

ENGINEERING STANDARD
FOR
CONCRETE STRUCTURES

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0. INTRODUCTION

This Engineering Standard provides a means of establishing minimum requirements to be considered for designs by engineers and consultants of Iranian Oil Industries.

The Standard is intended to cover the usual types of all structural members, both large and small. The requirements of this Engineering Standard shall also be met and applied, where there is use of software packages.

This Engineering Standard consists of one part which specifies the minimum requirements for design of sections and three Appendices specifying the requirements for design of structural elements.

1. SCOPE

This Engineering Standard provides minimum requirements for design of reinforced, non reinforced, prestressed and precast concrete structural elements of any structure erected at the plants of oil, gas and petrochemical Industries. The standard has been prepared on the assumption that all load-bearing elements are of concrete.

This Standard shall govern in all matters pertaining to design, wherever this Standard is in conflict with requirements mentioned in other specifications referenced in this Standard.

This Standard does not include design of tanks, reservoirs, silos, chimneys, piles, and piers, which are dealt with in their relevant standards.

2. REFERENCES

In this Standard the following standards and codes are referred to and to the extent specified form a part of this Standard:

ACI (AMERICAN CONCRETE INSTITUTE)

ACI 116R- 1989	"Cement and Concrete Terminology"
ACI 201-1R- 1989	"Guide for Making a Condition Survey of Concrete in Service"
ACI 201-2R- 1989	"Guide to Durable Concrete"
ACI 210R- 1989	"Erosion of Concrete in Hydraulic Structure"
ACI 212-1R- 1989	"Admixtures for Concrete"
ACI 212-2R- 1989	"Guide for Use of Admixtures in Concrete"
ACI 214- 1989	"Rec. Practice for Evaluation of Strength Test Results of Concrete"
ACI 215R- 1989	"Considerations for Design of Concrete Structures Subjected to Fatigue Loading"
ACI 117- 1989	"Standard Tolerance for Concrete Construction & Materials"
ACI 224R- 1989	"Control of Cracking in Concrete Structures"
ACI 315 -1980, 86	"Details and Detailing of Concrete Reinforcement"
ACI 318M- 1983, 89	"Bldg. Code Requirements for Rein. Concrete"
ACI 435-3R- 1989	"Allowable Deflections"
ACI 442R- 1989	"Response of Concrete Bldg. to Lateral Forces"
ACI 343R- 1989	"Analysis and Design of Reinforced Concrete Bridge Structures"
ACI 304R 74-24- 1989	"Guide to Joint Sealants for Concrete Structures"
ACI 506R- 1989	"Guide to Shotcrete"
ACI 544.1R-82- 1986	"State of the Art Report on Fiber Reinforced Concrete"
ACI 544-2R- 1986	"Measurement of Properties of Fiber Reinforced Concrete"
ACI 548R- 1988	"Polymers in Concrete"
ACI 301- 1989	"Specs. for Structural Concrete for Bldg."
ACI 318.1M- 318.1RM-89- 1989	"Building Code Requirements for Structural Plain Concrete"

ASTM (AMERICAN SOCIETY FOR TESTING AND MATERIALS)

For ASTM standards referred to in this Standard see Part 2, Section 3.8 of ACI 318M-89.

3. DEFINITIONS AND TERMINOLOGY

For definition of terms used in this Standard reference is made to Part 1, Chapter 2 of ACI 318M-89 "Building Code Requirements for Reinforced Concrete".

4. SYMBOLS AND ABBREVIATIONS

a	= depth of equivalent rectangular stress block
A	= area of that part of cross section between flexural tension face and center of gravity of gross section, mm ²
A_b	= area of an individual bar, mm ²
A_c	= area of core of spirally reinforced compression member measured to outside diameter of spiral, mm ²
A_s and A'_s	= area of compression reinforcement, mm ²
A_{st}	= total area of longitudinal reinforcement, (bars or steel shapes), mm ²
A_t	= area of structural steel shape, pipe, or tubing in composite section, mm ²
A_v	= area of shear reinforcement within a distance s , or area of shear reinforcement perpendicular to flexural tension reinforcement within a distance s for deep flexural members, mm ²
A_l	= loaded area
A_2	= the area of the lower base of the largest frustum of a pyramid, cone, or tapered wedge contained wholly within the support and having for its upper base the loaded area, and having side slopes of 1 vertical to 2 horizontal
b	= width of compression face of member, mm
b	= effective compressive flange width of a structural member, mm
b_o	= perimeter of critical section for slabs and footings, mm
b_t	= width of that part of cross section containing the closed stirrups resisting torsion
b_v	= width of cross section at contact surface being investigated for horizontal shear
b_w	= web width, or diameter of circular section, mm
b_1	= width of the critical section measured in the direction of the span for which moments are determined, mm
b_2	= width of the critical section measured in the direction perpendicular to b_1 , mm.
c	= distance from extreme compression fiber to neutral axis, mm
C	= cross-sectional constant to define torsional properties
d and d'	= distance from extreme compression fiber to centroid of tension or compression reinforcement, respectively, mm
d_b	= nominal diameter of bar, wire, or prestressing strand, mm
d_c	= thickness of concrete cover measured from extreme tension fiber to center of bar or wire located closest thereto, mm
d_p	= distance from extreme compression fiber to centroid of prestressed reinforcement
d_s	= distance from extreme tension fiber to centroid of tension reinforcement mm
D	= dead loads, or related internal moments and forces
e	= eccentricity of load parallel to axis of member measured from centroid of gross section

E	= load effects of earthquake, or related internal moments and forces
E_c	= Modulus of elasticity of concrete, MPa
El	= flexural stiffness of compression member
E_s	= modulus of elasticity of reinforcement, MPa
f_c	= specified compressive strength of concrete, MPa
f_{cr}	= average strength to be used as basis for selecting concrete proportions, MPa
f_{cr}	= required average compressive strength of concrete used as the basis for selection of concrete proportions, MPa
f_{ci}	= compressive strength of concrete at time of initial prestress, MPa
f_{ct}	= average splitting tensile strength of low-density aggregate concrete, MPa
f_d	= stress due unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied loads, MPa
f_r	= modulus of rupture of concrete, MPa
f_s	= calculated stress in reinforcement at service loads, MPa
f_t	= tensile strength of concrete
f_y	= specified yield strength of nonprestressed reinforcement, MPa
f_{yh}	= specified yield strength of transverse reinforcement, MPa
h	= overall thickness of member, mm
h_c	= cross-sectional dimension of column core measured center-to-center of confining reinforcement
H	= loads due to weight and pressure of soil, water in soil or other materials, or related internal moments and forces
k	= effective length factor for compression members
l	= span length of beam or one-way slab, clear projection of cantilever, mm
l_d	= development length, mm
l_{dh}	= development length for a bar with a standard hook
l_n	= length of clear span in direction that moments are being determined, measured face-to-face of supports
l_1	= length of span in direction that moments are being determined, measured center-to-center of supports
l_2	= length of span transverse to l_1 measured center-to-center of supports
L	= live loads, or related internal moments and forces

M	= moment
M_a	= maximum moment in member at stage deflection is computed
M_c	= factored moment to be used for design of compression member
M_d	= moment due to dead load
M_{cr}	= cracking moment
M_{max}	= maximum factored moment at section due to externally applied loads
M_o	= total factored static moment
M_n	= nominal moment strength
M_s	= portion of slab moment balanced by support moment
M_u	= factored moment at section
n	= number of consecutive strength tests
N	= number of bars in a layer being spliced or developed at a critical section
r	= radius of gyration of cross section of a compression member
T	= cumulative effects of temperature, creep, shrinkage, and differential settlement
T_u	= factored torsional moment at section
U	= required strength to resist factored loads or related internal moments and forces
v	= design shear stress
v_c	= permissible shear stress carried by concrete, MPa
V_n	= nominal shear strength
V_s	= nominal shear strength provided by shear reinforcement
w	= load per unit length or per unit area
W_c	= mass of concrete, kg/m ³
y	= longer overall dimension of rectangular part of cross section
y_t	= distance from centroidal axis of gross section, neglecting reinforcement, to extreme fiber intention
α_t	= coefficient as a function of y_1/x_1
δ_b	= moment magnification factor for frames braced against sidesway, to reflect effects of member curvature between ends of compression member
δ_s	= moment magnification factor for frames not braced against sidesway, to reflect lateral drift resulting from lateral and gravity loads
λ (lambda)	= multiplier for additional long-time deflection
$\rho(\text{rho})$	= ratio of non-prestressed tension reinforcement
ρ'	= reinforcement ratio for nonprestressed compression reinforcement, A'_s / bd
ρ_w	= $A_s / b_w d$

5. UNITS

This Standard is based on International System of Units (SI), except where otherwise specified.

6. GENERAL REQUIREMENTS

6.1 Design Methods

6.1.1 In design of reinforced concrete structures, members shall be proportioned for adequate strength in accordance with provisions of this Standard using load factors and strength reduction factors ϕ specified in Section 9.3 of ACI 318M-1989.

6.1.2 Loading

6.1.2.1 Design provisions of this Standard are based on the assumption that structures shall be designed to resist all applicable loads.

6.1.2.2 Service loads shall be in accordance with IPS-E-CE-500.

6.1.2.3 In design for wind and earthquake loads, integral structural parts shall be designed to resist the total lateral loads.

6.1.2.4 Consideration shall be given to effects of forces due to prestressing, crane loads, vibration, impact, shrinkage, temperature changes, creep, and unequal settlement of supports.

6.2 Methods of Analysis

6.2.1 Structural stability and stiffness

6.2.1.1 Special attention shall be paid to the structural stability and stiffness of buildings. If it is not obvious whether a building is of sufficient stability, a stability analysis of the horizontal and vertical stiffening members shall be made, any dimensional deviations and any accidental eccentricity of vertical loads also being taken into account.

6.2.1.2 If the stiffening members are of pliable material, any deformations they may undergo shall be taken into account in the analysis. Safety against buckling shall be checked for vertical stiffening members, unless vertical stiffening is provided by such members as section walls or stair wells.

6.2.1.3 If masonry walls also make a contribution to stiffening they shall be considered load bearing walls.

6.2.2 Allowance for edge restraint

When calculating the mid span moment in the end span, restraint at the outer support shall not be taken into account unless suitable detailing arrangements have been made and an appropriate analysis carried out. The torsion resistance of beams shall not be taken into account unless a realistic picture of their torsional stiffness has been obtained. Otherwise, torsional stiffness shall be neglected.

6.3 Load Distribution

6.3.1 All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the theory of elastic analysis, except as modified according to Sub-clause 6.3.4. It is permitted to simplify design by using the assumptions specified in Sub-clause 6.5 through 6.8.

6.3.2 Except for prestressed concrete, approximate methods of frame analysis are permitted for buildings of usual types of construction, spans, and story heights.

