



भारतीय मानक ब्यूरो

(उपभोक्ता मामले, खाद्य एवं सार्वजनिक वितरण मंत्रालय, भारत सरकार)

BUREAU OF INDIAN STANDARDS

(Ministry of Consumer Affairs, Food & Public Distribution, Govt. of India)

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प्रारंभिक मसौदा

हमारा संदर्भ : सीईडी 2:2/टी-5

01 जनवरी 2025

तकनीकी समिति : सिमेन्ट और कंक्रीट विषय समिति, सीईडी 2

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महोदय/महोदया,

निम्नलिखित मानक का मसौदा संलग्न है:

प्रलेख संख्या	शीर्षक
सीईडी 2(27096)P	संरचनात्मक कंक्रीट – रीति संहिता (IS 456 का पाचवा पुनरीक्षण) का प्रारंभिक मसौदा, आईसीएस: 91.100.30

कृपया इस मसौदे की जाँच करें और इसमें और सुधार के संबंध में अपनी सम्मतियाँ साझा करें।

सम्मतियाँ भेजने की अंतिम तिथि: 31 जनवरी 2025

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धन्यवाद।

भवदीय

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सिविल अभियांत्रिकी विभाग

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भारतीय मानक ब्यूरो
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PRELIMINARY DRAFT

Our Reference: CED 2:2/T-5

01 January 2025

Technical Committee: Cement and Concrete Sectional Committee, CED 2

Addressed To: All the Members of CED 2, CED 2:1, CED 2:2, CED 2:1/P1, CED 2:2/P1, CED 2:2/P5, CED 2:2/P7, CED 2:2/P8, CED 2:2/P9, CED 2:2/P10, CED 46/P8 and Working Groups under CED 2

Dear Member,
Please find enclosed the following draft:

Doc No.	Title
CED 02(27096)P	Preliminary Draft Indian Standard Structural concrete – Code of practice (<i>fifth revision</i> of IS 456) ICS 91.100.30

Kindly examine the attached draft and forward your comments for further improvement.

Last Date for comments: 31 January 2025

Comments if any, may please be made in the enclosed format and emailed at ced2@bis.gov.in or sent at the above address. Additionally, comments may be sent online through the BIS e-governance portal, www.manakonline.in. In case no comments are received, or comments received are of editorial nature, kindly permit us to presume your approval for the above document.

Thanking you,

Yours faithfully,

Sd/-

Jitendra Kumar Chaudhary
Assistant Director/Scientist 'B'
Civil Engineering Department
Email: ced2@bis.gov.in

Encl: As above

FORMAT FOR SENDING COMMENTS ON THE DOCUMENT

[Please use A4 size sheet of paper only and type within fields indicated. Comments on each clause/sub-clause/ table/figure, etc, be stated on a fresh row. Information/comments should include reasons for comments, technical references and suggestions for modified wordings of the clause. **Comments through e-mail to ced2@bis.gov.in shall be appreciated.**]

Doc. No.: CED 2(27096)P

BIS Letter Ref: CED 2:2/T-5

Title: Preliminary Draft Indian Standard Structural concrete – Code of practice
(*fifth revision* of IS 456) ICS 91.100.30

Last date of comments: **31 January 2025**

Name of the Commentator/ Organization:

Sl No.	Clause/ Para/ Table/ Figure No. commented	Comments/ Modified Wordings	Justification of Proposed Change
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NOTE- Kindly insert more rows as necessary for each clause/table, etc

BURAU OF INDIAN STANDARDS*Preliminary Draft***STRUCTURAL CONCRETE – CODE OF PRACTICE***(Fifth revision of IS 456)**ICS 91.100.30**Cement and Concrete Sectional Committee, CED 02***FOREWORD***(Formal clauses to be added later)*

This standard was first published in 1953 under the title 'Code of practice for plain and reinforced concrete for general building construction' and subsequently revised in 1957. The code was further revised in 1964 and published under modified title 'Code of practice for plain and reinforced concrete', thus enlarging the scope of use of this code to structures other than general building construction also. The third revision was published in 1978, and it included limit state approach to design. Thereafter it was revised in 2000 and it included substantive clauses on acceptance criteria and durability of concrete. This is the fifth revision of the standard.

This fifth revision addresses advances made in the state-of-the-art internationally and in the state-of-the-practice in India in design and construction of structures built with structural concrete. In particular, it addresses the significant developments that have taken place over the last two decades in the field of concrete technology and durability of concrete structures. Also, over the last four decades, design engineering has seen major changes. Hence, this revision attempts to bring in an improved understanding of structural behaviour; some of these have been raised regularly during structural design, peer review and proof-checking of concrete structures in India.

This revision incorporates a number of significant changes. The salient changes in this revision are:

- 1) The title of the standard has suitably been modified to 'Structural Concrete – Code of Practice' which now addresses three types of concretes, namely plain, reinforced and prestressed concrete.
- 2) The provisions related to prestressed concrete had been added.
- 3) The basis of design is elaborated. The standard now requires concrete structures to be designed for six design criteria, namely safety, serviceability, durability, robustness, integrity and restorability. Each criteria is deemed to have been designed for, if the structural design is demonstrated to meet the requirements laid down in the standard.
- 4) High strength concretes and reinforcing steels have been considered in the drafting of the provisions in this revision.

- 5) In general, the basic principles are maintained. Axial force and bending moment capacities are based on strain, and shear force and torsional moment capacities on strength. But, the method of estimating moment capacities has been improved based on principles of mechanics, by considering the limiting strain values in steel and concrete, instead of the secondary compression failure strain of concrete.
- 6) All design strength values of concrete and steel are given in the form of expressions, now showing explicitly the partial safety factor γ_M for material.
- 7) In the limit state of safety, the definition of design strength under flexure of a concrete member is revised from that corresponding to secondary compression failure of the member to that corresponding to reaching limiting strains.
- 8) The concept of design service life has been introduced quantitatively.
- 9) Provisions related to serviceability, durability and fire resistance have been rationalized.

The provisions of this standard are applicable only to those structures whose global stability at the structure level is assured by examining their static and dynamic equilibriums, especially against overturning, sliding, subsidence and floatation. It is presumed that local instability at the member level will be eliminated by due proportioning of the members and through appropriate connections with adjoining members.

After the publication of this standard, IS 1343: 2012 'Prestressed Concrete – Code of Practice' will standard withdrawn.

In the formulation of this standard, assistance has been derived from the following publications:

- 1) BS EN 206:2013+A1:2016, Concrete. Specification, Performance, Production and Conformity, European Committee for Standardization (CEN), May 2013/16
- 2) EN 1992 (informally, EC 2), 2018, Draft Eurocode 2: Design of Concrete Structures, European Committee for Standardization (CEN), May 2018
- 3) ACI 318: 2019, Building Code Requirements for Reinforced Concrete, American Concrete Institute
- 4) AS 3600: 2018, Concrete structures, Standards Association of Australia
- 5) DIN 1045, 2012, Concrete, Reinforced and Prestressed Concrete Structures, Standard by Deutsches Institut Fur Normung E.V. (German National Standard)
- 6) CEB FIP Model Code 2020, Comite Euro International Du Beton
- 7) ISO 19338 Performance and Assessment Requirements for Design Standards on Structural Concrete, International Organization for Standardization, 2007

The composition of the technical committee responsible for the formulation of this standard is given in Annex G.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2: 2022 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

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Preliminary Draft Standard

Structural Concrete – Code of Practice

(Fifth revision of IS 456)

1 SCOPE

1.1 The provisions of this standard are applicable for design, construction, maintenance and assessment of plain, reinforced and prestressed concrete structures. Also, they are applicable to the concrete part of steel-concrete composite structures.

1.2 The provisions of this standard

- a) are applicable only to design of concrete structures made with normal weight aggregates, and not to structures made with lightweight or heavy weight aggregates; and
- b) shall be read in conjunction with the provisions of Part 2 of SP 7.

1.3 The provisions of this standard are applicable to the structural systems and members; using the materials and material considerations as specified hereunder.

1.3.1 *Structural Systems*

1.3.1.1 *Plain concrete system*

The provisions of this standard shall be applicable to design concrete structures where reinforcement, if provided, is ignored in the determination of their strengths.

1.3.1.2 *Reinforced concrete system*

The provisions of this standard shall be applicable to design of concrete elements of all structures, including but not limited to buildings, liquid retaining structures, bridges, retaining walls, industrial structures, dams and associated appurtenant structures, chimneys, bins, silos and bunkers, cooling towers, tunnels and foundation systems.

1.3.1.3 *Prestressed concrete system*

The provisions of this standard are applicable to design of the following prestressed concrete (PSC) structures:

- a) Post-tensioned structures, with internal or external prestressing, and
- b) Pre-tensioned structures.

1.3.1.4 *Precast concrete system*

The provisions of this standard are applicable only to design individual components of precast concrete structures, and not for design of the whole structure.

1.3.1.5 Steel-concrete composite systems

The provisions of this standard are applicable only to design the concrete component of the members of steel-concrete composite structures, and not for design of the whole structure. The design of all members, joints, connections, and their compatibility with the whole composite structure shall be demonstrated based on:

- a) the provisions given in the relevant standards, and
- b) the principles laid down in this standard.

1.3.1.6 Additional requirements for concrete structures

Additional requirements for design, construction, maintenance and assessment of concrete structures such as earthquake resistant structures, blast resistant structures, liquid retaining structures, cooling towers, bridges, chimneys, bins, silos and bunkers, hydraulic structures, coastal structures, shells, folded plates, arches, prefabricated concrete elements, precast concrete products (including concrete pipes and poles) and composite structures are provided in the most recent revisions of the respective standards. These standards shall be used in conjunction with the provisions of this standard.

1.3.2 Structural Materials

Standard materials shall be used so that the properties of the concrete structures constructed are consistent with the assumptions made and requirements laid down in this standard during planning, design and construction stages of the structure. The grades of materials for which the provisions of this standard are applicable shall conform to the requirements given in 7.

2 REFERENCES

The Indian Standards listed in Annex A contain provisions which through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated in Annex A.

3 TERMINOLOGY

For the purpose of this standard, the definitions given in IS 4845 and IS 6461 (Parts 1 to 12) shall generally apply. In addition, the terms defined hereunder shall apply.

3.1 Competent Authority — The authority that has legal jurisdiction to approve the plan, design and construction of the structure at the proposed site.

3.2 Competent Ground — Ground on which the structure is to be constructed. It should be free from hazards like liquefaction, slope failure, and vertical bearing failure, and can resist safely the loads imposed by the superstructure.

