependent factor for permanent load
λ	Permanent deflection multiplier
ρ'	Ratio of compression reinforcement
σ_s	Stress in the reinforcement at cracking, calculated by assuming cracked section
σ_{sr}	Stress in the reinforcement at first cracking, calculated by assuming cracked section
ω	Crack width
ω_{max}	Limit of allowable crack width

13.1 - GENERAL

In addition to the provision of adequate safety against collapse, reinforced concrete structures and members should be designed and reinforced so that under the envisaged service loads there is no excessive cracking, deformation, deflection and vibration in the members and the whole structure.

13.2 - DEFLECTION CONTROL

13.2.1 - General Regulations

In members subjected to flexure, such as slabs and beams, deflections which would impair their function, affect their appearance and cause cracking or crushing in adjacent non-structural connected members should not be allowed to occur. Immediate deflections due to permanent and live loads and deflections due to shrinkage and creep effects for these members should be computed by considering their cracked state.

In the case of beams and especially slabs, which do not support or are not attached to nonstructural members sensitive to deflections and whose depth to span length ratios are within the limits given in Table 13.1, deflection calculations need not be carried out.

Table 13.1 Depth/span ratios for flexural members for which deflection calculations are not necessary

Member	Simple support	Exterior span	Interior Span	Cantilever
One-way slab	1/20	1/25	1/30	1/10
Two-way slab (short span)	1/25	1/30	1/35	-
Joist slab	1/15	1/18	1/20	1/8
Beam	1/10	1/12	1/15	1/5

If more reliable methods based on the principles of structural mechanics, which also take reinforced concrete behavior into account, are not used, the immediate deflections can be calculated by using the approximate method given in Clause 13.2.2 and the time-dependent deflections by the method presented in Clause 13.2.3.

13.2.2 - Approximate Calculation for Instantaneous Deflection

Instantaneous deflection values of reinforced concrete flexural members under permanent and live loads with uncracked sections in the span ($M_{max} \leq M_{cr}$) should be calculated in accordance with structural mechanics principles by using the gross sectional moment of inertia.

For cracked sections ($M_{max} > M_{cr}$), the instantaneous deflection should be calculated as given above using the effective moment of inertia values obtained from Equation 13.1 and by considering support conditions. The modulus of elasticity E_c to be used in this calculation should be taken from Section 3.3.

$$I_{ef} = \left(\frac{M_{cr}}{M_{\max}}\right)^3 I_c + \left[1 - \left(\frac{M_{cr}}{M_{\max}}\right)^3\right] I_{cr} \quad (13.1)$$

$$M_{cr} = 2.5 f_{ctd} \frac{I_c}{y} \quad (13.2)$$

The cracking moment for the section should be calculated in accordance with Section 13.2. For continuous beams and slabs, the two moment of inertia values of the span section and the support section (average of two supports) should be calculated separately by using Equation13.1 and then the average of these two values should be used as the effective moment of inertia. In the case of cantilevers, the moment of inertia of the support section should be used as the effective moment of inertia.

13.2.3 - Time Dependent Deflection Calculation

Calculation of additional long-term deflections due to shrinkage and creep in reinforced concrete structures should be made in accordance with Clause 3.3.4. If more precise calculations are not required, the total deflection, including the time dependent deflection can be obtained by using Equation 13.3.

$$\delta_t = \delta_i + \delta_{ig}\,\lambda \quad (13.3)$$

$$\lambda = \frac{\gamma_t}{1 + 50\rho'} \quad (13.4)$$

The coefficient, γ_t, in the above equation, which depends on the duration of the permanent loading, can be obtained from Table 13.2. ρ’ is the ratio of compression reinforcement in the section.

Table 13.2 Permanent Load Duration Coefficient

Loading Duration	Duration Coefficient γ_t
5 years or more	2.0
12 months	1.4
6 months	1.2
3 months	1.0

13.3 - Deflection Limits

Allowable maximum deflections for flexural members are given in Table 13.3: ℓ_n is the clear span.

Table 13.3 Deflection Limits

Flexural member and location	Cause of deflection	Span/Deflection
Roof member with no partition walls	Instantaneous deflection due to live load	ℓ_n / 180
Floor member with no partition wall	Instantaneous deflection due to live load	ℓ_n / 360
Roof or floor member with partition walls(*)	Sum of deflection due to permanent load and deflection due to the remainder of live the load	ℓ_n / 480
Roof or floor member with partition walls		ℓ_n / 240

(*) Carrying members that may be affected by large deflections or where a partition wall exists.

13.4 - CRACK CONTROL

13.4.1 - General Regulations

Wide cracks that would impair the appearance of structures or lead to reinforcement corrosion should not be allowed. The width of cracks caused by deflections due to bending, shear or torsional effects and due to external loading as well as by volumetric deformations such as shrinkage and creep, or by tensile stresses due to support movements, should not exceed the values given in Table 13.4.