6.3.3 In lieu of frame analysis, the following approximate moments and shears are permitted for design of continuous beams and one-way slabs (slabs reinforced to resist flexural stresses in only one direction), provided:

- a) there are two or more spans;
- b) spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 percent;
- c) loads are uniformly distributed;
- d) unit live load does not exceed three times unit dead load, and;
- e) members are prismatic.

Positive moment

End spans

Discontinuous end	
Unrestrained	$W_u l_n^2 / 11$
Discontinuous end integral with support	$W_u l_n^2 / 14$
Interior spans	$W_u l_n^2 / 16$

Negative moment at exterior face of first interior support

Two spans	$W_u l_n^2 / 9$
More than two spans	$W_u l_n^2 / 10$

Negative moment at other faces of interior supports $W_u l_n^2 / 11$

Negative moment at face of all supports for:

Slabs with spans not exceeding 3 m; and beams where ratio of sum of column stiffnesses to beam stiffness exceeds eight at each end of the span	$W_u l_n^2 / 12$
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Negative moment at interior face of exterior support for members built integrally with supports

Where support is a spandrel beam	$W_u l_n^2 / 24$
Where support is a column	$W_u l_n^2 / 16$

Shear in end members at face of first interior support $1.15 W_u l_n^2 / 2$

Shear at face of all other supports $W_u l_n^2 / 2$

6.3.4 Redistribution of negative moments in continuous non-prestressed flexural members

6.3.4.1 Except where approximate values for moments are used, it is permitted to increase or decrease negative moments calculated by elastic theory at supports of continuous flexural members for any assumed loading arrangement by not more than

$$20 - 1 \frac{\circ}{\circ} \frac{\circ}{b} \text{ per cen}$$

6.3.4.2 The modified negative moments shall be used for calculating moments at sections within the spans.

6.3.4.3 Redistribution of negative moments shall be made only when the section at which moment is reduced is so designed that ρ or $\rho - \rho'$ is not greater than $0.50 \rho_b$.

Where:

$$\rho_b = \frac{0.85 \beta_1 f'_c}{f_y} - \frac{600}{600 + f_y}$$

6.4 Modulus of Elasticity

6.4.1 Modulus of elasticity E_c for concrete may be taken as $w_c^{1.5} 0.043 \sqrt{f'_c}$ (in MPa) for values of w_c between 1500 and 2500 kg/m³. For normal density concrete, E_c may be taken as $4700 \sqrt{f'_c}$.

6.4.2 Modulus of elasticity E_s for nonprestressed reinforcement may be taken as 200,000 MPa.

6.4.3 Modulus of elasticity E_s for prestressing tendons shall be determined by tests or supplied by the manufacturer.

6.5 Stiffness

6.5.1 Any reasonable assumptions may be adopted for computing relative flexural and torsional stiffnesses of columns, walls, floors, and roof systems. Assumptions shall be consistent throughout analysis.

6.5.2 Effect of haunches shall be considered both in determining moments and in design of members.

6.6 Span Length

6.6.1 Span length of members not built integrally with supports shall be considered the clear span plus depth of member but need not exceed distance between centers of supports.

6.6.2 In analysis of frames or continuous construction for determination of moments, span length shall be taken as the distance center-to-center of supports.

6.6.3 For beams built integrally with supports, moments at faces of support should be used for design.

6.6.4 Solid or orribbed slabs built integrally with supports, with clear spans not more than 3 meter, may be analyzed as continuous slabs on knife edge supports with spans equal to the clear spans of the slab and width of beams otherwise neglected.

6.7 Columns

6.7.1 Columns shall be designed to resist the axial forces from factored loads on all floors or roof and the maximum moment from factored loads on a single adjacent span of the floor or roof under consideration. Loading condition giving the maximum ratio of moment to axial load shall also be considered.

6.7.2 In frames or continuous construction, consideration shall be given to the effect of unbalanced floor or roof loads on both exterior and interior columns and of eccentric loading due to other causes.

6.7.3 In computing gravity load moments in columns, it is permitted to assume as fixed far ends of columns built integrally with the structure.

6.7.4 Resistance to moments at any floor or roof level shall be provided by distributing the moment between columns immediately above and below the given floor in proportion to the relative column stiffnesses and conditions of restraint.

6.8 T-Beam Construction

6.8.1 In T-beam construction, the flange and web shall be built integrally or otherwise effectively bonded together.

6.8.2 Width of slab effective as a T-beam flange shall not exceed one-quarter of the span length of the beam, and the effective overhanging flange width on each side of the web shall not exceed:

- a) eight times the slab thickness, and
- b) one-half the clear distance to the next web.

6.8.3 For beams with a slab on one side only, the effective overhanging flange width shall not exceed:

- a) one-twelfth the span length of the beam,
- b) six times the slab thickness, and
- c) one-half the clear distance to the next web.

6.8.4 Isolated beams, in which the T-shape is used to provide a flange for additional compression area, shall have a flange thickness not less than one-half the width of web and an effective flange width not more than four times the width of web.

6.8.5 Where primary flexural reinforcement in a slab that is considered as a T-beam flange (excluding joist construction) is parallel to the beam, reinforcement perpendicular to the beam shall be provided in the top of the slab in accordance with the following:

6.8.5.1 Transverse reinforcement shall be designed to carry the factored load on the overhanging slab width assumed to act as a cantilever. For isolated beams, the full width of overhanging flange shall be considered. For other T-beams, only the effective overhanging slab width need be considered.

6.8.5.2 Transverse reinforcement shall be spaced not farther apart than five times the slab thickness, nor 500 mm.

6.9 Joist Construction

6.9.1 Joist construction consists of a monolithic combination of regularly spaced ribs and a top slab arranged to span in one direction or two orthogonal directions.

6.9.2 Ribs shall be not less than 100 mm in width; and shall have a depth of not more than $3\frac{1}{2}$ times the minimum width of rib.

6.9.3 Clear spacing between ribs shall not exceed 800 mm.

6.9.4 Joist construction not meeting the limitations of 6.9.1 through 6.9.3 shall be designed as slabs and beams.

6.9.5 When permanent burned clay or concrete tile fillers of material having a unit compressive strength at least equal to that of the specified strength of concrete in the joists are used:

6.9.5.1 For shear and negative-moment strength computations, it is permitted to include the vertical shells of fillers in contact with the ribs. Other portions of fillers shall not be included in strength computations.

6.9.5.2 Slab thickness over permanent fillers shall be not less than one-twelfth the clear distance between ribs, nor less than 40 mm.

6.9.5.3 In one-way joists, reinforcement normal to the ribs shall be provided in the slab as required by Clause 8.7.

6.9.6 When removable forms or fillers not complying with 6.9.5 are used:

6.9.6.1 Slab thickness shall be not less than one-twelfth the clear distance between ribs, nor less than 50 mm.

6.9.6.2 Reinforcement normal to the ribs shall be provided in the slab as required for flexure, considering load concentrations, if any, but not less than required by Clause 8.7.

6.9.7 Where conduits or pipes as permitted by Section 6.3 of ACI 318 M-89 are embedded within the slab, slab thickness shall be at least 25 mm greater than the total overall depth of the conduits or pipes at any point. Conduits or pipes shall not impair significantly the strength of the construction.

6.9.8 For joist construction, contribution of concrete to shear strength V_c is permitted to be 10 percent more than that specified in Clause 9. It is permitted to increase shear strength using shear reinforcement or by widening the ends of ribs.

6.10 Separate Floor Finish

6.10.1 A floor finish shall not be included as part of a structural member unless placed monolithically with the floor slab or designed in accordance with requirements of Appendix C.

6.10.2 It is permitted to consider all concrete floor finishes as part of required cover or total thickness for nonstructural considerations.

6.11 Alternate Design Method

6.11.1 Scope

Nonprestressed reinforced concrete members may be designed using service loads (without load factors) and permissible service load stresses in accordance with provisions of this Clause. For design of members not covered by this Clause, appropriate provisions of this Standard shall apply.

All applicable provisions of this Standard for nonprestressed concrete except Clause 8.4 of ACI 318M-89 shall apply to members designed by the Alternate Design Method.

Flexural members shall meet requirements for deflection control in Clause 7.4 of this Standard, and requirements of Section 10.4 through 10.7 of ACI-318M-89.

6.11.2 General

Load factors and strength reduction factors shall be taken as unity for members designed by the Alternate Design Method.

Members may be proportioned for 75 percent of capacities required by other parts of this Clause when considering wind or earthquake forces combined with other loads, provided the resulting section is not less than that required for the combination of dead and live load.

When dead load reduces effects of other loads, members shall be designed for 85 percent of dead load in combination with the other loads.

6.11.3 Permissible service load stresses

Stresses in concrete shall not exceed the following:

a) Flexure

Extreme fiber stress in compression 0.45 f'_c .

b) Shear ⁽¹⁾

Beams and one-way slabs and footings:

Shear carried by concrete, v_c $\sqrt{f'_c} / 11$

Maximum shear

Carried by concrete plus shear reinforcement $v_c + 3 \sqrt{f'_c} / 8$

Joists ⁽²⁾

Shear carried by concrete, v_c $\sqrt{f'_c} / 10$

Two-way slabs and footings:

Shear carried by Concrete, v_c $1 + \frac{2}{\sqrt{c}} \sqrt{f'_c} / 12$
but not greater than $\sqrt{f'_c} / 6$.

c) Bearing on loaded area ⁽²⁾ $0.3 f'_c$.

Tensile stress in reinforcement f_s shall not exceed the following:

a) Grade 40 or Grade 50 reinforcement 140 MPa.

b) Grade 60 reinforcement or greater and welded wire fabric (plain or deformed)..... 170 MPa.

c) For flexural reinforcement, 10 mm or less in diameter,
in one-way slabs of not more than 4 m span, $0.50 f_y$
but not greater than 200 MPa.

Notes:

1) For more detailed calculation of shear stress carried by concrete v_c and shear values for lightweight aggregate concrete see 6.11.8.

2) Designed in accordance with Clause 6.9 of this Standard.

6.11.4 Development and splices of reinforcement

Development and splices of reinforcement shall be as required in Clause 10 of this Standard. In satisfying requirements of Section 12.11.3 of ACI-318M-89, M_n shall be taken as computed moment capacity assuming all positive moment tension reinforcement at the section to be stressed to the permissible tensile stress f_s , and V_u shall be taken as unfactored shear force at the section.

6.11.5 Flexure

For investigation of stresses at service loads, straight-line theory (for flexure) shall be used with the following assumptions:

a) Strains vary linearly as the distance from the neutral axis, except, for deep flexural members with overall depth-span ratios greater than 2/5 for continuous spans and 4/5 for simple spans, a nonlinear distribution of strain shall be considered.

b) Stress-strain relationship of concrete is a straight line under service loads within permissible service load stresses.

c) In reinforced concrete members, concrete resists no tension.

d) Modular ratio, $n = E_s / E_c$, may be taken as the nearest whole number (but not less than 6). Except in calculations for deflections, value of n for low-density concrete shall be assumed to be the same as for normal weight concrete of the same strength.

e) In doubly reinforced flexural members, an effective modular ratio of $2 E_s / E_c$ shall be used to transform compression reinforcement for stress computations. Compressive stress in such reinforcement shall not exceed permissible tensile stress.