3.3 Design Basis Report (DBR) — A document authored by the owner of the structure (in consultation with the structural designer) which declares before the start of the process of design, all considerations namely design standards to be adopted, field investigations at the site of the project, materials, loads, mandatory load combinations, methods of analysis, methods of design and detailing, and special considerations.

3.4 Durability — The ability of the structure or structural element to assure that the degradation is limited such that performance of the structure remains above the requirement over a specified period of time in the relevant environment.

3.5 Durability Criteria — Under the action of design loads specified in this standard, capacity of a structure or a part thereof to

- a) sustain deterioration of materials under (physical, chemical and biological) environmental actions within specified limits, and
- b) serve through intended design life without any avoidable disruption.

3.6 Integrity Criteria — Under the action of design loads specified in this standard, capacity of a structure or a part thereof to

- a) have redundancy of such type that allows redistribution of loads to adjoining members;
- b) sustain sufficient strength and local deformation capacity in residual structure to carry additional loads induced, without exceeding the design limit states of inelastic deformability; and
- c) not collapse either globally or locally, if one member or a part of the structure exceeds the design limit states of safety.

3.7 Restorability Criteria — Under the action of design loads specified in this standard, capacity of a structure or a part thereof to

- a) sustain no damage in such structural elements, which are difficult to be stiffened and/or strengthened after damage;
- b) permit stiffening and/or strengthening in the other elements, which are amenable for modifications after damage, within a specified limit of inelastic deformation; and
- c) be brought back to original condition (if not better), if damaged (when permitted) under the action of design loads (especially under earthquake ground shaking load) considered in design.

3.8 Robustness Criteria — Under the action of design loads caused due to fire, blast and impact arising out of human errors (or any deliberate acts) specified in this standard, capacity of a structure or a part thereof to restrict

- a) internal stress-resultant to within specified strength limits; and
- b) inelastic deformations to within specified deformation limits.

3.9 Strength Criteria — Under the action of design loads specified in this standard, capacity of a structure or a part thereof to

- a) sustain only limited or no damage, including the effects of fatigue loading;
- b) possess sufficient strength to resist effects of lateral loads with sufficient margin; and
- c) undergo such deformation that results in neither partial collapse nor complete collapse.

3.10 Serviceability Criteria — Under the action of design loads (other than those arising from blast, impact and fire) specified in this standard, capacity of a structure or a part thereof to

- a) restrict stresses within the specified limits;
- b) sustain cracks of widths smaller than specified crack width limits; and
- c) undergo deformations (including deflections, rotations and vibrations) smaller than specified deformation limits.

3.11 Maintenance — Process of preserving the as-built condition of the structure.

3.12 Nominal Cover — The design depth of concrete cover. This dimension is used in design and indicated in construction drawings.

3.13 Quality Assurance — Rigorous, independent monitoring and correction undertaken by competent third party professionals or professional agencies (other than those involved in the quality control effort) to ensure that the design intent is actually realized in structures, which needs to be undertaken in each activity in the process of quality control.

3.14 Quality Control — Adopting and ensuring formal procedures and processes that are based on scientific principles and professionally agreed norms, in the process of development of a structure, including conceptualizing structural configuration; selecting construction materials; performing structural design; preparing good construction structural drawings; converting structural drawings into a structure at site; and undertaking post-construction activities that protect the health of the structure from deterioration.

3.15 Regular Structure — A structure that is composed of a regular grid of structural elements, which provide definite and direct load paths to transfer the forces (in horizontal and vertical planes) to the foundation with the least number of bends, and have uniform distributions in plan and elevation of structural system from the point of view of mass, stiffness, strength and deformability.

3.16 Reliable — Consistently good in safety, quality, and performance, which can be trusted.

3.17 Repair — Locally replace the structural material to restore or enhance the functionality (visually and otherwise) of the structure, and prevent deterioration of the structural members. This does not enhance the stiffness, strength and deformability of the structure and its structural members.

3.18 Retrofit — Restore or enhance the stiffness, strength and deformability of the structure and its structural members.

4 SYMBOLS

For the purpose of this standard, the symbols used in this section shall have the following meanings with respect to the structure or member or condition, unless otherwise defined elsewhere in this standard:

<i>Symbol</i>	<i>Meaning</i>
B	Overall width of a concrete beam or structural wall
b	Overall breadth of a concrete beam
D	Overall depth of a concrete beam or slab, or overall dimension of a rectangular column in the direction under consideration
d	Effective depth of a concrete beam
E_c	Modulus of elasticity (in MPa) of concrete, obtained from tests conducted as per IS 516
E_s	Modulus of elasticity (in MPa) of reinforcing steel bars, obtained from tests conducted as per IS 1599, IS 1608, 1786 and IS 2062
E_p	Modulus of elasticity (in MPa) of prestressing steel rods, wires or strands, obtained from tests conducted as per IS 6006 and IS 14268
f_{ck}	Characteristic compressive strength (in MPa) of concrete, obtained from tests conducted as per IS 516
f_y	Characteristic tensile strength of reinforcing steel bars, obtained from tests conducted as per IS 1786
f_p	Characteristic tensile strength of prestressing steel wires or strands, obtained from tests conducted as per IS 6006 and IS 14268
M_X	Bending Moment induced in a member about X-axis
M_Y	Bending Moment induced in a member about Y-axis
P	Axial Force induced in a member along Z-axis
T	Torsional Moment induced in a member about Z-axis
V_X	Shear Force induced in a member when bending about Y-axis
V_Y	Shear Force induced in a member when bending about X-axis
D	Chord displacements in a frame member
d	Relative to chord displacements in a frame member
γ_L	Partial safety factors for loads
γ_M	Partial safety factors for material strengths

5 BASIS OF DESIGN

5.1 The concrete structures designed and detailed as per the provisions of this standard shall possess (throughout its design service life) margins specified herein with respect to

- a) strength corresponding to the stress-resultants at cross-sections of members, when the structure is subjected to design load combinations as per **6.4**,
- b) deformations induced in members and in the structure, when the structure is subjected to design load combinations as per **6.4**, and
- c) durability.

The bases of design of concrete structures specified herein shall be applicable only to structures that are constructed on competent ground, and are stable under the action of imposed design loads.

5.1.1 *Characteristic Loads and Materials*

The design of structures shall be based on characteristic values of design loads, load combinations, and characteristic values of material properties specified in this standard.

5.1.2 *Principles of Mechanics*

Design of structures resting on competent ground shall be based on

- a) *Equilibrium Equations* - The structure as a whole or in parts shall satisfy six equilibrium equations, namely three force equilibrium equations and three moment equilibriums.
- b) *Constitutive Laws of Materials* - The stress-strain relation specified in this standard shall be applicable at each material point in the structure; and
- c) *Geometric Compatibility Conditions* - The geometric compatibility shall be upheld between the following:
 - 1) the structure and the supports, and
 - 2) the adjoining members within the structure.

5.1.3 *Methods of Analysis*

The stress-resultants in members, reactions at supports, and deformations in members and structure shall be obtained by the methods of structural analysis specified hereunder, unless otherwise stated.

- a) Static analysis shall be deemed to be sufficient when it is acceptable to represent all loads by equivalent static loads and when inertia forces are not significant. The following methods of static analysis shall be admissible:
 - 1) Three-dimensional linear static analysis in all structures;

- 2) Two-dimensional linear static analysis may suffice in regular structures, with direct load paths in the form of regular structural grid along all three directions; or
 - 3) Simplified estimates provided in **8.4** may be used in limited cases, as specified therein.
- b) Dynamic analysis shall be employed when the applied loads are vibratory in nature and inertia forces are significant. The following methods of dynamic analysis shall be admissible:
- 1) Three-dimensional linear dynamic analysis in all structures for estimating the effects for examining the strength and serviceability criteria (**5.2.1** and **5.2.2**);
 - 2) Three-dimensional nonlinear dynamic analysis in select structures for estimating the effects for examining the additional robustness and integrity criteria (see **5.2.4** and **5.2.5**). Both material and geometric nonlinearities shall be considered. The characteristics of nonlinear member models assumed therein (such as initial & degrading stiffnesses, cracking, yield & ultimate strengths, and cyclic hysteresis rules) shall be consistent with those of the materials used. These may be estimated analytically based on the properties of constituent materials, or ascertained experimentally.

5.1.4 Methods of Design

5.1.4.1 The internal resistance capacities (namely strength and deformation capacities) of members of concrete structures shall be estimated based on accepted theories, experimental evidences, professional experience and need for durability. Unless required otherwise,

- a) Strength of members shall be estimated as per the limit state method specified in **9.3**;
- b) Serviceability level stresses and deformations in a structure and its members shall be estimated as per the permissible stress method specified in **9.4**;
- c) Durability of members of a structure shall be examined as per the empirical method specified in **9.5**; and
- d) Robustness, integrity and restorability of a structure and its members shall be addressed as per the procedure specified in **9.6**, **9.7** and **9.8** respectively.

5.1.4.2 Strength design and serviceability check for concrete structures using new technology and systems may be accepted based on experimental studies also in case design and detailing are based on:

- a) Results from experimental investigations on prototype model(s) and appropriate analytical studies and satisfy the strength and serviceability requirements laid down in this standard, and

- b) The said experimental investigations and appropriate analytical studies mentioned in (a) above are duly approved by a technical committee of subject experts appointed by the competent authority.

5.1.5 Assessment and Strengthening of Existing Concrete Structures

When required by the statutory authority, the safety of an existing structure shall be assessed quantitatively as per the relevant Indian Standards. And, in case the structure is found to be structurally deficient, it shall be strengthened with least disturbance to the existing structure, towards enhancing the structural characteristics for which it is deficient, and demonstrated quantitatively that the strengthening intervention recommended is effective in eliminating the structural deficiency.

5.2 Criteria for Design

The six design criteria namely strength, serviceability, durability, robustness, integrity and restorability (see Fig. 1) shall be considered in design of structures, which are specified in terms of performance expectations on the structural behavior, and design requirements on the quantitative calculations. Design of concrete structures shall comply with the applicable criteria, as specified hereunder:

- a) All structures shall meet the strength, serviceability and durability criteria. And, select structures, as determined by the owner and/or competent authorities, shall meet the robustness, integrity and/or restorability criteria also; in such cases, the design basis report (DBR) of the structure shall reflect the same.
- b) A structure shall be designed on the basis of the quantitative strength criteria specified in **5.2.1**, and checked to meet the applicable ones of the remaining five criteria, namely quantitative serviceability, durability and robustness criteria specified in **5.2.2**, **5.2.3** and **5.2.4**, and qualitative integrity and restorability criteria specified in **5.2.5** and **5.2.6**.