Table 13.4 Limits for Crack Width

Environment	ω_{max}
Interior of structure under normal environmental condition	0.4 mm
Interior of structure under moist and exterior of structure under normal exterior environmental conditions	0.3 mm
Exterior of structure under moist environmental conditions	0.2 mm
Interior and exterior of structure under unfavorable (aggressive) environmental conditions	0.1 mm

Crack control is not necessary if all the requirements given below are satisfied:

- Deformed bars have been used
- In the tensile zone of reinforced concrete members, at least the minimum tension reinforcement defined in Section 7.3 has been provided
- Unfavorable environmental conditions do not exist
- The reinforcement spacing does not exceed 200mm.

13.4.2 - Crack Width Calculation

The design crack width, ω, limits of which are given in Table 13.4, can be calculated by using the equation below for members with deformed bars. For plain bars this value should be increased by multiplying it with a factor of 1.7. The stress in the reinforcement, σ_s, should be calculated by using unfactored loads. However for steel stress, $0.7\times f_{yd}$ can be used as an approximation.

$$\omega = 1.3(A_t c)^{\frac{1}{3}} \sigma_s \times 10^{-5} \qquad (13.5)$$

A_t described in Figure 13.1 is the effective concrete area for each reinforcing bar resisting tension. If the diameters of the reinforcing bars differ, an equivalent "n" value should be obtained by dividing the total reinforcement area by the diameter of the biggest reinforcing bar.

If the total amount of tension reinforcement in the member is more than 1.2 times that required, the crack width calculated can be reduced by this ratio.

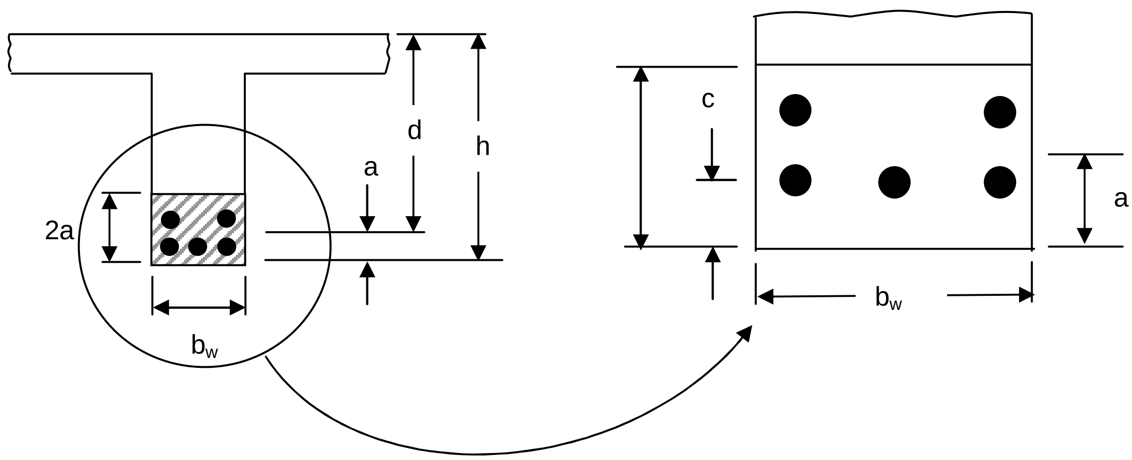


Figure 13.1 Reinforcement area in cracked region for members under pure bending

14 - EVALUATION OF BUILDING STRENGTH

14.0 - NOTATION

h	Member height
ℓ_t	Span length in case of members under test, the length of the short edge in slabs The bigger value of the distance between two support centers and the sum of the clear span length and the member depth (twice the net span in cantilevers)
Δ_{m1}	The maximum deflection value measured under load in the first loading test
Δ_{m2}	The maximum deflection value measured under load in the second loading test
Δ_{mp1}	The maximum residual deflection value measured in the first loading test after load removal
Δ_{mp2}	The maximum residual deflection value measured in the second loading test after load removal

14.1 - GENERAL REGULATIONS

If there is any doubt about the structural safety of the whole building or some parts of the building, the investigation and evaluation of the safety should be carried out in accordance with the requirements of this section. If the effects of the problems in the building on the structural safety are fully understood and if the dimensions of the structural members and their material properties required for analysis have been established, an analytical evaluation of the structural safety can be conducted. However, if the effects of the problems in the building on the structural safety have not been fully understood or the member dimensions and material properties have not been determined, a loading test should be carried out.