6.11.6 Compression members with or without flexure

Combined flexure and axial load capacity of compression members shall be taken as 40 percent of that computed in accordance with provisions in Chapter 10 of ACI 318M-89.

Slenderness effects shall be included according to requirements of 10.10 and 10.11 in Eq.(10-7) and (10-8) of ACI 318M-89 the term P_u shall be replaced by 2.5 times the design axial load, and ϕ shall be taken equal to 1.0.

Walls shall be designed in accordance with Appendix A of this Standard with flexure and axial load capacities taken as 40 percent of that computed using Appendix A. In Eq.(A.1), ϕ shall be taken equal to 1.0.

6.11.7 Shear and torsion

Design shear stress v shall be computed by:

$$v = \frac{V}{b_w d}$$

Where:

V is design shear force at section considered.

When the reaction, in direction of applied shear, introduces compression into the end regions of a member, sections located less than a distance d from face of support may be designed for the same shear v as that computed at a distance d .

Whenever applicable, effects of torsion, in accordance with provisions of Clause 9 of this Standard, shall be added. Shear and torsional moment strengths provided by concrete and limiting maximum strengths for torsion shall be taken as 55 percent of the values given in Clause 9 of this Standard.

6.11.8 Shear stress carried by concrete

a) For members subject to shear and flexure only, shear stress carried by concrete v_c shall not exceed $0.09 \sqrt{f'_c}$ unless a more detailed calculation is made in accordance with 6.11.8(d).

b) For members subject to axial compression, shear stress carried by concrete v_c shall not exceed $.09 \sqrt{f'_c}$ unless a more detailed calculation is made in accordance with 6.11.8(e).

c) For members subject to significant axial tension, shear reinforcement shall be designed to carry total shear, unless a more detailed calculation is made using:

$$v_c = \frac{2}{1 + 0.6 \frac{N}{A_g}} \sqrt{f'_c} = 11$$

Where:

N is negative for tension. Quantity N/A_g shall be expressed in MPa.

d) For members subject to shear and flexure only, v_c may be computed by:

$$v_c = \sqrt[2]{\frac{q}{f_c}} = 12 + 9 \sqrt[3]{\frac{Vd}{M}}$$

but v_c shall not exceed $\sqrt{f'_c} / 7$. Quantity Vd/M shall not be taken greater than 1.0,

Where:

M is design moment occurring simultaneously with V at section considered.

e) For members subject to axial compression, v_c may be computed by:

$$v_c = \sqrt[2]{1 + \frac{N}{A_g}} \sqrt[3]{\frac{q}{f_c}} = 11$$

Quantity N/A_g shall be expressed in MPa.

Shear stresses carried by concrete v_c apply to normal weight concrete. When low-density aggregate concrete is used, one of the following modifications shall apply:

a) When f_{ct} is specified and concrete is proportioned in accordance with Section 4.2 of ACI-318M-89, $1.8 f_{ct}$ shall be substituted for $\sqrt{f'_c}$ but the value of $1.8 f_{ct}$ shall not exceed $\sqrt{f'_c}$.

b) When f_{ct} is not specified, the value of $\sqrt{f'_c}$ shall be multiplied by 0.75 for "all-lightweight" concrete and by 0.85 for "sand-lightweight" concrete. Linear interpolation is permitted when partial sand replacement is used.

In determining shear stress carried by concrete v_c , whenever applicable, effects of axial tension due to creep and shrinkage in restrained members shall be considered and effects of inclined flexural compression in variable-depth members may be included.

6.11.9 Shear stress carried by shear reinforcement

6.11.9.1 Types of shear reinforcement

Shear reinforcement may consist of:

- a) stirrups perpendicular to axis of member;
- b) welded wire fabric with wires located perpendicular to axis of member making an angle of 45° or more with longitudinal tension reinforcement;
- c) longitudinal reinforcement with bent portion making an angle of 30° or more with longitudinal tension reinforcement;
- d) combinations of stirrups and bent longitudinal reinforcement;
- e) spirals.

6.11.9.2 Design yield strength of shear reinforcement shall not exceed 400 MPa.

6.11.9.3 Stirrups and other bars or wires used as shear reinforcement shall extend to a distance d from extreme compression fiber and shall be anchored at both ends according to Clause 10.12 of this Standard, to develop design yield strength of reinforcement.

6.11.10 Spacing limits for shear reinforcement

- a) Spacing of shear reinforcement placed perpendicular to axis of member shall not exceed $d/2$, nor 600 mm.
- b) Inclined stirrups and bent longitudinal reinforcement shall be so spaced that every 45-deg line, extending toward the reaction from middepth of member ($d/2$) to longitudinal tension reinforcement, shall be crossed by at least one line of shear reinforcement.
- c) When $(v - v_c)$ exceeds $\sqrt{f'_c} / 6$, maximum spacing given in 6.11.10.a and 6.11.10.b shall be reduced by one-half.

6.11.11 Minimum shear reinforcement

6.11.11.1 A minimum area of shear reinforcement shall be provided in all reinforced concrete flexural members where design shear stress v is greater than one-half the permissible shear v_c carried by concrete, except:

- a) slabs and footings;
- b) concrete joist construction defined by Clause 6.9 of this Standard;
- c) beams with total depth not greater than 250 mm, $2\frac{1}{2}$ times thickness of flange, or one-half the width of web, whichever is greatest.

6.11.11.2 Minimum shear reinforcement requirements of 6.11.11.1 are waived if shown by test that required ultimate flexural and shear strength can be developed when shear reinforcement is omitted.

6.11.11.3 Where shear reinforcement is required by 6.11.11.1 or by analysis, minimum area of shear reinforcement shall be computed by:

$$A_v = \frac{b_w s}{3 f_y}$$

Where:

b_w and s are in millimeters

6.11.12 Design of shear reinforcement

6.11.12.1 Where design shear stress v exceeds shear stress carried by concrete v_c , shear reinforcement shall be provided in accordance with 6.11.12.2 through 6.11.12.8.

6.11.12.2 When shear reinforcement perpendicular to axis of member is used:

$$A_v = \frac{(v - v_c) b_w s}{f_s}$$

6.11.12.3 When inclined stirrups are used as shear reinforcement,

$$A_v = \frac{(v - v_c) b_w s}{f_s (\sin \alpha + \cos \alpha)}$$

6.11.12.4 When shear reinforcement consists of a single bar or single group of parallel bars, all bent up at the same distance from the support,

$$A_v = \frac{(v - v_c) b_w d}{f_s \sin \alpha}$$

Where:

$(v - v_c)$ shall not exceed $\sqrt{f'_c} / 8$.

6.11.12.5 When shear reinforcement consists of a series of parallel bent-up bars or groups of parallel bent-up bars at different distances from the support, required area shall be computed by Eq. of Subclause 6.11.12.3.

6.11.12.6 Only the center three-quarters of the inclined portion of any longitudinal bent bar shall be considered effective for shear reinforcement.

6.11.12.7 When more than one type of shear reinforcement is used to reinforce the same portion of a member, required area shall be computed as the sum of the various types separately. In such computations, v_c shall be included only once.

6.11.12.8 Value of $(v - v_c)$ shall not exceed $3 \sqrt{f'_c} / 8$.

6.11.13 Shear-Friction

Where it is appropriate to consider shear transfer across a given plane such as an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times, shear-friction provisions of Clause 9.5 of this Standard may be applied, with limiting maximum stress for shear taken as 55 percent of that given in Section 11.7.5 of ACI-318M-89. Permissible stress in shear-friction reinforcement shall be that given in Clause 6.11.3.

6.11.14 Special provisions for slabs and footings

6.11.14.1 Shear capacity of slabs and footings in the vicinity of concentrated loads or reactions is governed by the more severe of two conditions:

- a) Beam action for slab or footing, with a critical section extending in a plane across the entire width and located at a distance d from face of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Clause 6.11.7 through 6.11.9.
- b) Two-way action for slab or footing, with a critical section perpendicular to plane of slab and located so that its perimeter is a minimum, but need not approach closer than $d/2$ to perimeter of concentrated load or reaction area. For this condition, the slab or footing shall be designed in accordance with Clause 6.11.14.2 and 6.11.14.3.

6.11.14.2 Design shear stress v shall be computed by:

$$v = \frac{V}{b_o d}$$

Where:

V and b_o shall be taken at the critical section defined in Sub-clause 6.11.14.1(b).

6.11.14.3 Design shear stress v shall not exceed v_c given by the following Eq. unless shear reinforcement is provided:

$$v_c = \left(1 + \frac{2}{\beta_c} \right) \sqrt{f'_c} \quad \beta_c = 12$$

but v_c shall not exceed $\sqrt{f'_c} / 6$. β_c is the ratio of long side to short side of concentrated load or reaction area. When lightweight aggregate concrete is used, the modifications of Sub-clause 6.11.8 shall apply.

6.11.14.4 If shear reinforcement consisting of bars or wires is provided in accordance with section 11.12.3 of ACI-318-89, v_c shall not exceed $\sqrt{f'_c} / 12$, and v shall not exceed $\sqrt{f'_c} / 4$.

6.11.14.5 If shear reinforcement consisting of steel I- or channel-shaped sections (shearheads) is provided in accordance with Clause 9.10.4 of this Standard, v on the critical section defined in 6.11.14.1(b) shall not exceed $0.3 \sqrt{f'_c}$ and v on the critical section defined in Clause 9.10.4.2 of this Standard shall not exceed $\sqrt{f'_c} / 6$. In Eq.(11.40) and (11.41) of ACI-318M-89, design shear force V shall be multiplied by 2 and substituted for V_u .

6.11.15 Special provisions for other members

For design of deep flexural members, brackets and corbels, and walls, the special provisions of Clause 9 of this Standard shall be used, with shear strengths provided by concrete and limiting maximum strengths for shear taken as 55 percent of the values given in Clause 9. In Section 11.10.6 of ACI-318M-89 the design axial load shall be multiplied by 1.2 if compression and 2.0 if tension, and substituted for N_u .

6.11.16 Composite concrete flexural members

For design of composite concrete flexural members, permissible horizontal shear stress v_h shall not exceed 55 percent of the horizontal shear strengths given in Appendix C.

7. STRENGTH AND SERVICEABILITY REQUIREMENTS

7.1 Required Strength

7.1.1 Required strength U to resist dead load D and live load L shall be at least equal to:

$$U = 1.4D + 1.7L$$

7.1.2 If resistance to structural effects of a specified wind load W are included in design, the following combinations of D , L , and W shall be investigated to determine the greatest required strength U :

$$U = 0.75 (1.4D + 1.7L + 1.7W)$$

where load combinations shall include both full value and zero value of L to determine the more severe condition, and

$$U = 0.9D + 1.3W$$

but for any combination of D , L , and W , required strength U shall not be less than Eq. under Subclause 7.1.1.