The following three steps shall be adopted to design structures:

- a) Under the action of the most unfavourable combination of force and displacement loads produced on the structure and combined using partial safety factor γ_L for loads as per the load combinations specified in **6.4**, the following shall be evaluated by linear structural analysis:
 - 1) Stresses (namely normal and shear stresses) induced at critical cross-sections of the structure,
 - 2) Vibrations (namely relative displacements, relative velocities and absolute accelerations) induced at critical locations of the structure, and
 - 3) Deformations (namely translations and rotations) induced at all locations in members and the structure.

These shall be under the action of the loads specified in **6**. When doing so, the modulus of concrete shall be taken as the tangent of the design stress-strain curve at a stress of $(0.4f_{ck})/\gamma_M$.

- b) The design stress-resultant capacities of members (namely limiting design axial force P_L , shear forces V_{LX} and V_{LY} , bending moments M_{LX} and M_{LY} and torsional moment T_L capacities either individually or in combination) shall be estimated by the limit state method using principles of mechanics and the appropriate partial safety factors γ_M for materials specified in this standard.
- c) The stress-resultants demands at any section estimated in (a) above shall be less than the design stress-resultant capacities of members estimated in (b) above.

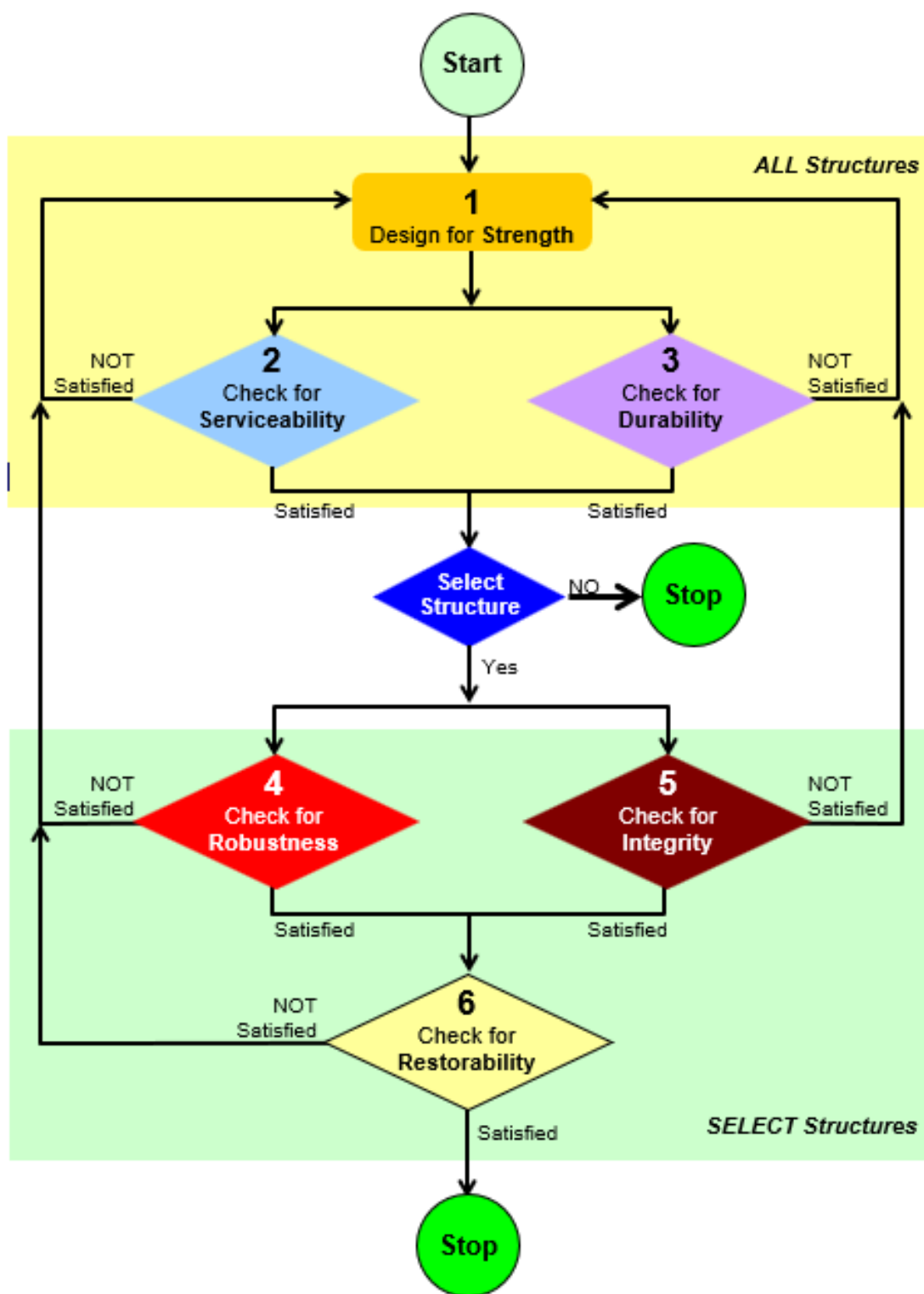


FIG. 1 SEQUENCE OF MEETING THE SIX CRITERIA FOR STRUCTURAL DESIGN OF CONCRETE STRUCTURES

5.2.1 Strength Criteria

5.2.1.1 Performance requirements

The structure (or a part thereof) shall possess sufficient:

- a) Strength to sustain only limited or no damage under the action of loads applied as per basic load combinations specified in Table 3,
- b) Strength to resist inelastic effects, under the action of loads applied as per accidental load combinations specified in Table 3, and
- c) Deformation capacity that results in no collapse of the structure, either partially or completely, under the action of loads applied as per extreme load combinations specified in Table 3.

5.2.1.2 Design requirements

The structure shall comply with the following:

- a) Strains in concrete and steel at a point in the critical cross-sections of a member shall be within the limiting values specified in **9.3.2.3** and **9.3.2.4** when estimating the limiting design axial force and limiting design bending moment capacities respectively, or a combination thereof; and
- b) Stresses in concrete and steel at a point in the critical cross-sections of a member shall be within the limiting values specified in **9.3.2.5** and **9.3.2.6** when estimating the limiting design shear force and limiting design torsional moment capacities, respectively.

5.2.1.3 The detailed requirements for strength criteria are given at **9.3**.

5.2.2 Serviceability Criteria

5.2.2.1 Performance requirements

The structure (or a part thereof) shall possess sufficient stiffness to ensure that:

- a) Stresses induced at the critical sections do not result in significant nonlinearities in concrete and steel, or initiate cracking at a material point level under fatigue loading,
- b) Crack widths do not expose reinforcement bars resulting in excessive corrosion, and
- c) Deformations do not damage the architectural elements and utilities,

under the action of loads applied as per basic load combinations given in Table 4.

5.2.2.2 Design requirements

The structure shall comply with the following:

- a) Stresses in concrete and steel at a point shall be within the permissible values specified in **9.4.2** when estimating the design axial force, design shear force, design bending moment and design torsional moment capacities;
- b) Crack widths shall be within the permissible values specified in **9.4.1.3**, and
- c) Deformations (namely relative translation, relative rotation and accelerations) in members or the structure shall be within the permissible values specified in **9.4.1**.

5.2.2.3 The detailed requirements for serviceability criteria are given at **9.4**.

5.2.3 Durability Criteria**5.2.3.1 Performance requirements**

The structure (and every part of it) shall sustain such deterioration of the materials under environmental actions (that is, physical, chemical and biological actions), which do not jeopardize its intended service, under the action of loads applied as per basic load combinations given in Table 4.

5.2.3.2 Design criteria

The structure shall comply with the following:

- a) Grade of concrete shall comply with the requirement specified in **7.1.2**;
- b) Cement grade shall comply with the requirement specified in **7.1.1.2**;
- c) Free water-cement ratio as specified in this standard; and
- d) Cover to reinforcing and prestressing steel shall comply with the requirement specified in **9.2.1.5**.

5.2.3.3 The detailed requirements for durability criteria are given at **9.5**.

5.2.4 Robustness Criteria**5.2.4.1 Performance requirements**

The structure (or a part thereof) shall sustain:

- a) Inelastic stress-resultants induced in it without losing its equilibrium and thereby without local or global collapse, and
- b) Deformation demands without fracture in steel or crushing of concrete,

under the action of loads applied as per accidental load combinations given in Table 5.

5.2.4.2 Design requirements

The structure shall comply with the following:

a) Blast Resistance

The requirements of IS 4991 (Part 1) shall be complied with for explosions above ground, and of IS 6922 for underground blasts while designing concrete structures.

b) Fire Resistance

Flame penetration and heat transmission without demands such that:

- 1) Transmission time of heat to steel reinforcement bars in the interior of the structural elements shall be within the limits specified in **9.6.1**.
- 2) Inelastic strength demands (namely axial force, bending moment, shear force and torsional moment, or their combinations thereof at all sections of members) shall be less than the corresponding reduced inelastic design capacities at elevated temperatures estimated as specified in **9.6.1**.
- 3) Inelastic deformation demands (namely relative inelastic translation at a location in the structure or relative inelastic rotation in a member) shall be less than the corresponding deformation capacities of members estimated as specified in **9.6.1** and
- 4) Overall response does not result in collapse, examined as specified in this standard (progressive collapse requirement).

c) Impact Resistance

Impact of the projectile on the structure shall be such that:

- 1) Inelastic strength demands (namely axial load, bending moment, shear force and torsional moment, or their combinations thereof at all sections of members) are less than the corresponding inelastic design capacities estimated as specified in **9.6.4**.
- 2) Inelastic deformation demands (namely relative inelastic translation at a location in the structure or relative inelastic rotation in a member) shall be less than the corresponding deformation capacities of members estimated as specified in **9.6.4**; and
- 3) Overall response does not result in collapse, examined as specified in this standard (progressive collapse requirement).

5.2.4.3 The detailed requirements for robustness criteria are given at **9.6**.

5.2.5 Integrity Criteria

5.2.5.1 Performance requirements

The structure (or a part thereof) shall possess such virtues of structural behaviour that it sustains:

- 1) No collapse, if it is a *Normal Structure*,
- 2) Some structural damage, if it is an *Important Structure*,
- 3) Limited structural damage, if it is a *Critical Structure*, and
- 4) No structural damage, if it is a *Special Structure*,

under the action of loads applied as per integrity load combinations given in Table 6. Here, the four structural categories mentioned above shall be considered as defined in IS 1893 (Part 1).