14.2 - ANALYTICAL EVALUATION

14.2.1 - Determination of Member Dimensions and Material Properties

It is sufficient from the view point of structural safety to measure the cross-sectional dimensions of structural elements at critical sections. The locations and diameters of reinforcing bars should be determined by on-site measurements. However, if the measurements conducted at a sufficient number of carefully chosen points on a variety of members in different parts of the structure are found to confirm the design drawings, it may be assumed that the diameter and location of reinforcement for the whole building as shown in the design drawings are correct. The engineer in charge of evaluation should determine the number of locations where measurements have to be made.

If it is necessary to determine concrete strength, a comprehensive study based on non-destructive tests methods should be carried out. However, the results obtained from these tests should definitely be correlated with core test results that are much more reliable.

Dimensions and locations of cores should be chosen by the engineer so that the safety of the structure will not be adversely affected. Core samples should be packed and transported so that they will not lose their natural humidity.

Cores should not be stored in water and should be capped before testing. Each core should be inspected by the engineer before testing in order to determine whether it is suitable. Cracks, voids, pieces of reinforcing bars, etc., in cores should be noted and considered in the evaluation of the test results.

The results of tests on cores should be evaluated only by a responsible engineer. In these evaluations, effects of the dimension and shape should be considered and necessary modifications should be made in the strength evaluation.

If considered necessary, the reinforcement strength should be determined by carrying out material tests on representative samples of reinforcing bars taken from the structure. If there is the possibility of a change in the material properties of the reinforcement due to fire, it is definitely necessary to carry out such tests.

If the required dimensions of coating materials for members and slabs and of non-structural partition walls are determined by carrying out a suitable number of reliable measurements and tests, the permanent load factor can be reduced from 1.4 to 1.2 in the analytical evaluation of structural safety. If the material strengths have also been determined with reliability, the material factors to be used may also be reduced. However, the material factor can never be less than 1.3 for concrete and/or less than 1.1 for the reinforcing steel.

14.3 - LOAD TEST

14.3.1 - General Regulations

Loading tests should be carried out only in connection with flexural problems in a structure. If the sudden collapse of certain members or the whole structure is in question during the load test (e.g. due to stability, shear or punching), load tests should not be carried out for reasons of safety.

The number and arrangement of spans or panels to be loaded and the loading arrangement should be selected so as to cause the maximum deflection and internal forces in the most critical regions of the structure. More than one test load arrangement for different regions of the structure should be used if a single load arrangement is not found adequate for all critical regions.

In the load test, the total load on the selected spans or panels, taking into account the existing permanent load, should not be less than 0.85×(1.4G+1.7Q).

A load test should not be carried out until the concrete in the floor to be loaded is at least 56 days old. However if the owner, contractor, control organization and design engineer agree among themselves that a load test be permitted before the concrete is 56 days old, the load test may be conducted without reference to the age of the concrete.

14.3.2 - Application Principles

Measurements should be made at the most critical regions for deflections and cracks. The reference zero values for all measurements should be obtained at most one hour before the start of the loading test.

The test load should be applied in at least four stages and the load increments should be as equal as possible.

For the application of test load, bags of cement, bricks, tiles, sand, water, etc. can be used. The loading should be as uniformly distributed as possible, and the transfer of load to certain locations due to the arching action resulting from the stacking of the load should be prevented.

After the total test load has been in place for 24 hours, a set of measurements should be taken, immediately after which the total test load should be removed. A set of final measurements should also be taken 24 hours after removal of the test load.

14.3.3 - Acceptance Criteria

Structures or structural members subjected to load shall be considered to have adequate safety if the criteria given below are taken to be satisfied,

The structure or portion thereof, which has been tested, should show no evidence of severe damage such as crushing of compressed concrete or buckling of compression reinforcement.

In the case of loaded members, diagonal cracks which could to lead to shear failure should not be present.

In the case of loaded members, cracks along the flexural reinforcement, which may be considered a sign of bond failure, should not be present.

The maximum deflection under load, Δ_{m1}, and the maximum residual deflection after removing load, Δ_{mp1}, should not exceed the values given below.

$$\frac{\Delta_{m1}}{\ell_t} \le 0.00005\frac{\ell_t}{h} \qquad (14.1)$$

$$\Delta_{mp1} \le 0.25\Delta_{m1} \qquad (14.2)$$

If the above conditions are not satisfied, another load test may be permitted.

The second test should be carried out at least 72 hours after the removal of the first test loading. If the condition below is satisfied, the member or structure to which test loading has been applied for the second time may be considered to be safe under vertical loading.

$$\Delta_{mp2} \le 0.2\Delta_{m2} \qquad (14.3)$$

If this condition also is not satisfied, the loading test cannot be repeated.

14.3.4 - Safety Provisions

Load tests should be conducted in taking necessary measures with care to provide for the safety of the workers during the test. The engineer conducting the load test is also responsible for the safety measures. The engineer is obliged to monitor the test continuously and stop the test in case any dangerous situation is observed to arise.