7.1.3 If resistance to specified earthquake loads or forces E are included in design, load combinations of Clause 7.1.2 shall apply, except that $1.87E$ shall be substituted for W .

7.1.4 If resistance to earth pressure H is included in design, required strength U shall be at least equal to:

$$U = 1.4D + 1.7L + 1.7H$$

except that where D or L reduce the effect of H , $0.9D$ shall be substituted for $1.4D$ and zero value of L shall be used to determine the greatest required strength U . For any combination of D , L , and H , required strength U shall not be less than Eq. under Subclause 7.1.1.

7.1.5 If resistance to loadings due to weight and pressure of fluids with well-defined densities and controllable maximum heights F is included in design, such loading shall have a load factor of 1.4, and be added to all loading combinations that include live load.

7.1.6 If resistance to impact effects is taken into account in design, such effects shall be included with live load L .

7.1.7 Where structural effects T of differential settlement, creep, shrinkage, or temperature change may be significant in design, required strength U shall be at least equal to:

$$U = 0.75 (1.4D + 1.4T + 1.7L)$$

but required strength U shall not be less than

$$U = 1.4 (D + T)$$

Estimations of differential settlement, creep, shrinkage, or temperature change shall be based on a realistic assessment of such effects occurring in service.

7.2 Design Strength

7.2.1 Design strength provided by a member, its connections to other members, and its cross sections, in terms of flexure, axial load, shear, and torsion, shall be taken as the nominal strength calculated in accordance with requirements and assumptions of this Standard multiplied by a strength reduction factor ϕ .

7.2.2 Strength reduction factor ϕ shall be as follows:

7.2.2.1 Flexure, without axial load	0.90.
7.2.2.2 Axial load, and axial load with flexure. (For axial load with flexure, both axial load and moment nominal strength shall be multiplied by appropriate single value of ϕ .)	
a) Axial tension, and axial tension with flexure	0.90.
b) Axial compression, and axial compression with flexure:	
Members with spiral reinforcement	0.75.
Other reinforced members	0.70.

expect that for low values of axial compression ϕ may be increased in accordance with the following:

For members in which f_y does not exceed 400 MPa with symmetric reinforcement, and with $(h-d'-d_s)/h$ not less than 0.70, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_g$ to zero.

For other reinforced members, ϕ may be increased linearly to 0.90 as ϕP_n decreases from $0.10 f'_c A_g$ or ϕP_b , whichever is smaller, to zero.

7.2.2.3 Shear and torsion	0.85.
7.2.2.4 Bearing on concrete	0.70.

7.2.3 Development lengths specified in Chapter 12 of ACI 318M-95 do not require a ϕ factor .

7.2.4 In regions of high seismic risk, strength reduction factors ϕ shall be given as above except for the following:

7.2.4.1 Except for determining the strength of joints, the shear strength reduction factor shall be 0.6 for any structural member if its nominal shear strength is less than the shear corresponding to the development of the nominal flexural strength of the member. The nominal flexural strength shall be determined considering the most critical factored axial loads and including earthquake effects. Shear strength reduction factor for joints shall be 0.85.

7.2.4.2 The strength reduction factor for axial compression and flexure shall be 0.5 for all frame members with factored axial compressive forces exceeding $(A_g f'_c / 10)$ if the transverse reinforcement does not conform to Section 21.4.4 of ACI 318M-1989.

7.2.4.3 Design strength for reinforcement

Designs shall not be based on a yield strength of reinforcement f_y in excess of 550 MPa, except for prestressing tendons.

7.3 Bearing Strength

7.3.1 Design bearing strength on concrete shall not exceed $\phi (0.85 f'_c A_1)$, except when the supporting surface is wider on all sides than the loaded area, design bearing strength on the loaded area may be multiplied by $\sqrt{A_2 / A_1}$ but not more than 2.

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

7.3.3 Clause 7.3 does not apply to post-tensioning anchorages.

7.4 Control of Deflections

7.4.1 Reinforced concrete members subject to flexure shall be designed to have adequate stiffness to limit deflections or any deformations that may adversely affect strength or serviceability of a structure at service loads.

7.4.2 Minimum thickness stipulated in Table 1 shall apply for one-way construction not supporting or attached to partitions or other construction likely to be damaged by large deflections, unless computation of deflection indicates a lesser thickness may be used without adverse effects.

TABLE 1 - MINIMUM THICKNESS OF NON-PRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE COMPUTED⁽¹⁾

	SIMPLY SUPPORTED	MINIMUM THICKNESS, h		CANTI-LEVER
		ONE END CONTINUOUS	BOTH ENDS CONTINUOUS	
Members	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	‘ = 20	‘ = 24	‘ = 28	‘ = 10
Beams or ribbed one-way slabs	‘ = 16	‘ = 18.5	‘ = 21	‘ = 8

1) Span length ℓ is in millimeters.

Values given shall be used directly for members with normal weight concrete ($w_c = 2300 \text{ kg/m}^3$) and Grade 400 reinforcement. For other conditions, the values shall be modified as follows:

- For structural lightweight concrete having unit weights in the range 1500-2000 kg/m³, the values shall be multiplied by $(1.65 - 0.005 w_c)$ but not less than 1.09, where w_c is the unit weight in kg/m³.
- For f_y other than 400 MPa, the values shall be multiplied by $(0.4 + f_y / 700)$.

7.4.2.1 Where deflections are to be computed, deflections that occur immediately on application of load shall be computed by usual methods or formulas for elastic deflections, considering effects of cracking and reinforcement on member stiffness.

7.4.2.2 Deflection computed in accordance with 7.4.2.1 through 7.4.2.5 shall not exceed limits stipulated in Table 2.

TABLE 2 - MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

TYPE OF MEMBER	DEFLECTION TO BE CONSIDERED	DEFLECTION LIMITATION
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\frac{L}{180}$ (1)
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections	Immediate deflection due to live load L	$\frac{L}{360}$
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections	That part of the total deflection occurring after attachment of nonstructural elements	$\frac{L}{480}$ (2)
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflections	(sum of the long-time deflection due to all sustained loads and the immediate deflection due to any additional live load) ⁽³⁾	$\frac{L}{240}$ (4)

Notes:

- 1) Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponded water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.
- 2) Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.
- 3) Long-term deflection shall be determined in accordance with Clause 7.4.2.5 but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.
- 4) But not greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

7.4.2.3 Unless stiffness values are obtained by a more comprehensive analysis, immediate deflection shall be computed with modulus of elasticity E_c for concrete as specified in Clause 6.4.1 (normal density or low density concrete) and with the effective moment of inertia as follows, but not greater than I_g :

$$I_e = \frac{M_{cr}}{M_a}^3 I_g + 1 - \frac{M_{cr}}{M_a}^3 I_{cr}$$

Where:

$$M_{cr} = \frac{f_r I_g}{y_t}$$

and for normal density concrete:

$$f_{cr} = 0.7 \sqrt{f'_c}$$

When low-density aggregate concrete is used, one of the following modifications shall apply:

- a) When f_{ct} is specified and concrete is proportioned in accordance with IPS-C-CE-200, f_r shall be modified by substituting $1.8 f_{ct}$ for $\sqrt{f'_c}$ but the value of $1.8 f_{ct}$ shall not exceed $\sqrt{f'_c}$.
- b) When f_{ct} is not specified, f_r shall be multiplied by 0.75 for "all-low-density" concrete, and 0.85 for "sand-low-density" concrete. Linear interpolation may be used when partial sand replacement is used.

7.4.2.4 For continuous members, effective moment of inertia may be taken as the average of values obtained from Eq. under Sub-clause 7.4.2.3 for the critical positive and negative moment sections. For prismatic members, effective moment of inertia may be taken as the value obtained from Eq. under Sub-clause 7.4.2.3 at midspan for simple and continuous spans, and support for cantilevers.

7.4.2.5 Unless values are obtained by a more comprehensive analysis, additional long-term deflection resulting from creep and shrinkage of flexural members (normal density or light density concrete) shall be determined by multiplying the immediate deflection caused by the sustained load considered, by the factor:

$$\mu = \frac{\rho'}{1 + 50 \rho'}$$

where ρ' shall be the value at midspan for simple and continuous spans, and at support for cantilevers. Time dependent factor ξ for sustained loads may be taken equal to:

5	years or more	2.0
12	months	1.4
6	months	1.2
3	months	1.0

7.4.3 Two-way construction (non-prestressed)

7.4.3.1 Clause 7.4.3 shall govern minimum thickness of slabs or other two-way construction designed in accordance with provisions of Chapter 13 of ACI-318M-89 and conforming with the requirement of Section 13.6.1.2 of the same chapter.

Thickness of slabs without interior beams spanning between the supports on all sides shall satisfy requirements of one of 7.4.3.2, 7.4.3.3, or 7.4.3.6.

Thickness of slabs with beams spanning between supports on all sides shall satisfy requirements of one of 7.4.3.3, or 7.4.3.6.

7.4.3.2 Minimum thickness of slabs without interior beams spanning between the supports shall be in accordance with the provisions of Table 3 and shall not be less than the following values:

- a)** Slabs without drop panels as defined in 13.4.7.1 and 13.4.7.2 of ACI 318M-89..... 120 mm.
- b)** Slabs with drop panels as defined in 13.4.7.1 and 13.4.7.2 of ACI 318M-89 100 mm.

7.4.3.3 Minimum thickness of slabs with or without beams spanning between the supports on all sides and having a ratio of long to short span not exceeding 2, shall be in accordance with Eq. (9-11), (9-12) and (9-13) of ACI 318M-89, but in no case shall the thickness be less than:

- a)** for $\alpha_m < 2.0$ 120 mm
- b)** for $\alpha_m \geq 2.0$ 90 mm.

TABLE 3 - MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS

YIELD STRESS f_y , MPa ⁽¹⁾	WITHOUT DROP PANELS ⁽²⁾			WITH DROP PANELS ⁽²⁾		
	Exterior Panels		Interior Panels	Exterior Panels		Interior Panels
	Without edge beams	Without edge beams ⁽³⁾		Without edge beams	Without edge beams ⁽³⁾	
300	$\frac{n}{33}$	$\frac{n}{36}$	$\frac{n}{36}$	$\frac{n}{36}$	$\frac{n}{40}$	$\frac{n}{40}$
400	$\frac{n}{30}$	$\frac{n}{33}$	$\frac{n}{33}$	$\frac{n}{33}$	$\frac{n}{36}$	$\frac{n}{36}$

Notes:

1) For values of reinforcement yield stress between 300 and 600 MPa minimum thickness shall be obtained by linear interpolation.

2) Drop panel is defined in Section 13.4 of ACI 318M-89.

3) Slabs with beams between columns along exterior edges. The value of a for the edge beam shall not be less than 0.8.