5.2.5.2 Design requirements

The structure shall comply with the following:

- 1) The structural system shall be simple with sufficient alternate load paths to safely transmit the inertia forces induced during ground shaking or ground settlement to the ground, without the irregularities specified in IS 1893 (Part 1).
- 2) The vertical members shall be of large cross-sectional area to provide at least a minimum initial lateral stiffness as specified in IS 1893 (Part 1).
- 3) The members shall be designed such that the overall lateral strength shall be as specified in IS 1893 (Part 1).
- 4) Select members shall possess deformation capacities (namely relative inelastic rotation at their critical cross-sections) such that the overall lateral deformability of the structure (relative inelastic displacement in it) meets the lateral deformation demand estimated as per IS 1893 (Part 1).

5.2.5.3 The detailed requirements for integrity criteria are given at **9.7**.

5.2.6 Restorability Criteria

5.2.6.1 Performance requirements

A structure shall be designed and constructed such that:

- a) When it is damaged under the action of loads applied as per extreme load combinations specified in Table 6, it can be brought back conveniently and speedily

to original condition (if not better) by structural retrofitting. Therefore, a structure (or a part thereof) shall be designed such that it sustains damage:

- 1) in select members as per capacity design requirements specified in IS 13920,
 - 2) at accessible critical locations within these select members,
 - 3) of the inelastic & ductile type as per capacity design requirements specified in IS 13920,
 - 4) of limited extent that can be retrofitted, and
 - 5) that can be retrofitted by techniques and materials available at the time of design.
- b) When it deteriorates under the action of aggressive environment, it can be brought back by structural repair to original condition (if not better). Therefore, a structure (or a part thereof) shall be:
- 1) Designed to sustain such levels of deterioration in materials, which can be subjected to structural repairs by the techniques and materials available at the time of design to undertake the needed structural repair in a timely manner, and
 - 2) Provided at the design stage itself with a maintenance schedule to undertake timely structural repairs on the structure,

and the deterioration controlled to within acceptable levels.

5.2.6.2 Design requirements

The structure shall comply with the following:

- a) It shall be ensured that only nominal repair will suffice (without any stiffening and/or strengthening of the member(s) and structure), after the design loads appear on the structure;
- b) The location of possible damage under design earthquake ground shaking shall be pre-determined to be at such locations that are accessible, through:
 - 1) Design (by proportioning the stiffness and strength of members), when designing a new structure, and
 - 2) Design (by proportioning the stiffness and strength of existing members being modified and/or of new structural members being added to the structure), when retrofitting an existing structure;
- c) The environment at the site of the structure shall be identified realistically, and the effects of chlorination, carbonation and sulphate attack shall be estimated accurately, and corrective actions undertaken as per **9.5.4.4.2.1, 9.5.5**; and

- d) A maintenance schedule shall be provided at the time of design, and be such that it is based on measurements made on the structure and its materials.

5.2.6.3 The detailed requirements for restorability criteria are given at **9.8**.

5.3 Process of Design

Structures shall be designed by:

- a) Prescriptive process, when the design basis report requires the structure to meet the requirements only three criteria, namely of strength, serviceability and durability; and
- b) Closed-loop process, when the design basis report requires the structure to meet the requirements all six criteria, namely of strength, serviceability, durability, robustness, integrity and restorability.

5.3.1 Prescriptive Process

This consists of a set of quantitative experience-based requirements specified in **9.1** to **9.5**. The designer shall comply with the steps in this process and render implicitly the structure as deemed to meet the intent of the standard.

5.3.2 Closed-Loop Process

This consists of a set of:

- a) Quantitative experience-based requirements specified in **9.1** to **9.5**, and
- b) Qualitative performance requirements specified in **9.6** to **9.8** in the form of procedures, complying with which establishes explicitly that the structure has met the intent of the standard.

5.4 Structural Systems

Structural systems adopted in construction of concrete structures are:

- a) Reinforced concrete structural systems
- b) Prestressed structural systems

Structures built with any of the above systems shall meet the design criteria, as applicable.

5.4.1 Reinforced Concrete Structural Systems

5.4.1.1 Reinforced concrete structures may be composed of any of the following basic structural systems:

- a) Moment frame system, and

- b) Structural wall system,
 - c) Braced frame system,
 - d) Advanced systems (such as tube, frame-tube, etc.),
- or a combination thereof.

5.4.1.2 While selecting a structural system for a RC structure, the following shall be considered:

- a) Structural wall systems perform better than moment frame systems;
- b) Extreme caution shall be exercised when adopting flat slab type flooring systems in concrete structures;
- c) Connections and joints shall never fail or enter inelastic behaviour, even though members may fail;
- d) Reasonable parity shall be ensured (see **13.4**) in the grades of concrete and reinforcing steel employed – combinations of high grade steel with low grade concrete shall not be admissible, while combinations of low grade steel with high grade concrete may be wasteful; and
- e) Construction technology employed shall be sensitive to the mass, age of loading on creep and shrinkage deflections and method of erection of the structure.

5.4.2 *Prestressed Concrete Structural Systems*

When prestressed structural systems are subjected to earthquake ground shaking, resulting in reversal of stresses during the imposed cyclic loading, their behaviour is questionable, and such structures are vulnerable to collapse. Design shall ensure that all details of reversal are examined with care and only when there is quantitative evidence of satisfactory behaviour, the use of such structures shall be permitted in earthquake prone areas, after due diligence by the statutory authority. To permit the use of prestressed concrete structures, the following shall be performed:

- a) Rigorous analytical studies,
- b) Detailed experimental studies on representative samples of the full-scale structures, sub-assemblages and members, along with all needed materials tests conducted alongside, and
- c) Development of provisions to be complied with to ensure compliance of the six design criteria specified in this standard.

5.4.2.1 *Methods of prestressing*

- a) Concrete structures shall be prestressed by either of the following methods:
 - 1) External prestressing method, and
 - 2) Unbonded and bonded internal prestressing method

- b) Both pre-tensioning and post-tensioning are permitted. But, when post-tensioning is undertaken using proprietary items, the efficacy shall be demonstrated through testing of these systems at a national laboratory in India.

5.4.2.2 Direction of prestressing

Prestressed Concrete structures shall be composed of the following members or a combination of these thereof:

- a) Horizontally prestressed structural members, or
- b) Vertically prestressed structural members.

Also, these prestressed concrete members may be combined with normal concrete members, like:

- a) Bridges composed of prestressed superstructure and the rest with normal members, and
- b) Buildings with only slabs prestressed.

5.4.2.3 Effects of prestressing

When choosing to *prestress* select components of concrete structures:

- a) Prestressing method employed shall be commensurate with strength of concrete on the day of prestressing, and its implications on construction sequence; and
- b) Differential deformations shall be considered in prestressed members (owing to prestressing) and those in normal members, owing to differential stiffness and strength of the prestressed members over those of the normal members, imposing compatibility of deformations at the interface of normal and prestressed members.

5.4.2.4 Inelasticity in prestressed Members

No inelasticity shall be permitted in prestressed members.

5.5 Quality Control and Quality Assurance

5.5.1 Relevant Indian Standards shall be complied with for materials, production, workmanship, maintenance and use of structure to realize the design objectives. Constructability, maintainability and usability (during design service life) shall be integral part of the process of design along with the usual activities of choosing materials, selecting suitable structural system, preparing accurate analytical models, performing structural design, and providing reinforcement detail.

5.5.2 Quality in construction shall be ensured through quality control (which is exercised by the construction agency), and quality assurance (which is exercised by an independent third party peer review agency).

5.5.3 The owner shall ensure that quality systems are in place to build safe, serviceable, sustainable, aesthetic and economical structures to adopt quality control and assurance.

5.6 Joints and Connections

5.6.1 Both cast in-situ and precast concrete structures shall be designed such that all member joints and connections shall remain elastic (under the action of the forces and moments transmitted through them under the action of design loads combined as per Table 3), except when stated otherwise in other relevant Indian Standards.

5.6.2 When joints and connections sustain inelasticity even after being designed for the design loads, the structure shall be provided with additional lateral restraint in the form of braces or structural walls made-by adequate moment connections or by a system of bracings.

5.7 Maintenance

5.7.1 *Types of Maintenance in Focus*

Maintenance shall be ensured of concrete structures to undertake the needed:

- a) Minimal structural repairs to prevent deterioration of structural members, and
- b) Detailed structural repairs to stop further deterioration of structural members.

The owners shall ensure that systems are in place to undertake maintenance of structures throughout the design service life of the structures.

5.7.2 *Actions of Maintenance*

The designer shall identify the actions to be taken during the design service life of the structure, to prevent its undue deterioration.

5.8 Loads and Materials

5.8.1 *Loads*

Concrete structures shall be designed to resist the effects of the relevant design loads represented by their characteristic values considered as per **6.3** and combined as per **6.4**. The loads to be considered shall be identified by the owner and documented in the design basis report.

5.8.2 Materials

Concrete structures shall be designed with available construction materials, whose engineering properties are established as per the relevant Indian Standards for the said purpose, and whose design properties shall be taken as per 7.

6 LOADS

Two types of loads shall be considered in the structural design of concrete structures, namely force loads and deformation loads, which are specified as characteristic loads in 6.1 and 6.2. Loads that cause more severe effects on concrete structures than those specified in 6.1 and 6.2 can be adopted in the design at the discretion of the owner.

a) *Special Considerations*

Special considerations shall be given to design and detailing of structures that are subjected to stress concentrations and fatigue, as applicable, arising out of all or a few of them.

b) *Design Basis Report*

All loads applicable to the design of concrete structures shall be declared in the design basis report.

6.1 Force Loads

The force loads to be considered in the design of concrete structures, as applicable shall be:

- a) Dead load,
- b) Imposed load,
- c) Snow load, including avalanches,
- d) Earth pressure load,
- e) Construction load,
- f) Prestress load,
- g) Wind load,
- h) Liquid pressure load, including hydrostatic, hydrodynamic and other liquid pressures,
- i) Machine vibration load,
- j) Blast load, and
- k) Impact load.

The applicable loads shall be specified explicitly in the design basis report of the design.

6.1.1 Dead Load

Dead load shall be considered as specified in IS 875 (Part 1) as a downward static force acting on the structure. The unit weights of concretes shall be taken as per Table 1 and of the others as per IS 875 (Part 1).

When the materials being used are different from those listed IS 875 (Part 1), the dead load of a structure shall be estimated using unit weights of the materials used in its construction.

Table 1 Unit Weight of Concrete made with different Aggregates
(Clause 6.1.1)

SI No.	Concrete with different aggregates	Unit Weight (kN/m ³)
(1)	(2)	(3)
i)	Light weight aggregate	10
ii)	Medium weight aggregate	16
iii)	Normal weight aggregate	24
iv)	Heavy weight aggregate	30
v)	Very Heavy weight aggregate	36

6.1.2 Imposed Load

Imposed Load shall be considered as specified in IS 875 (Part 2) for buildings, and as specified in relevant standards for highway and railway bridges, respectively, as a downward force, unless modified otherwise by the said standards.