7.4.3.4 For slabs without beams, but with drop panels extending in each direction from centerline of support a distance not less than one-sixth the span length in that direction measured center-to-center of supports, and a projection below the slab at least one-quarter the slab thickness beyond the drop, thickness required by Eq.(9-11), (9-12) or (9-13) of ACI 318M-89, may be reduced by 10 percent.

7.4.3.5 At discontinuous edges, an edge beam shall be provided with a stiffness ratio α not less than 0.80; or the minimum thickness required by Equations mentioned in Sub-clause 7.4.3.4 shall be increased by at least 10 percent in the panel with a discontinuous edge.

7.4.3.6 For slab thickness less than the minimum thickness refer to Section 9.5.3.6 of ACI-318M-89.

7.4.3.7 Deflection for prestressed concrete shall be calculated according to Sections 9.5.4.1, 9.5.4.2, 9.5.4.3 of ACI 318M-89.

7.4.4 Deflection for composite flexural members shall be according to section 9.5.5 of ACI 318M-89.

8. FLEXURE AND AXIAL LOADS

8.1 Design Assumptions

8.1.1 Provisions of Chapter 10 of ACI 318M-1989 shall apply for design of members subject to flexure or axial loads or to combined flexure and axial loads.

8.1.2 The design analysis shall allow for an adequate safety margin between the working load and shall ensure that the structure continues to function properly under working load.

8.1.3 Allowance for creep shall only be made when the slenderness ratio of compression members in rigid systems is higher than 70 and in sway frames, higher than 45, while at the same time, the design relative eccentricity e/d is greater than 2.

8.1.4 Deformations due to creep shall be calculated for compression members subjected to permanent loads in service conditions (including any imposed loads). Calculations shall be based on permanent deflections of the linear elements and eccentricities including any accidental eccentricity.

8.2 Use of Computer in Design

8.2.1 Calculations pertinent to design shall be filed with the drawings when required by the AR⁽¹⁾. When computer programs are used, design assumptions and identified input and output data may be submitted in lieu of calculations.

8.2.2 When a computer program has been used by the designer, only skeleton data⁽²⁾ should normally be required.

8.2.2.1 Input data should be identified as to member designation, applied loads, and span lengths, etc.

8.2.2.2 Output data should include member designation and the shears, moments, and reactions at key points in the span.

8.2.2.3 Output data for column design, shall include moment magnification factors, where applicable.

8.3 Model Analysis

Model analysis may be used to supplement structural analysis and design calculations.

8.3.1 Documentation of the model analysis should be provided with the related calculations.

8.3.2 Model analysis should be performed by an engineer or architect having experience in this technique.

8.4 Special Provisions for Earthquake Resistance

8.4.1 Seismic risk level of a region shall be determined by BHRC Pub. No. 82 (Iranian Code No. 2800). Procedure of design shall be in accordance with ACI 318M-89, Chapter 21 "Special Provisions for Seismic Design".

8.5 Analysis by Calculation

8.5.1 General

An analysis shall be carried out for all major loads which may be expected to occur during the construction and service life of a building, its stiffness, structural stability and, in certain cases, the adverse redistribution of action effects due to creep also being taken into account.

8.5.2 Calculation of resistance action-effects

8.5.2.1 Analysis shall be based on the assumption of imposed loads on their most unfavorable arrangement, this being determined with the aid of influence lines, where appropriate. If, however, buildings may be taken to be subjected to uniformly distributed imposed loads, it is sufficient to base calculations on the assumption that each span is subjected to a full load in its most unfavorable configuration, the loading pattern changing from span to span.

8.5.2.2 The method of analysis of statically indeterminate structures shall be by elastic analysis.

1) AR = Authorized Representative of the Owner.

2) Skeleton data consist of sufficient input and output data and other information to allow the AR to perform a detailed review and make comparisons using other program and manual calculations.

8.5.2.3 In all calculations a Poisson's ratio μ , equal to 0.2 shall be assumed.

8.5.2.4 The coefficient of thermal expansion and contraction for normal weight concrete may be taken as 1.2×10^{-5} per °C.

8.5.2.5 Thermal and shrinkage coefficient for low-density concrete shall be determined for the type of low-density aggregate used.

8.5.3 Calculation of indirect action-effects

8.5.3.1 Consideration of the influence of shrinkage, temperature gradients and the settlement of columns, etc. is mandatory if their effect on the analysis is adverse and optional if their effect is beneficial. Where the effect is beneficial consideration of reduced stiffness as a result of cracking is optional, whilst this is obligatory where the effect is unfavorable. Any reduction in the indirect action-effects as a result of creep may be taken into account.

8.5.3.2 If structures are divided into sufficiently short sections by joints, the influence of creep, shrinkage and differences in temperature may generally be neglected.

8.6 Distribution of Flexural Reinforcement in Beams and One-way Slabs for Crack Control

8.6.1 Distribution of flexural reinforcement in two way slabs shall be as required by Section 13.4 of ACI 318M-89.

8.6.2 Flexural tension reinforcement shall be well distributed within maximum flexural tension zones of a member cross section as required by Clause 8.6.3.

8.6.3 When design yield strength f_y for tension reinforcement exceeds 300 MPa, cross sections of maximum positive and negative moment shall be so proportioned that the quantity Z given by:

$$Z = f_s \sqrt{d_c} A$$

does not exceed 30 MN/m for interior exposure and 25 MN/m for exterior exposure. Calculated stress in reinforcement at service load f_s (MPa) shall be computed as the moment divided by the product of steel area and internal moment arm. In lieu of such computations, f_s may be taken as 60 percent of specified yield strength f_y .

8.6.4 Provisions of Clause 8.6.3 may not be sufficient for structures subject to very aggressive exposure or designed to be water tight. For such structures, special investigations and precautions are required.

8.6.5 Where flanges of T-beam construction are in tension, part of the flexural tension reinforcement shall be distributed over an effective flange width equal to 1/10 the span. If the effective flange width exceeds 1/10 the span, some longitudinal reinforcement shall be provided in the outer portions of the flange.

8.6.6 If the depth of a web exceeds 1 m longitudinal skin reinforcement having a total area equal to at least 10 percent of the area of the flexural tension reinforcement shall be placed near the side faces of the web and distributed in the zone of flexural tension with a spacing not more than the web width, nor 300 mm. Such reinforcement may be included in strength computations only if a strain compatibility analysis is made to determine stresses in the individual bars or wires. The total area of longitudinal skin reinforcement in both faces need not exceed one half of the required flexural tensile reinforcement.

8.7 Shrinkage and Temperature Reinforcement

8.7.1 Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement shall be provided in structural slabs where the flexural reinforcement extends in one direction only.

8.7.1.1 Shrinkage and temperature reinforcement shall be provided in accordance with either Sub-clause 8.7.2 or 8.7.3.

8.7.2 Deformed reinforcement conforming to IPS-C-CE-200 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

8.7.2.1 Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than 0.0014:

- a) Slabs where grade 300⁽¹⁾ deformed bars are used 0.0020.
- b) Slabs where Grade 400⁽¹⁾ deformed bars or welded wire fabric (plain or deformed) are used 0.0018.
- c) Slabs where reinforcement with yield stress exceeding 400 MPa measured at a yield strain of 0.35 percent is used $\frac{0.0018 \times 60,000}{f_y}$

8.7.2.2 Shrinkage and temperature reinforcement shall be spaced not farther apart than five times the slab thickness, nor 500 mm.

8.7.2.3 At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the specified yield strength f_y in tension.

8.7.3 Prestressing tendons conforming to IPS-M-CE-165 used for shrinkage and temperature reinforcement shall be provided in accordance with the following:

8.7.3.1 Tendons shall be proportioned to provide a minimum average compressive stress of 1 MPa on gross concrete area using effective prestress, after losses, in accordance with Chapter 18 of ACI 318M-89.

8.7.3.2 Spacing of tendons shall not exceed 2 m.

8.7.3.3 When spacing of tendons exceeds 1.4 meter, additional bonded shrinkage and temperature reinforcement conforming to Sub-clause 8.7.2 shall be provided between the tendons at slab edges extending from the slab edge for a distance equal to the tendon spacing.

8.7.3.4 Spacing of tendons in seismic zones shall be according to Chapter 21 of ACI 318M-89.

8.7.4 Tie reinforcement

8.7.4.1 All nonprestressed bars shall be enclosed by lateral ties, at least $\phi 10$ in size for longitudinal bars $\phi 30$ or smaller, and at least $\phi 15$ in size for $\phi 35$, $\phi 45$, $\phi 55$ bars, and bundled longitudinal bars. Deformed wire or welded wire fabric of equivalent area is allowed.

8.7.4.2 Vertical spacing of ties shall not exceed 16 longitudinal bar diameters, 48 tie bar or wire diameters, or least dimension of the compression member.

8.7.4.3 Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle of not more than 135° and no bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie may be used.

1) For designation of grades refer to ASTM A615 m-88.

8.7.4.4 Ties shall be located vertically not more than $\frac{1}{2}$ a tie spacing above the top of footing or slab in any story, and shall be spaced as provided herein to not more than $\frac{1}{2}$ a tie spacing below the lowest horizontal reinforcement in slab or drop panel above.

8.7.4.5 Where beams or brackets frame from four directions into a column, ties may be terminated not more than 80 mm below the lowest reinforcement in shallowest of such beams or brackets.

8.7.4.6 Ties for seismic regions shall be designed according to Chapter 21 of ACI 318M-89.

9. SHEAR AND TORSION

9.1 General

The design analysis of shear reinforcement shall not take into account the compressive strength of the concrete.

9.1.1 Torsion

- 1) A check for torsion moments in beams, T-beams, etc., need not be carried out unless such moments are required for structural stability.
- 2) The torsional stiffness of beams may be neglected for the purpose of the analysis. If it is taken into account, however, allowance shall be made for the fact that in the transition from one state¹⁾ to another, cracking causes a greater decrease in torsional stiffness than in bending stiffness.

If the analysis does not take account of torsional stiffness, the torsion moments and their transmission to the supports shall instead be taken into account by making suitable reinforcement detailing provisions.

1) Notes:

State 1: Is the state of reinforced concrete assuming full contribution of the concrete in the tension zone.

State 2: Is the state of reinforced concrete where the contribution of the concrete in the tension zone is neglected.

9.1.2 Shear forces

- 1) The shear forces which play a key role in the calculation of shear stresses and bond stresses in buildings may be determined on the assumption that all spans are fully loaded, any continuity of spans or any restraint being taken into account.
- 2) Where openings, recesses, sharp increases in depth, etc., result in a relatively large local reduction in cross section, the shear forces acting in this reduced cross section shall be calculated on the assumption of the least favorable loading conditions in that section.

9.2 Buckling

9.2.1 The check for safety against buckling shall be based on an assumption of coexistent skew bending and longitudinal compression, in which the buckling configuration is generally free of torsion. The accidental eccentricities in two direction, shall be calculated separately for both main axes and shall be taken into account in combination with the design eccentricity.