6.1.2.1 Critical positioning of imposed load

Imposed load shall be placed on the structure at such locations that give the most unfavourable effect on it, with respect to the stress-resultant in focus. In the absence of detailed estimate of this effect, this standard requires consideration with respect to imposed load arrangement at least of the following:

a) When design imposed load is more than 0.75 times the design dead load, then:

- 1) Maximum positive bending moment near mid-span of a slab panel shall be assumed to occur when the full design imposed load is on the slab panel and on alternate slab panels; and
- 2) Maximum negative bending moment in the slab at a support shall be assumed to occur when the full design imposed load is on the entire slab system; and

b) When design imposed load is less than 0.75 times the design dead load, then:

- 1) Maximum positive bending moment near mid-span of a slab panel shall be assumed to occur when 75 percent of the full design imposed load is on the slab panel and on alternate slab panels;
- 2) Maximum negative bending moment in the slab at a support shall be assumed to occur when 75 percent of the full design imposed load is on the adjacent panels; and

In no case shall design bending moments be taken less than those occurring with full design live load on all slab panels.

The above considerations shall be valid for design of continuous beams also.

6.1.2.2 Reduction in imposed load

The Imposed Load can be reduced up to 50 percent for the purposes of design; such reductions shall not be permitted in the design of the following structures:

- a) Structures related to garages and places of public assembly, and
- b) Structures on whom the imposed load is 5 kN/m² or more.

The reduction in the column axial load arising from imposed load, shall be applicable depending on the number of storeys in the column, specified in **3.2** of IS 875 (Part 2), except when the load combination being considered includes earthquake load in it.

6.1.2.3 Imposed load in estimation of deflection owing to creep

When computing the deflections due to creep, only that portion of the live load shall be considered, which are present permanently on the structure.

6.1.2.4 Fatigue

Effect of fatigue induced owing to vehicular loads shall be considered as specified in IS 875 (Part 5) on the design strengths of the constituent materials, when the vibrating force imposes large number of cycles of oscillating stresses.

6.1.3 Snow Load

Snow load shall be considered as specified in IS 875 (Part 4) as a downward force acting on the structure arising from natural snow acting on it.

6.1.4 Earth Pressure Load

Earth pressure load shall be considered as specified in IS 875 (Part 5) for static actions and in IS 1893 (Parts 1 to 11) for dynamic actions arising from effects of earthquake ground shaking, as vertical and horizontal forces acting on the structure.

6.1.5 Prestress Load

Prestress force in prestress steel shall be considered as a concentrated force applied at the locations where the strands are anchored on the member being prestressed. When doing so, both short-term and long-term losses in prestress force as specified in **8.2.1.3** shall be considered.

6.1.6 Wind Load

Wind Load shall be considered as specified in IS 875 (Part 3) as force applied normal to the surface of the structure arising from natural wind acting on it. It shall be considered along such direction(s), which produces the worst effect on the structure.

6.1.7 Liquid Pressure Load

Liquid pressure load shall be considered as specified in IS 875 (Part 5) as a force applied normal to the surfaces of the structure arising from water or any other fluid acting on it statically or dynamically, as applicable, either from within or outside the structure.

6.1.8 Machine Vibration Load

Machine vibrations shall be considered as specified in IS 875 (Part 5) as concentrated forces applied at the location of the anchors of the machines, such that the worst effect is accounted for on the structure.

The load arising from machine vibrations shall be considered as:

- 1) 1.5 times the peak load, when equivalent static analysis is performed, unless detailed analyses show otherwise, and
- 2) Actual temporal history of the vibrating load, when dynamic analysis is performed.

Effect of fatigue mentioned in **6.1.2.4** shall be considered in the context of machine vibrations also.

6.1.9 Blast Load

Blast load shall be considered as specified in IS 4991 as a force imposed on the structure arising from a denotation above ground, on ground or underground, and as the consequent oscillations of the ground arising from it.

6.1.10 Impact Load

Impact load shall be considered as specified in IS 875 (Part 5) as a force applied in the direction of the impact on the structure arising from manmade action of:

- a) collision or sudden braking of a vehicle moving either on land or in water, or a crane moving on gantry inside the structure; and
- b) projectile impinging on it.

6.1.11 Construction Load

Construction load shall be considered as specified in IS 875 (Part 2) as a force applied on the structure during the construction stage, including considerations in design for:

- a) The unfinished state of the structure,
- b) The distinctly different early age properties of concrete,
- c) The temporary nature of some loads.
- d) The sequence of construction in multi-storey buildings (during shoring and re-shoring sequence of formwork for casting RC elements in the higher levels using the same formwork), which can be considerable.

The structural design of enabling works (like formwork) shall consider the loads arising during construction.

6.2 Deformation Loads

The deformation loads to be considered in the design of concrete structures are:

- a) Creep,
- b) Shrinkage,
- c) Elastic Shortening,
- d) Temperature – Ambient Air,
- e) Temperature – Fire,
- f) Earthquake Ground Shaking, including tsunamis, and
- g) Foundation Settlement.

These shall be taken as described in the sub-sections below. The applicable loads shall be specified explicitly in the design basis report of the design.

Effects of fluctuations in creep, shrinkage and temperature (ambient air) in RC structures shall be estimated and expansion joints be provided, if the largest plan dimension of the structure exceeds 45m, unless detailed calculations show otherwise. This effect can be ignored in the design of multi-storey RC buildings up to 15m in height, if the largest dimension does not exceed 45m in plan.

6.2.1 Creep

Creep shall be considered as specified in **7.1.4.4** as a time-dependant strain induced in the members of concrete structures.

6.2.2 Shrinkage

Shrinkage shall be considered as specified in **7.1.4.5** as a time-dependant strain induced in the members of concrete structures.

6.2.3 Elastic Shortening

Elastic Shortening shall be considered as axial compressive strain, which arise in the members of concrete structures from axial compression:

- a) Prestressed concrete members estimated on the adjoining members that are cast in-situ with them; and
- b) Vertical members of buildings that are 100m or more tall imposed as displacement on all adjoining members. The storey-wise construction of such buildings shall be considered in structural analysis.

6.2.4 Temperature – Ambient Air

Temperature (ambient air) effects shall be considered as specified in IS 875 (Part 5). The elongation, contraction and bending of members subjected to differential or total increase in temperatures shall be estimated as per **8.10.3**.

The effects of these member deformations on stress-resultants of adjoining members shall be estimated through structural analysis, and considered in structural design. Shortening of members of the supporting structure and their flexibility are the bases of structural analysis.

6.2.5 Temperature – Fire

Temperature (Fire) effects shall be considered as strains imposed on members. The prescriptive requirements of fire protection specified in **9.2.2.2** shall be met with. In the absence of this compliance, the effects of flames imposed on concrete members shall be estimated using time-dependant stress-strain curve considered suitably in nonlinear structural analysis.

6.2.6 Earthquake Ground Shaking

Earthquake ground shaking shall be considered as specified in IS 1893 (Parts 1 to 11) as a time-dependant displacement imposed at the base of structures along the three Cartesian directions.

6.2.7 Foundation Settlement

Foundation settlement shall be considered as specified in IS 1904 as flexibility of the soil and time-dependant soil consolidation related settlements in the stability calculations and safety-related structural analysis.

The settlement of foundations need not be considered, when the total instantaneous and time-dependant settlements (namely total and relative vertical deflections and rotations) do not exceed the permissible values specified in **9.4.1.6**.

6.3 Design Loads

6.3.1 Characteristic Loads

The force loads and the deformation loads described in **6.1** and **6.2** shall be considered as characteristic force loads and characteristic deformation loads, respectively.

6.3.2 Design Loads

In the design of concrete structures, design loads L_D shall be considered as given by:

$$L_D = L_C \gamma_L,$$

where

L_C = Characteristic load specified in **6.3.1**, and

γ_L = Partial safety factor appropriate for the load type and the design criteria being considered specified in Tables 3 to 6.

6.3.3 Partial Safety Factors γ_L for Loads

The partial safety factor γ_L for different design loads shall be as specified in Tables 3 to 6 for five design criteria. Where the consequences (such as loss of life and disruption of economy) of a structure is considered to be more serious under the action of a certain design load than those considered in this standard through γ_L , higher values of γ_L than those given in Tables 3 to 6 may be adopted in the design of the structure; the same shall be documented in the Design Basis Report.

6.4 Load Combinations

6.4.1 Three minimum sets of load combinations, namely:

- a) Set 1 Basic load combinations,
- b) Set 2 Accidental load combinations, and
- c) Set 3 Extreme load combinations.

shall be considered as specified in Table 2. Each of these loads shall be combined with the other loads, such that their worst effect on the structure is accounted for.

6.4.2 The load combinations specified above are the minimum combinations that need to be considered. Further:

- a) All load combinations are considered in design, which are likely to cause maximum effect on the structure. If new combinations are more critical than the minimum combinations specified in Tables 3 to 6, the same shall be considered.
- b) Additional load combinations may be specified by other Indian Standards when designing specific structures; even those shall be considered when designing those structures.