9.3 Shear Strength

9.3.1 Design of cross sections subject to shear shall be based on:

$$V_u \leq \phi V_n$$

Where:

V_u is factored shear force at section considered and,
 V_n is nominal shear strength computed by:
 $V_n = V_c + V_s$

Where:

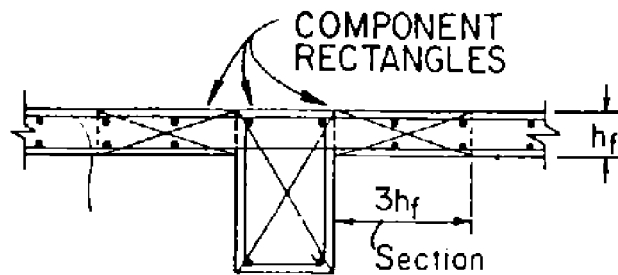
V_c is nominal shear strength provided by concrete and,
 V_s is nominal shear strength provided by shear reinforcement.

9.3.2 For shear strength provided by concrete for nonprestressed and prestressed members reference is made to Section 11.3 and 11.4 of ACI 318M-89 respectively and for shear strength provided by shear reinforcement to Section 11.5.6 of the same code.

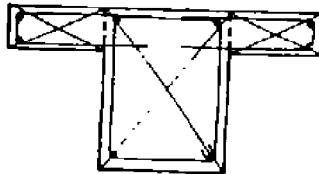
9.4 Combined Shear and Torsion Strength for Non-prestressed Members with Rectangular or Flanged Sections

9.4.1 Torsion effects shall be included with shear and flexure where factored torsional moment T_u exceeds $\phi[(\sqrt{f'_c} / 20) \Sigma x^2 y]$. Otherwise, torsion effects may be neglected. (For calculation of $\Sigma x^2 y$ see Fig. 2.)

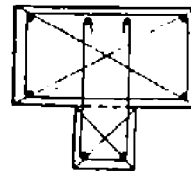
9.4.2 For members with rectangular or flanged sections, the sum $\Sigma x^2 y$ shall be taken for the component rectangles of the section, but the overhanging flange width used in design shall not exceed 3 times the flange thickness.



a)



b)



c)

COMPONENT RECTANGLES FOR THE CALCULATION OF $\Sigma x^2 y$
Fig. 2

9.4.3 Design of cross sections subject to torsion shall be based on

$$T_u \leq \phi T_n$$

Where:

T_u is factored torsional moment at section considered and,

T_n is nominal torsional moment strength computed by $T_n = T_c + T_s$ where T_c is nominal torsional moment strength provided by concrete in accordance with section 11.6.6 of ACI 318M-89, and

T_s is nominal torsional moment strength provided by torsion reinforcement in accordance with Section 11.6.9 of the same code.

9.5 Shear Friction

9.5.1 Where it is appropriate to consider shear transfer across a given plane, such as: an existing or potential crack, an interface between dissimilar materials, or an interface between two concretes cast at different times, provisions of Section 11.7 of ACI 318M-89 shall be applied.

9.5.2 Shear-friction reinforcement shall be appropriately placed along the shear plane and shall be anchored to develop the specified yield strength on both sides by embedment, hooks, or welding to special devices.

9.6 Special Provisions for Deep Flexural Members

9.6.1 For members with ρ_n / d less than 5 that are loaded on one face and supported on the opposite face so that compression struts can develop between the loads and supports, provisions of Section 11.8 of ACI 318M-89 shall apply.

9.6.2 Adequate anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment; such as sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which tension reinforcement is not parallel to compression face.

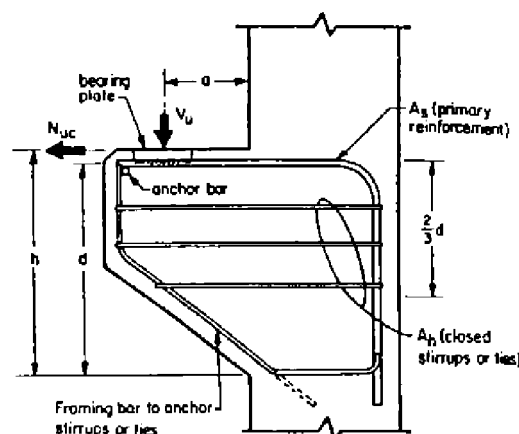
9.7 Special Provisions for Brackets and Corbels

9.7.1 For brackets and corbels with a shear span-to-depth ratio a/d not greater than unity, and subject to a horizontal tensile force N_{uc} not larger than V_u , provisions of Section 11.9 of ACI 318M-1989 shall apply. Distance d shall be measured at face of support. (See Fig. 3.)

9.7.2 Section at face of support shall be designed to resist simultaneously a shear V_u , a moment $[V_u a + N_{uc}(h-d)]$, and a horizontal tensile force N_{uc} .

9.8 Special Provisions for Walls

9.8.1 Design for shear forces perpendicular to face of wall shall be in accordance with provisions for slabs in Section 11.12 of ACI 318M-89. Design for horizontal shear forces in plane of wall shall be in accordance with Section 11.10.2 through 11.10.8 of ACI 318M-89. For design of shear reinforcement for walls refer to Section 11.10.9 of the same code.



NOTATION USED IN CLAUSE 9.7.1

Fig. 3

9.9 Transfer of Moments to Columns

9.9.1 When gravity load, wind, earthquake, or other lateral forces cause transfer of moment at connections of framing elements to columns, the shear resulting from moment transfer shall be considered in the design of lateral reinforcement in the columns.

9.9.2 Except for connection not part of a primary seismic load-resisting system that are restrained on four sides by beams or slabs of approximately equal depth, connections shall have lateral reinforcement not less than that required by Equation:

$$A_v = \frac{b_w s}{3 f_y}$$

Where b_w and s are in millimeters, within the column for a depth not less than that of the deepest connection of framing elements to the columns.

9.10 Special Provisions for Slabs and Footings

9.10.1 The shear strength of slabs and footings in the vicinity of columns, concentrated loads, or reactions is governed by the more severe of two conditions:

9.10.1.1 Beam action where each critical section to be investigated extends in a plane across the entire width. For beam action the slab or footing shall be designed in accordance with Sections 11.1 through 11.5 of ACI 318M-89.

9.10.1.2 Two-way action where each of the critical sections to be investigated shall be located so that its perimeter b_o is a minimum but need not approach closer than $d/2$ to:

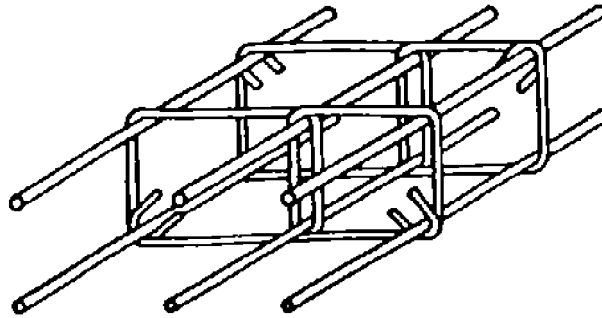
- a) edges or corners of columns, concentrated loads, or reaction areas, or,
- b) changes in slab thickness such as edges of capitals or drop panels.

For two-way action the slab or footing shall be designed in accordance with references stated in Clause 8.10.2.

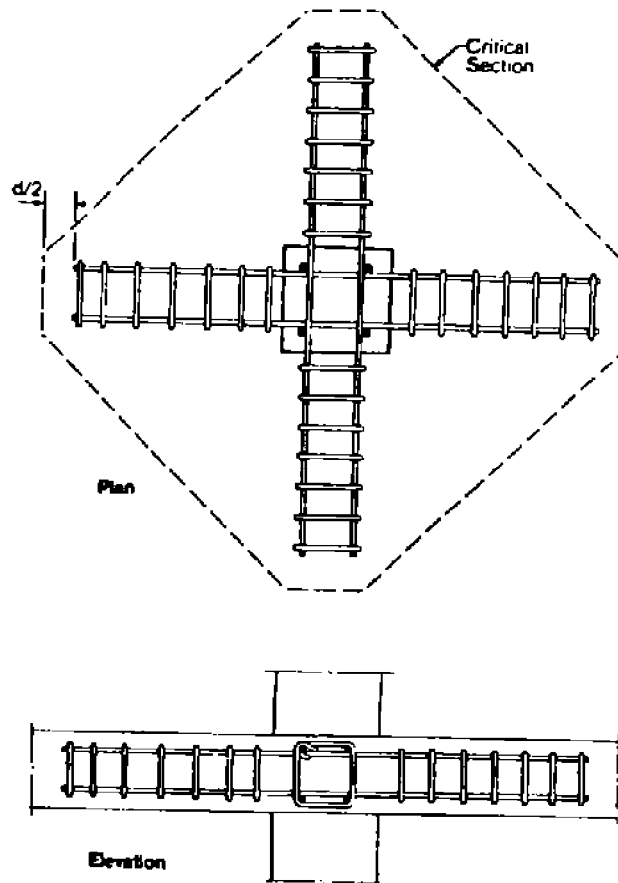
9.10.1.3 For square or rectangular columns, concentrated loads, or reaction areas, the critical sections may have four straight sides.

9.10.2 For additional special provisions and equations reference is made to Sections 11.12.2 through 11.12.6 of ACI 318M-89.

9.10.3 Shear reinforcement consisting of bars or wires can be used in slabs provided that it is well anchored. The anchorage detail for slabs and slab-column joints shall be as shown in Fig. 4 (a, b, c).



a) Slab Stirrups

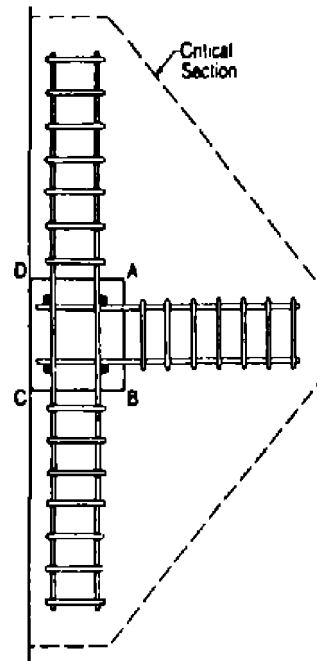


b) Arrangement of stirrup shear reinforcement, interior column

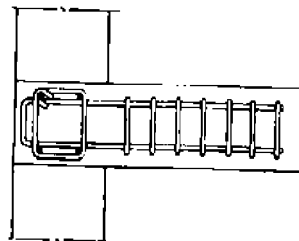
ANCHORAGE DETAILS

Fig. 4

(to be continued)



PLAN



Elevation

c) Arrangement of stirrup shear reinforcement, edge column

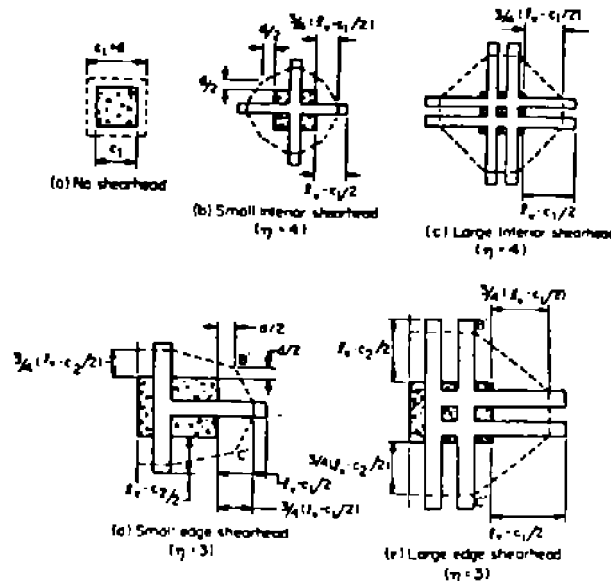
ANCHORAGE DETAILS

Fig. 4

9.10.4 Each shearhead shall consist of steel shapes (steel I or channel-shaped sections) fabricated by welding with a full penetration weld into identical arms at right angles. Shearhead arms shall not be interrupted within the column section.

9.10.4.1 A shearhead shall not be deeper than 70 times the web thickness of the steel shape.

9.10.4.2 The critical slab section for shear shall be perpendicular to the plane of the slab and shall cross each shearhead arm at three-quarters the distance $[l_v - (c_1 / 2)]$ from the column face to the end of the shearhead arm. (Fig. 5)



LOCATION OF CRITICAL SECTION DEFINED IN SUB-CLAUSE 9.10.4.2
Fig. 5

10. DEVELOPMENT AND SPLICES OF REINFORCEMENT

10.1 Development of Reinforcement-General

Calculated tension or compression in reinforcement at each section of reinforced concrete members shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only.

10.1.1 The values of $\sqrt{f'_c}$ used for calculation in this clause shall not exceed 0.69 MPa.

10.2 Development of Deformed Bars and Deformed Wire in Tension

10.2.1 Development length for deformed bars and deformed wire in tension shall be according to Section 12.2.1 through 12.2.5 of ACI 318M-89.

10.2.2 Product of the basic development length of 12.2.2 as modified by Section 12.2.3 through 12.2.5 shall not be less than 300 mm.

10.3 Development of Deformed Bars in Compression

10.3.1 Development length for deformed bars in compression shall be according to Section 12.3.1 through 12.3.3.2 of ACI 318M-89.

10.4 Development of Bundled Bars

10.4.1 Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20 percent for three-bar bundle, and 33 percent for four-bar bundle.

10.5 Development of Standard Hooks in Tension

10.5.1 Development length for deformed bars in tension terminating in a standard hook shall be computed according to Section 12.5 of ACI-318M-89.

10.5.2 Basic development Length ℓ_{hb} for a hooked bar with f_y equal to 400 MPa shall be $100 d_b \sqrt{f_c}^{(1)}$.

10.6 Mechanical Anchorage

10.6.1 Any mechanical device capable of developing the strength of reinforcement without damage to concrete is allowed as anchorage.

10.6.2 Test results showing adequacy of such mechanical devices shall be presented to the AR⁽²⁾.

10.6.3 Development of reinforcement may consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the point of maximum bar stress and the mechanical anchorage.

10.7 Development of Welded Deformed Wire Fabric in Tension

10.7.1 Development length of welded deformed and plain wire fabric in tension shall be according to Sections 12.7 and 12.8 of ACI 318M-89.

10.8 Development of Prestressing Strand

10.8.1 Development of pretensioning strands shall be according to Section 12.9.1 through 12.9.3 of ACI 318M-89.

10.9 Development of Flexural Reinforcement-General

10.9.1 Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

10.9.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent.

10.9.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the effective depth of member or $12 d_b$, whichever is greater, except at supports of simple spans and at free end of cantilevers.

10.9.4 Continuing reinforcement shall have an embedment length not less than the development length ℓ_d beyond the point where bent or terminated tension reinforcement is no longer required to resist flexure.

10.9.5 Flexural reinforcement shall not be terminated in a tension zone unless one of the following conditions is satisfied:

10.9.5.1 Shear at the cutoff point does not exceed two-thirds that permitted, including shear strength of shear reinforcement provided.

10.9.5.2 Stirrup area in excess of that required for shear and torsion is provided along each terminated bar or wire over a distance from the termination point equal to three-fourths the effective depth of member. Excess stirrup area A_v shall be not less than $0.4 b_w s / f_y$. Spacing s shall not exceed $d / 8 \beta_b$ where β_b is the ratio of area of reinforcement cut off to total area of tension reinforcement at the section.

1) Constant carries unit of N/mm.

2) AR = Authorized Representative of the Owner.

10.9.5.3 For ϕ 35 bar and smaller, continuing reinforcement provides double the area required for flexure at the cutoff point and shear does not exceed three-fourths that permitted.

10.9.6 Adequate anchorage shall be provided for tension reinforcement in flexural members where reinforcement stress is not directly proportional to moment, such as: sloped, stepped, or tapered footings; brackets; deep flexural members; or members in which tension reinforcement is not parallel to compression face.

10.10 Development of Positive Moment Reinforcement

10.10.1 At least one-third the positive moment reinforcement in simple members and one-fourth the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm.

10.10.2 When a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by Clause 10.10.1 shall be anchored to develop the specified yield strength f_y in tension at the face of support.

10.11 Development of Negative Moment Reinforcement

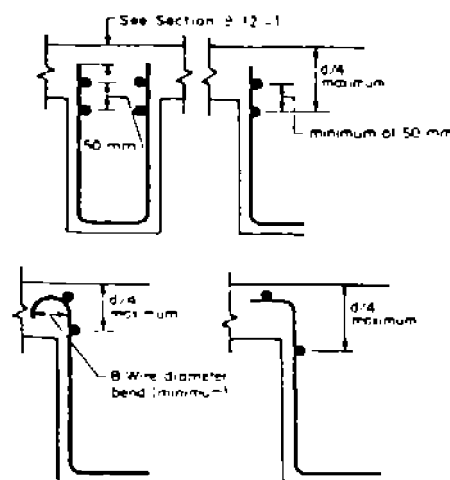
10.11.1 Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

10.11.2 Negative moment reinforcement shall have an embedment length into the span as required by Sub-Clauses 10.1 and 10.9.3.

10.11.3 At least one-third the total tension reinforcement provided for negative moment at a support shall have an embedment length beyond the point of inflection not less than effective depth of member, $12 d_b$, or one-sixteenth the clear span, whichever is greater.

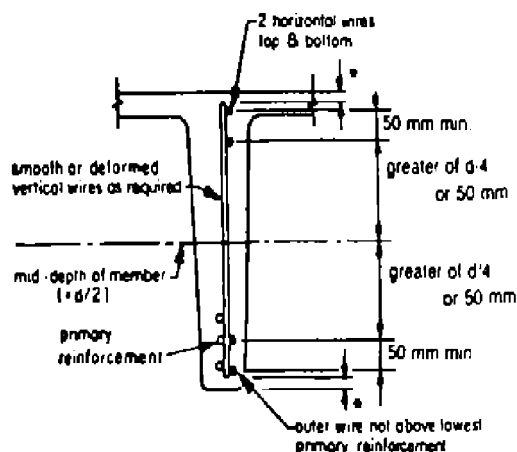
10.12 Development of Web Reinforcement

10.12.1 Web reinforcement shall be carried as close to compression and tension surfaces of member as cover requirements and proximity of other reinforcement will permit. For further detail refer to Section 12.13 of ACI-318M-89 and Figs. 5 and 6 of this Standard.



ANCHORAGE IN COMPRESSION ZONE OF WELDED SMOOTH WIRE FABRIC U-STIRRUPS

Fig. 6



* See Section 10.12.1.

ANCHORAGE OF SINGLE LEG WELDED WIRE FABRIC SHEAR REINFORCEMENT

Fig. 7

10.13 Splices of Reinforcement-General

10.13.1 Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the AR.

10.14 Splices of Deformed Bars and Deformed Wire in Tension

10.14.1 Minimum length of lap for tension lap splices shall be as required for Class A, or B, splice, but not less than 300 mm, where:

Class A Splice.

Class B Splice.

Where l_d is the tensile development length for the specified yield strength f_y in accordance with Clause 10.2, without the modification factor of excess reinforcement.

10.15 Splices of Deformed Bars in Compression

10.15.1 Compression lap splice length shall be $0.07 f_y d_b$, for f_y of 400 MPa or less, or $(0.13 f_y - 24)d_b$ for f_y greater than 400 MPa, but not less than 300 mm. For f'_c less than 20 MPa, length of lap shall be increased by one-third.

10.16 Special Splice Requirements for Columns

10.16.1 Lap splices, butt welded splices, mechanical connections, or end-bearing splices shall be used with the limitations of 10.16.2 through 10.16.4. A splice shall satisfy requirements for all load combinations for the column.

10.16.2 Lap splices in columns

10.16.2.1 Where the bar stress due to factored loads is compressive, lap splices shall conform to 10.15.1 and, where applicable, to 10.16.2.4 or 10.16.2.5.

10.16.2.2 Where the bar stress due to factored loads is tensile and does not exceed $0.5 f_y$ in tension, lap splices shall be Class B tension lap splices if more than one-half of the bars are spliced at any section, or Class A tension lap splices if half or fewer of the bars are spliced at any section and alternate lap splices are staggered by .

10.16.2.3 Where the bar stress due to factored loads is greater than $0.5 f_y$ in tension, lap splices shall be Class B tension lap splices.

10.16.2.4 In tied reinforced compression members, where ties throughout the lap splice length have an effective area not less than $0.0015 h_s$, lap splice length may be multiplied by 0.83, but lap length shall not be less than 305 mm. Tie legs perpendicular to dimension h shall be used in determining effective area.

10.16.2.5 In spirally reinforced compression members, lap splice length of bars within a spiral may be multiplied by 0.75, but lap length shall not be less than 305 mm.

10.16.3 Welded splices or mechanical connectors in columns

Welded splices or mechanical connectors in columns shall meet the requirements of Section 12.14.3.3 or 12.14.3.4 of ACI 318M-89.

10.16.4 End bearing splices in columns

End bearing splices complying with Section 12.16.4 of ACI 318-89 may be used for column bars stressed in compression, provided the splices are staggered or additional bars are provided at splice locations. The continuing bars in each face of the column shall have a tensile strength, based on the specified yield strength f_y , not less than $0.25 f_y$ times the area of the vertical reinforcement in that face.

10.17 For splices of welded deformed and plain wire fabric in tension refer to Sections 12.18 and 12.19 of ACI-318M-89, respectively.

11. PLAIN CONCRETE DESIGN

11.1 General Requirements

11.1.1 Plain concrete shall not be used for structural members where special design considerations are required for earthquake or blast, unless explicitly permitted by the legally adopted general building code.