Table 2 Loads to be considered in the Three Sets of Combinations
(Clause 6.4.1)

Sl No.	Load	Three Sets of Load Combinations			
		Set 1 Basic Combination s	Set 2 Accidental Combination s	Set 3 Extreme Combination s	
A. FORCE LOADS					
1	Dead Load (DL)	Yes	Yes	Yes	
2	Imposed Load (IL)		Yes	Yes	
3	Snow Load (SL)		Yes		
4	Earth Pressure Load (EPL)		Yes	Where applicable	Where applicable
5	Prestress Load (PL)	Yes	Where applicable	Where applicable	
6	Wind Load (WL)		Yes		
7	Liquid Pressure Load (LPL)		Yes	Where applicable	Where applicable
8	Machine Vibration Load (MVL)		Yes		
9	Blast Load (BL)		Yes		
10	Impact Load (IPL)		Yes		
11	Construction Load (CL)		Yes		
B. DISPLACMENT LOADS					
1	Creep (CL)	Yes			
2	Shrinkage (ShL)	Yes			
3	Elastic Shortening (ESL)	Yes			
4	Temperature – Ambient Air (TAL)	Yes			
5	Temperature – Fire (TFL)		Yes		
6	Earthquake Shaking (EL)			Yes	
7	Foundation Settlement (FSL)			Yes	

6.4.3 The structure shall be examined under the action of loads arising from combining the design loads (as stated in **6.3**) as per:

- a) Table 3, when designing to meet the strength criteria (**5.2.1**), wherein:

- 1) In prestress load (PL), the minimum and maximum values in bonded and unbonded Tendons represent upper & lower bounds of estimated design prestress force. Both cases shall be considered separately as sub-cases when PL is combined with other loads in load combinations.
 - 2) While assessing the long term effects due to creep, the full dead load and that part of Live Load likely to be permanent alone may be considered.
- b) Table 4, when checking compliance of the serviceability criteria (5.2.2), where:
- 1) In prestress load (PL), the minimum and maximum values of prestress force in bonded and unbonded tendons represent upper and lower bounds of estimated design prestress force, and shall be considered as separate sub-cases of PL, and hence in the load combinations.
 - 2) While assessing the long term effects due to creep, the full dead load and that part of Imposed Load that is likely to be permanent shall be considered.
- c) Table 4 (same as that for serviceability criteria) when checking compliance of the durability criteria (5.2.3).
- d) Table 5, when checking compliance of the robustness criteria (5.2.4),
- e) Table 6, when checking compliance of the integrity criteria (5.2.5), wherein the earthquake overstrength factor α shall be taken as per IS 1893 (Parts 1 to 11), and
- f) Table 3 to 6 (as applicable), when checking compliance of the restorability criteria (5.2.6), unless stated otherwise in other relevant standards [such as for IS 13935 (Parts 1 to 11) for earthquake retrofit of structures].

Table 3 Partial Safety Factors γ_L to be applied on Characteristic Loads when designing Members of Concrete Structures to meet the Strength Criteria
(Clause 6.4.2 and 6.4.3)

S.No.	Load	Set 1							Set 2				Set 3			
		Basic Combinations							Accidental Combinations				Extreme Combinations			
		11	12	13	14	15	16	17	21	22	23	24	31	32	33	34
A. Force Loads																
1	Dead Load (DL)	1.5	1.5	1.2	0.9	1.5	1.2	1.2	1.2	1.2	1.2	1.2	0.9	1.5	1.2	1.2
2	Imposed Load (IL)	1.5		1.2		1.5	1.2	1.2	1.2	1.2		1.2			1.2	1.2
3	Snow Load (SL)							1.2								
4	Earth Pressure Load (EPL)	1.5	1.5	1.2	1.2	1.5	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.5	1.2	1.2
5	Prestress Load (PL) <i>in prestressed structures only</i>															
	(a) Bonded Tendons															

	<i>Minimum</i>	1.0 1.0 1.0 1.0 1.0 1.0 1.0	1.0 1.0 1.0 1.0	1.0 1.0 1.0 1.0
	<i>Maximum</i>	1.0 1.3 1.0 1.3 1.0 1.0 1.0	1.0 1.0 1.0 1.0	1.0 1.0 1.3 1.0
	(b) Unbonded Tendons			
	<i>Minimum</i>	1.0 0.8 1.0 0.8 1.0 1.0 1.0	1.0 1.0 1.0 1.0	0.8 1.0 0.8 1.0
	<i>Maximum</i>	1.0 1.25 1.0 1.25 1.0 1.0 1.0	1.0 1.0 1.0 1.0	1.25 1.0 1.25 1.0
6	Wind Load (WL)	1.5 1.2 1.5		
7	Liquid Pressure Load (LPL)			
	Static	1.5 1.5		
	Dynamic	1.5		
8	Machine Vibration Load (MVL)	1.2		
9	Blast Load (BL)		1.2	
10	Impact Load (ImL)		1.2	
11	Construction Load (CNL)		1.2	
B. Deformation Loads				
1	Creep (CL)	1.0 1.0 1.0 1.0 1.0 1.0 1.0		
2	Shrinkage (ShL)	1.0 1.0 1.0 1.0 1.0 1.0 1.0		
3	Elastic Shortening (ESL)			
	(a) Dead Load	1.3 1.3 1.3 1.3 1.3 1.3 1.3		
	(b) Live Load	1.3 1.3 1.3 1.3 1.3 1.3 1.3		
	(c) Prestress <i>in prestressed structures only</i>	1.3 1.3 1.3 1.3 1.3 1.3 1.3		
4	Temperature - Ambient Air (TAL)	1.2 1.2 1.2 1.2 1.2 1.2 1.2		
5	Temperature - Fire (TFL)			
6	Earthquake Shaking (EL)			1.0 1.0 1.0
7	Foundation Settlement (FSL)			1.0

Table 4 Partial Safety Factors to be applied on Characteristic Loads when designing Members of Concrete Structures to meet the Serviceability Criteria
(Clause 6.4.2 and 6.4.3)

S.No.	Load	Set 1								Set 2				Set 3			
		Basic Combinations								Accidental Combinations				Extreme Combinations			
		11	12	13	14	15	16	17	21	22	23	24	31	32	33	34	

A. Force Loads										
1	Dead Load (DL)	1.0	1.0	1.0	0.9	1.0	1.0	1.0		
2	Imposed Load (IL)	1.0	0.8	0.5		1.0	1.0	1.0		
3	Snow Load (SL)							1.0		
4	Earth Pressure Load (EPL)	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
5	Prestress Load (PL) <i>in prestressed structures only</i>									
	(a) Bonded Tendons									
	Minimum	0.9	0.9	0.9	0.9	0.9	0.9	0.9		
	Maximum	1.1	1.1	1.1	1.1	1.1	1.1	1.1		
	(b) Unbonded Tendons									
	Minimum	0.95	0.95	0.95	0.95	0.95	0.95	0.95		
	Maximum	1.05	1.05	1.05	1.05	1.05	1.05	1.05		
6	Wind Load (WL)		0.8	0.2	1.0					
7	Liquid Pressure Load (LPL)									
	Static				1.0	1.0				
	Dynamic					1.0				
8	Machine Vibration Load (MVL)							1.0		
9	Blast Load (BL)									
10	Impact Load (ImL)									
11	Construction Load (CNL)									
B. Deformation Loads										
1	Creep (CL)	1.0		1.0	1.0	1.0	1.0	1.0		
2	Shrinkage (ShL)	1.0		1.0	1.0	1.0	1.0	1.0		
3	Elastic Shortening (ESL)									
	(a) Dead Load	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
	(b) Live Load	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
	(c) Prestress <i>in prestressed structures only</i>	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
4	Temperature - Ambient Air (TAL)	1.0	1.0	1.0	1.0	1.0	1.0	1.0		
5	Temperature - Fire (TFL)									
6	Earthquake Shaking (EL)									
7	Foundation Settlement (FSL)									

Table 5 Partial Safety Factors to be applied on Characteristic Loads when designing Members of Concrete Structures to meet the Robustness Criteria
(Clause 6.4.2 and 6.4.3)

S.No.	Load	Set 1							Set 2				Set 3																									
		Basic Combinations							Accidental Combinations				Extreme Combinations																									
		11	12	13	14	15	16	17	21	22	23	24	31	32	33	34																						
A. Force Loads																																						
1	Dead Load (DL)								1.2	1.2	0.9	1.2																										
2	Imposed Load (IL)								1.0	1.0	0.5	0.5																										
3	Snow Load (SL)								0.5																													
4	Earth Pressure Load (EPL)								1.0	1.0	1.0	1.0																										
5	Prestress Load (PL) <i>in prestressed structures only</i>																																					
	(a) Bonded Tendons																																					
	<i>Minimum</i>																																					
	<i>Maximum</i>																																					
	(b) Unbonded Tendons																																					
	<i>Minimum</i>																																					
	<i>Maximum</i>																																					
6	Wind Load (WL)																																					
7	Liquid Pressure Load (LPL)																																					
	Static																																					
	<i>Dynamic</i>																																					
8	Machine Vibration Load (MVL)																																					
9	Blast Load (BL)																			1.0																		
10	Impact Load (ImL)																			1.0								1.0										
11	Construction Load (CNL)																																					
B. Deformation Loads																																						
1	Creep (CL)																																					

2	Shrinkage (ShL)			
3	Elastic Shortening (ESL)			
	(a) Dead Load			
	(b) Live Load			
	(c) Prestress <i>in prestressed structures only</i>			
4	Temperature – Ambient Air (TAL)			
5	Temperature – Fire (TFL)			
6	Earthquake Shaking (EL)		1.0	
7	Foundation Settlement (FSL)			

Table 6 Partial Safety Factors to be applied on Characteristic Loads when designing Members of Concrete Structures to meet the Integrity Criteria
(Clause 6.4.2 and 6.4.3)

S.No.	Load	Set 1							Set 2				Set 3			
		Basic Combinations							Accidental Combinations				Extreme Combinations			
		11	12	13	14	15	16	17	21	22	23	24	31	32	33	34
A. Force Loads																
1	Dead Load (DL)												1.2		1.2	
2	Imposed Load (IL)												1.2		1.2	
3	Snow Load (SL)															
4	Earth Pressure Load (EPL)												1.2		1.2	
5	Prestress Load (PL) <i>in prestressed structures only</i>															
	(a) Bonded Tendons															
	<i>Minimum</i>												1.0		1.0	
	<i>Maximum</i>												1.0		1.0	
	(b) Unbonded Tendons															
	<i>Minimum</i>												0.8		0.8	
	<i>Maximum</i>												1.25		1.25	
6	Wind Load (WL)															
7	Liquid Pressure Load (LPL)															

	Static			1.0	1.0
	Dynamic			1.0	1.0
8	Machine Vibration Load (MVL)				
9	Blast Load (BL)				
10	Impact Load (ImL)				
11	Construction Load (CNL)				
B. Deformation Loads					
1	Creep (CL)				
2	Shrinkage (ShL)				
3	Elastic Shortening (ESL)				
	(a) Dead Load				
	(b) Live Load				
	(c) Prestress <i>in prestressed structures only</i>				
4	Temperature – Ambient Air (TAL)				
5	Temperature – Fire (TFL)				
6	Earthquake Shaking (EL)	Ω			
7	Foundation Settlement (FSL)		1.0		

7 STRUCTURAL MATERIALS

The minimum requirements given hereunder shall be achieved on site by ingredients, grades, testing, physical properties, mix, composition, reinforcing steel and prestressing steel so as to meet requirements of design and detailing specified in this standard.

Structural materials shall comply with the requirements laid down in this standard and in other relevant national standards, so that the required performance of the structure is assured as envisaged throughout the design service life of structures built using these structural materials.

7.1 Concrete

The ingredients, grades, methods of testing, physical properties and mix proportions of concrete shall comply with the requirements given hereunder.

7.1.1 Ingredients of Concrete

The basic ingredients of concrete shall be chosen based on the specifications specified hereunder.