11.2 Use of Plain Concrete

Plain concrete shall be limited to members that are continuously supported by soil or supported by other structural members capable of providing continuous vertical support, or where arch action assures compression under all conditions of loading.

11.2.1 Plain concrete for compression members, other than arches or walls, shall be limited to pedestals.

11.3 Design

11.3.1 All plain concrete should be designed in accordance with ACI-318. 1 RM-89.

12. PRESTRESSED CONCRETE DESIGN

For general information refer to Chapter 18 of ACI 318M-89.

13. DESIGN OF SHELLS AND FOLDED PLATE MEMBERS

For general information refer to Chapter 19 of ACI 318M-89.

d

$1.0 d$

$1.3 d$

APPENDICES

APPENDIX A DESIGN OF WALLS

A.1 General

A.1.1 Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

A.1.2 Walls subject to axial loads shall be designed in accordance with Clauses A.1, A.2 and either A.3 or A.4.

A.1.3 Design for shear shall be in accordance with Clause 9.8.1.

A.1.4 Unless demonstrated by a detailed analysis, horizontal length of wall to be considered as effective for each concentrated load shall not exceed center-to-center distance between loads, nor width of bearing plus four times the wall thickness.

A.1.5 Compression members built integrally with walls shall conform to Section 10.8.2 of ACI 318M-89.

A.1.6 Walls shall be anchored to intersecting elements such as floors, roofs, or to columns, pilasters, buttresses, and intersecting walls, and footings.

A.1.7 Quantity of reinforcement and limits of thickness required by Clauses A.2 and A.4 may be waived where structural analysis shows adequate strength and stability.

A.1.8 Transfer of force to footing at base of wall shall be in accordance with IPS-E-CE-120.

A.2 Minimum Reinforcement

A.2.1 Minimum vertical and horizontal reinforcement shall be in accordance with Clause A.2.2 and A.2.3 unless a greater amount is required for shear by Clause 9.8.

A.2.2 Minimum ratio of vertical reinforcement area to gross concrete area shall be:

- a) 0.0012 for deformed bars not large than $\phi 15$ with a specified yield strength not less than 1400 MPa, or,
- b) 0.0015 for other deformed bars, or,
- c) 0.0012 for welded wire fabric (smooth or deformed) not larger than W31 or D31.

A.2.3 Minimum ratio of horizontal reinforcement area to gross concrete area shall be:

- a) 0.0020 for deformed bars not larger than $\phi 15$ with a specified yield strength not less than 400 MPa, or,
- b) 0.0025 for other deformed bars, or,
- c) 0.0020 for welded wire fabric (smooth or deformed) not larger than W31 or D31.

A.2.4 Walls more than 250 mm thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

- a) One layer consisting of not less than one-half and not more than two-thirds of total reinforcement required for each direction shall be placed not less than 50 mm nor more than one-third the thickness of wall from exterior surface.
- b) The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than 20 mm nor more than one-third the thickness of wall from interior surface.

A.2.5 Vertical and horizontal reinforcement shall not be spaced further apart than three times the wall thickness, nor 500 mm.

A.2.6 Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

A.2.7 In addition to the minimum reinforcement required by Clauses A.2.1 and A.2.2 not less than two $\phi 15$ bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the corners of the openings but not less than 600 mm.

A.3 Walls Designed as Compression Members

Except as provided in Clause A.4 walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with provisions of Sections 10.2, 10.3, 10.10, 10.11, 10.12, 10.15 of ACI-318M-89, and Clauses A.1 and A.2 of Appendix A.

A.4 Empirical Design Method

A.4.1 Walls of solid rectangular cross section may be designed by the empirical provisions of Clause A.4 if resultant of all factored loads is located within the middle-third of the overall thickness of wall and all limits of A.1, A.2 and A.4 are satisfied.

A.4.2 Design axial load strength ϕP_{nw} of wall satisfying limitations of Clause A.4.1 shall be computed by Eq. (A.1) unless designed in accordance with Clause A.3.

$$\phi P_{nw} = 0.55 \phi f'_c A_g \left[1 + \frac{k_1 f'_c}{32 h} \right]$$

Where:

$\phi = 0.70$ and effective length factor k shall be:

For walls braced top and bottom against lateral translation and:

- a) Restrained against rotation at one or both ends (top and/or bottom) 0.8
- b) Unrestrained against rotation at both ends 1.0

For walls not braced against lateral translation 2.0

A.4.3 Minimum thickness of walls designed by empirical design method

A.4.3.1 Thickness of bearing walls shall not be less than 1/25 the supported height or length whichever is shorter, nor less than 100 mm.

A.4.3.2 Thickness of exterior basement walls and foundation walls shall not be less than 190 mm.

A.5 Nonbearing Walls

A.5.1 Thickness of nonbearing walls shall not be less than 100 mm nor less than 1/30 the least distance between members that provide lateral support.

A.6 Walls as Grade Beams

A.6.1 Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of Sections 10.2 through 10.7 of ACI-318M-89. Design for shear shall be in accordance with provisions of Clause 9 of this Standard.

A.6.2 Portions of grade beam walls exposed above grade shall also meet requirements of Clause A.2 of Appendix A.

APPENDIX B

DESIGN OF PRECAST CONCRETE MEMBERS

B.1 Design

B.1.1 Design of precast members shall consider all loading and restraint conditions from initial fabrication to completion of the structure, including form removal, storage, transportation, and erection.

B.1.2 In precast construction that does not behave monolithically, effects at all interconnected and adjoining details shall be considered to assure proper performance of the structural system.

B.1.3 Effects of initial and long-time deflections shall be considered, including effects on interconnected elements.

B.1.4 Design of joints and bearings shall include effects of all forces to be transmitted, including shrinkage, creep, temperature, elastic deformation, wind, and earthquake.

B.1.5 All details shall be designed to provide for manufacturing and erection tolerances and temporary erection stresses.

B.2 Precast Wall Panels

B.2.1 Precast bearing and nonbearing walls shall be designed in accordance with provisions of Appendix A.

B.2.2 Where precast panels are designed to span horizontally to columns or isolated footings, the ratio of height to thickness shall not be limited, provided the effect of deep beam action, lateral buckling, and deflections are provided for in the design. See Section 10.7 of ACI 318M-89.

B.3 Details

B.3.1 All details of reinforcement, connections, bearing seats, inserts, anchors, concrete cover, openings, lifting devices, fabrication, and erection tolerances shall be shown on the shop drawings.

B.3.2 When approved by the AR, embedded items (such as dowels or inserts) that either protrude from concrete or remain exposed for inspection may be embedded while concrete is in a plastic state provided:

B.3.2.1 Embedded items shall not be required to be hooked or tied to reinforcement within plastic concrete.

B.3.2.2 Embedded items shall be maintained in correct position while concrete remains plastic.

B.3.2.3 Embedded items shall be properly anchored to develop required factored loads.

B.4 Identification and Marking

B.4.1 Each precast member or element shall be marked to indicate location in the structure, top surface, and date of fabrication.

B.4.2 Identification marks shall correspond to the placing plans.

B.5 Side Connection of Precast Members

B.5.1 If a floor, ramp, etc., is an element made of precast members placed side by side, appropriate measures shall be taken to ensure that there are no differences in the deflection at the joints as a result of the differences in the stress to which the members are subjected.

B.5.2 The dispersal of shear forces in the joints shall be verified without taking into account the tensile strength of the concrete, loads being assumed to be in their least favorable configuration.

APPENDIX C

COMPOSITE CONCRETE FLEXURAL MEMBERS

C.1 General

C.1.1 An entire composite member or portions thereof may be used in resisting shear and moment.

C.1.2 Individual elements shall be investigated for all critical stages of loading.

C.1.3 If the specified strength, unit weight, or other properties of the various elements are different, properties of the individual elements or the most critical values, shall be used in design.

C.1.4 In strength computations of composite members, no distinction shall be made between shored and un-shored members.

C.1.5 All elements shall be designed to support all loads introduced prior to full development of design strength of composite members.

C.1.6 Reinforcement shall be provided as required to control cracking and to prevent separation of individual elements of composite members.

C.1.7 Composite members shall meet requirements for control of deflections in accordance with Section 9.5.5 of ACI 318M-89.

(Eq. A.1)

C.2 Shoring

When used, shoring shall not be removed until supported elements have developed design properties required to support all loads and limit deflections and cracking at time of shoring removal.

C.3 Vertical Shear Strength

C.3.1 When an entire composite member is assumed to resist vertical shear, design shall be in accordance with requirements of Clause 9, monolithically cast member of the same cross-sectional shape.

C.3.2 Shear reinforcement shall be fully anchored into interconnected elements in accordance with Clause 10.12.

C.3.3 Extended and anchored shear reinforcement may be included as ties for horizontal shear.

C.4 Horizontal Shear Strength

C.4.1 In a composite member, full transfer of horizontal shear forces shall be assured at contact surfaces of interconnected elements.

C.4.2 Unless calculated in accordance with Clause C.4.3, design of cross sections subject to horizontal shear shall be based on:

$$Vu \leq \phi V_{nh}$$

Where:

V_u is factored shear force at section considered and
 V_{nh} is nominal horizontal shear strength in accordance with the following.

C.4.2.1 When contact surfaces are clean, free of laitance, and intentionally roughened, shear strength V_{nh} shall not be taken greater than $0.55 b_v d$ in Newtons.

C.4.2.2 When minimum ties are provided in accordance with Clause C.5, and contact surfaces are clean and free of laitance, but not intentionally roughened, shear strength V_{nh} shall not be taken greater than $0.66 b_v d$ in Newton.

C.4.2.3 When minimum ties are provided in accordance with Clause C.5, and contact surfaces are clean, free of laitance, and intentionally roughened to a full amplitude of approximately 5 mm shear strength V_{nh} shall not be taken greater than $2.5 b_v d$ in Newtons.

C.4.2.4 When factored shear force V_u at section considered exceeds $\phi (2.5 b_v d)$, design for horizontal shear shall be in accordance with Clause 9.5.

C.4.3 Horizontal shear may be investigated by computing the actual change in compressive or tensile force in any segment, and provisions made to transfer that force as horizontal shear to the supporting element. The factored horizontal shear force shall not exceed horizontal shear strength ϕV_{nh} as given in Clauses C.4.2.1 through C.4.2.4 where area of contact surface A_c shall be substituted for $b_v d$.

C.4.4 When tension exists across any contact surface between interconnected elements, shear transfer by contact may be assumed only when minimum ties are provided in accordance with Clause C.5.

C.5 Ties for Horizontal Shear

C.5.1 When ties are provided to transfer horizontal shear, tie area shall not be less than that required by Section 11.5.5.3 of ACI 318M-89, and tie spacing shall not exceed four times the least dimension of supported element, nor 600 mm.

C.5.2 Ties for horizontal shear shall consist of single bars or wire, multiple leg stirrups, or vertical legs of welded wire fabric (smooth or deformed).

C.5.3 All ties shall be fully anchored into interconnected elements in accordance with Clause 10.12.