7.1.1.1 Aggregates

- a) *Normal weight aggregates* — Aggregates derived from natural sources and other than natural sources (manufactured aggregates such as iron slag, copper slag, bottom ash from thermal power plants, recycled concrete aggregates and recycled aggregates), shall comply with the requirements of IS 383. These are natural or synthetic aggregates with in-place density (unit weight) between 22.40 to 27.00 kN/m³.

NOTE — For aggregates having unit weight more than 24.00 kN/m³, higher unit weight than mentioned in IS 875 (Part 1) need to be considered depending upon the actual unit weight of aggregates.

- b) *Heavyweight aggregates* — Natural or synthetic aggregates with in-place density (unit weight) more than 27.00 kN/m³ and can range up to 45.00 kN/m³ for aggregates other than steel. Steel aggregates (if used) will have further higher density. Heavy weight aggregate is most commonly used for radiation shielding, counterweights and other applications where a high mass-to-volume ratio is desired. Goethite, Limonite, Barite, Illmenite, Magnetite, Hematite, Ferrophosphorus and Steel are some examples of heavy weight aggregate.

Specialist literature may be referred for designing concrete structures made using heavy weight aggregates (*see also 12*).

The engineering properties and durability of concrete made with heavy weight aggregates are different than normal concrete. Therefore the properties and design formulae given in this code (for normal concrete) shall not be applicable. Specialist literature should be consulted.

- c) *Lightweight aggregates* — Natural or synthetic with in-place density (unit weight) between 14.40 to 18.40 kN/m³. The lightweight is due to the cellular or high internal porous structure, which gives this type of aggregate a low specific gravity. The most important aspect of lightweight aggregate is the porosity. They have high absorption values, which requires a modified approach to concrete proportioning. Light weight aggregates namely sintered fly ash aggregates may be used for making light weight structural concrete (a type of special concrete). Sintered flyash coarse aggregates when used shall conform to IS 9142 (Part 2).

Specialist literature may be referred for designing concrete structures made using heavy weight aggregates (*see also 12*).

The engineering properties and durability of concrete made with light weight aggregates are different than normal concrete. Therefore the properties and

design formulae given for normal concrete shall not be applicable. Specialist literature should be consulted to take into consideration the important aspects of design such as development length, durability (cover for fire resistance), shear and torsion resistance of beams, deflection of beams, additional moments in slender columns etc. when lightweight aggregates are used. Also refer details given in chapter on “Special Concretes”.

d) *Size of Aggregate*

For normal concrete, the nominal maximum size of coarse aggregate should be as large as possible within the limits specified but in no case greater than one-fourth of the minimum thickness of the member, provided that the concrete can be placed without difficulty so as to surround all reinforcement thoroughly and fill the corners of the form. For most work, 20 mm nominal maximum size aggregate is suitable. Where there is no restriction to the flow of concrete into sections, 40 mm or larger size may be permitted. In high strength concrete, size lower than 20 mm is more suitable. In concrete elements with thin sections, closely spaced reinforcement or small cover, consideration should be given to the use of 10 mm nominal maximum size.

Plums (large aggregate used in mass concrete) above 160 mm and up to any reasonable size may be used in plain concrete work up to a maximum limit of 20 percent by volume of concrete when specifically permitted by the engineer-in-charge. The plums shall be distributed evenly and shall not be closer than 150 mm from the surface.

For heavily reinforced concrete members, the nominal maximum size of the aggregate should usually be restricted to 5 mm less than the minimum clear distance between the parallel bars or 5 mm less than the minimum cover to the reinforcement whichever is smaller.

7.1.1.2 Cement

- a) For general use in plain, reinforced and prestressed concrete construction, cement used shall be any of the following and the type selected should be appropriate for the intended use:
- 1) Ordinary Portland cement conforming to IS 269
 - 2) Portland slag cement conforming to IS 455
 - 3) Portland pozzolana cement (fly ash based) conforming to IS 1489 (Part 1)
 - 4) Portland pozzolana cement (calcined clay based), conforming to IS 1489 (Part 2)
 - 5) Composite cement conforming to IS 16415
- b) Other cements as given below can be used for specific applications (7.1.1.2.1 to 7.1.1.2.8)

- 1) Sulphate resisting Portland cement conforming to IS 12330
- 2) Microfine Ordinary Portland Cement conforming to IS 16993
- 3) Hydrophobic cement conforming to IS 8043
- 4) Low heat Portland cement conforming to IS 12600
- 5) Rapid-hardening Portland cement conforming to IS 8041
- 6) High alumina cement conforming to IS 6452
- 7) Supersulphated cement conforming to IS 6909
- 8) White Cement conforming to IS 8042

The attention of the users of cement is drawn to the fact that quality of various cements mentioned in **7.1.1.2** is to be determined on the basis of its conformity to the performance characteristics given in the respective Indian Standard Specification for that cement. Any trade-mark or any trade name indicating any special features not covered in the standard or any qualification or other special performance characteristics sometimes claimed/indicated on the bags or containers or in advertisements alongside the 'Statutory Quality Marking' or otherwise have no relation whatsoever with the characteristics guaranteed by the Quality Marking as relevant to that cement. Consumers are, therefore, advised to go by the characteristics as given in the corresponding Indian Standard Specification or seek specialist advice to avoid any problem in concrete making and construction.

7.1.1.2.1 Sulphate resisting Portland cement is a type of Portland Cement in which the amount of tri-calcium aluminate (C3A) is restricted to lower than 5 percent and 2(C3A + C4AF) lower than 25 percent.

7.1.1.2.2 Composite cement conforming to IS 16415 is a mixture of Portland cement clinker (IS 16353), fly ash [IS 3812 (Part 1)], granulated slag (IS 12089) and gypsum.

7.1.1.2.3 Low heat Portland cement conforming to IS 12600 shall be used with adequate precautions with regard to removal of formwork, etc.

7.1.1.2.4 High alumina cement conforming to IS 6452 is suitable for use only under special circumstances with the prior approval of the engineer-in-charge. Specialist literature may be consulted for guidance regarding the use of these types of cements. but in some cold regions, this cement may find use as a structural material taking advantage of high heat of hydration and high early strength development.

7.1.1.2.5 Supersulphated cement is used where concrete is likely to be attacked by sulphate. It is a product of granulated blast furnace slag, calcium sulphate and a small quantity of Portland cement or Portland cement clinker or any other suitable source of lime. This cement has chemical resistance to most of the aggressive conditions generally encountered in construction and to the attack of sulphates in particular. It is used generally for marine works, mass concrete jobs to resist the attack by aggressive waters, reinforced concrete pipes in ground waters, concrete construction in sulphate-bearing soils, in chemical works involving exposure to high concentration of sulphates of weak solutions of mineral acids, underside of bridges, over railways (steam driven locos) and for concrete

sewers carrying industrial effluents. Its use is not recommended by IS 6909, when the prevailing temperature is above 40°C. It should not be used for steam cured concrete products.

7.1.1.2.6 Microfine ordinary Portland cement conforming to IS 16993 is suitable for special applications like rock grouting, grouting concrete structures and underground construction for leak prevention, soil stabilization, etc.

7.1.1.2.7 Hydrophobic Portland cement is manufactured on special requirement for high rainfall areas to improve the shelf life of the cement.

7.1.1.2.8 Rapid hardening Portland cement (RHPC) has similar properties to that of ordinary Portland cement except that RHPC is more finely ground with slightly altered chemical composition.

7.1.1.3 *Water*

Water used for mixing and curing shall be clean and free from excessive amounts of total dissolved solids (TDS), oils, acids, alkalis, salts, sugar, organic materials or other substances that may be deleterious to steel and setting and strength development of concrete.

7.1.1.3.1 *Types of water*

In general, the suitability of water for the production of concrete depends upon its origin. Water to be used for mixing and curing of concrete shall be any of the types described below. Any type of water to be used for mixing and curing shall be tested as per stated procedures and shall meet all the requirements given in **7.1.1.3.2**.

7.1.1.3.1.1 *Potable water*

This water is considered as suitable for use in concrete if meeting the requirements of IS 10500 wherein the chloride content shall be limited to 500 PPM and alkalinity is less than or equal to 25 ml (**7.1.1.3.2**). Such water needs no further testing. In case of any doubt it shall be tested for conformity according to **7.1.1.3.2**.

7.1.1.3.1.2 *Natural surface water*

This water, if found suitable, can be used in concrete but shall satisfy the requirements of **7.1.1.3.2**.

7.1.1.3.1.3 *Treated water*

Appropriately treated water from industrial, municipal and other sources can be used in concrete.

7.1.1.3.1.4 Water recovered from processes in the concrete industry

This includes:

- a) water that was part of surplus concrete,
- b) water used to clean the inside of the stationery mixers, mixing drums of truck mixer or agitators and concrete pumps,
- c) process water from sawing, grinding and water blasting of hardened concrete.

This water may be taken from:

- a) Basins provided with suitable equipment that distributes the solid matter evenly throughout the water;
- b) Sedimentation basins or similar installations, provided the water is left in the basin for a sufficient amount of time to allow the solids to settle properly.

Water recovered from process in the concrete industry contains varying concentrations of very fine particles, the size of which is generally less than 0.25 mm. Water recovered from processes in the concrete industry or combined water can be used provided the following requirements are met:

- a) The additional mass of solid material in the concrete resulting from the use of water recovered from processes in the concrete industry shall be less than 1 percent mass fraction of the total mass of aggregates present in the concrete.
- b) The amount of recovered water shall be spread as evenly as possible over a day's production.

7.1.1.3.1.5 Water from underground sources**7.1.1.3.1.5.1 Recycled water**

Water recovered from concrete production operation or water generated from sewage, greywater or storm water systems and treated to a standard that is appropriate for its intended use.

7.1.1.3.1.5.2 Desalinated water

Desalinated water conforming to **7.1.1.3.2** can be used. However normal sea water or brackish water shall not be used for mixing of concrete. because of presence of harmful salts.

7.1.1.3.2 Testing and requirements of water

- a) *Preliminary Inspection*

The water shall be examined for oil and fats, detergents, colour, suspended matter, odour and humic matter shall meet the requirements given in Table 7 in accordance with test procedures stated below.

A small subsample shall be assessed as soon as possible after sampling. Bring any material that has settled back into suspension by shaking the sample. Pour 80 ml of the sample into a 100 ml measuring cylinder. Seal with a suitable stopper and shake the cylinder vigorously for 30 s. Smell the sample for any odours other than those of clean water. If in doubt about the odour, test the water for its odour level in accordance with national regulations for potable water. The odour level of the water shall be lower than the maximum level accepted for potable water. Observe the surface for foam. Set the cylinder in a place free from vibration and allow to stand for 30 min. After 2 min, check the sample for the continuing presence of foam and signs of any oils or fats. After 30 min have elapsed, observe the apparent volume of the settled solids and the colour of the water. Measure the pH using indicator paper or a pH meter. Then, add 0.5 ml hydrochloric acid, mix and then smell or test for the presence of hydrogen sulfide.

Put 5 ml of the sample into a test tube. Bring it to a temperature between 15 °C and 25°C by allowing it to stand indoors. Add 5 ml of 3 % sodium hydroxide solution, shake and leave for 1 h. Observe the colour.

Table 7 Requirements for Preliminary Inspection of Mixing Water
[Clause 7.1.1.3.2(a)]

Sl No.	Parameter	Requirements
(1)	(2)	(3)
i)	Oils and fats	Not more than visible traces
ii)	Detergents	Any foam should disappear within 2 min.
iii)	Colour	Water not from sources classified as potable: the colour shall be assessed qualitatively as pale yellow or pale
iv)	Odour	Water from sources classified as potable: no smell, except the odour allowed for potable water and a slight smell of cement; where blast-furnace slag is present in the water, a slight smell of hydrogen sulfide
		Water from other sources: no smell, except the odour allowed for potable water; no smell of hydrogen sulfide after addition of Hydrochloric Acid
v)	PH value	6.5-8.5
vi)	Humic Matter	The colour shall be assessed qualitatively as yellowish / brown or paler after addition of NaOH

b) Permissible limits for solids shall be as given in Table 8.

Table 8 Maximum Permissible Limit for Solids for Mixing and Curing Water
[Clause 7.1.1.3.2(b)]

Sl No.	Parameter	Tested as per	Maximum Permissible Limit	
			For mixing water	For curing water
(1)	(2)	(3)	(4)	(5)
i)	Organic	IS 3025 (Part 18)	200 mg/l	200 mg/l
ii)	Inorganic	IS 3025 (Part 18)	3000 mg/l	3000 mg/l
iii)	Sulphate (as SO_3^{2-})	IS 3025 (Part 24)	400 mg/l	800 mg/l
iv)	Chloride (as Cl^-)	IS 3025 (Part 32)	2000 mg/l for concrete not containing embedded steel and 500 mg/l for reinforced concrete work*.	500* mg/l for reinforced concrete or prestressed concrete work and 2000 mg/l for concrete not containing embedded steel.
v)	Suspended Matter	IS 3025 (Part 17)	2000 mg/l	2000 mg/l
* NOTE 1000 mg/l chloride in water for reinforced concrete work can be permitted provided the chloride content of hardened concrete is checked at the time of concrete mix design and conform to requirement of total chloride content in the concrete as per this standard.				

c) The pH value of water shall be not less than 6.

d) Higher alkalinity or acidity can affect strength and setting of concrete. The following concentrations give the maximum permissible values:

- 1) To neutralize 100 ml sample of water, using phenolphthalein as an indicator, it should not require more than 5 ml of 0.02 normal NaOH. The details of test are given in 8.1 of IS 3025 (Part 22).
- 2) To neutralize 100 ml sample of water, using mixed indicator, it should not require more than 25 ml of 0.02 normal H_2SO_4 . The details of test shall be as given in 8 of IS 3025 (Part 23).

However, in case of doubt regarding development of strength and setting of concrete due to alkalinity/acidity or higher organic matter, the suitability of water for making concrete shall be ascertained by the compressive strength and initial setting time tests specified below:

- 1) Average 28 days compressive strength of at least three 150 mm concrete cubes prepared with water proposed to be used shall not be less than 90 percent of the average of strength of three similar concrete cubes prepared with distilled water. The cubes shall be prepared, cured and tested in accordance with the requirements of IS 516.
- 2) The initial setting time of test block made with the appropriate cement and the water proposed to be used shall not be less than 30 min and shall not differ by $\pm$

30 min from the initial setting time of control test block prepared with the same cement and distilled water. The test blocks shall be prepared and tested in accordance with the requirements of IS 4031 (Part 5).

In addition to the requirement for the alkalinity/acidity or higher organic matter tests mentioned above, the same water with higher alkalinity /acidity or higher organic matter shall be used for concrete mix design in the laboratory for adoption at construction site.

- e) A suitable means of ensuring uniform distribution of the solid material in recovered water with a density greater than 1.01 kg/l shall be provided. Water with a density less than or equal to 1.01 kg/l may be assumed to contain negligible amounts of solid material.
- f) If it is expected to use potential alkali aggregate reactive aggregates in concrete by adopting appropriate measures and exposure conditions can promote deleterious reaction, the water shall be tested for its alkali content. The equivalent sodium oxide content shall not exceed 1500 mg/l. If these limits are exceeded, the water may be used only if it can be shown that actions have been taken to prevent deleterious alkali-silica reactions.

7.1.1.3.3 Testing and requirements of curing water

Permissible limits for solids shall be as given in Table 8. Water used for curing should not produce any objectionable stain or unsightly deposition on the concrete surface. The presence of tannic acid or iron compounds is objectionable.

7.1.1.3.4 Sampling of water for concrete making and curing

A sample of water not less than 5 litres shall be taken. The sample of water taken for testing shall represent the water proposed to be used for concreting with due account being paid to seasonal variation. The sample shall be stored in a clean container previously rinsed out with similar water.

7.1.1.4 Admixtures

Admixtures can be used to:

- a) Alter the composition of concrete by replacing some of the cement in the concrete by pozzolanic and hydraulic reaction (and are called mineral admixtures or supplementary cementitious materials)
- b) Enhance a certain function of concrete (and are called chemical admixtures).

The following types of admixtures shall be used in production of concrete.

7.1.1.4.1 Mineral admixtures/Supplementary cementitious materials (SCMs)

SCMs as given below may be used as part replacement of cement. They can be pozzolanic material or reactive mineral admixture or combination of both.

a) *Fly ash (pulverized fuel ash)*

Fly ash conforming to IS 3812 (Part 1) may be used as part replacement of ordinary Portland cement. For concrete having steel, the replacement level of fly ash shall be restricted to 35 percent by weight of cementitious material wherein fly ash shall have a minimum fineness of 280 m²/kg and lime reactivity of 4.5 N/mm² or more.

b) *Silica fume*

Silica fume conforming to IS 15388 may be used as part replacement of cement. Silica fume is usually used in proportion of 3 to 10 percent of the cementitious content of a mix. The total replacement by fly ash and silica fume shall not be more than 35 percent for reinforced cement concrete. The total replacement by GGBF slag and silica fume/Ultrafine Ground Granulated Blastfurnace Slag (UFGBS) shall not be more than 60 percent for reinforced cement concrete.

The above limits shall also apply where PPC or PSC is used. The fly ash or slag content mentioned on the bag shall be considered.

For mass concrete, structures below ground (not exposed to atmosphere) and for plain concrete, the use of more than one type of mineral admixture with ordinary Portland cement or a mineral admixture with blended cements such as PPC and PSC can be permitted after necessary trials (to control heat of hydration). However, silica fume may be used as an additional mineral admixture in reinforced concrete subjected to limits specified above. Uniform blending of the mineral admixtures with the cement should be ensured. In such cases, fly ash and GGBS together may be used within following limits:

Total replacement (<i>Max</i>)	= 45 percent of total cementitious content
Fly ash (<i>Max</i>)	= 20 percent
GGBS (<i>Max</i>)	= 25 percent

c) *Metakaolin*

Metakaolin conforming to IS 16354 may be used as pozzolanic material in concrete as part replacement of ordinary Portland cement only. The replacement level shall be maximum up to 15 percent.

d) *Ground Granulated Blast Furnace Slag (GGBS)*

GGBS conforming to IS 16714 may be used as part replacement of ordinary Portland cement. For concrete having embedded steel, the replacement level of GGBS shall be restricted to 50 percent by weight of cementitious materials. For resistance to sulphate and alkali aggregate reaction, GGBS up to 60 percent can be used.

e) *Ultrafine Ground Granulated Blast Furnace Slag (UFGBS)*

UFGBS conforming to IS 16715 may be used as part replacement of cement. The replacement level shall be up to 20 percent.

f) *Ultrafine Fly ash*

Ultrafine fly ash produced through classification route and conforming to CED 02 (26639) may be used as part replacement of ordinary Portland cement. The replacement level shall be up to 20 percent.

7.1.1.4.2 Chemical admixtures

7.1.1.4.2.1 Chemical admixtures, as mentioned in Table 9 if used, shall comply with IS 9103. Previous experience with and data on such materials should be considered in relation to the likely standards of supervision and workmanship to the work being specified.

7.1.1.4.2.2 Chemical admixtures can be achieved by a spectrum of materials and manufacturing processes, but shall comply with IS 9103. Further

- a) The workability, compressive strength and the slump loss of concrete with and without the use of admixtures shall be established during the trial mixes before use of admixtures.
- b) The relative density of liquid admixtures shall be checked for each drum containing admixtures and compared with the specified value before acceptance.
- c) If two or more admixtures are used simultaneously in the same concrete mix, data shall be obtained to assess their interaction and to ensure their compatibility.
- d) Admixtures shall neither impair durability of concrete nor combine with the constituent to form harmful compounds nor increase the risk of corrosion of reinforcement.
- e) The chloride content of admixtures shall be independently tested for each batch before acceptance.

7.1.1.4.2.3 The amount of admixture added to a mix shall be recorded in the production record. Additional dose of admixture may be added at project site and mixed adequately in mixer itself to regain the workability of concrete (if necessary) with mutual agreement between the producer/supplier and the purchaser/user of concrete. But, the producer/supplier shall assure the ultimate quality of concrete supplied by him and maintain record of quantity and time of addition.

7.1.1.4.2.4 Shrinkage compensating admixtures shall not be used in structural concrete.

Table 9 Types of Chemical Admixtures and their Constituents
(Clause 7.1.1.4.2.1)

SI No.	Type of admixture	Typical active constituents
(1)	(2)	(3)
i)	Air-entraining	Salts of wood resins Some synthetic detergents Salts of sulfonated lignin Salts of petroleum acids Salts of proteinaceous material Fatty and resinous acids and their salts Tall oils and gum rosin salts Alkylbenzene sulfonates Salts of sulfonated hydrocarbons
ii)	Retarding	Unrefined Lignosulphonate containing Sugars Modified Derivatives of unrefined Lignosulphonate containing Sugars Hydroxycarboxylic Acids and their Salts Carbohydrates including Sugars Heptonates related to Sugars and Starches
iii)	Accelerating	Calcium chloride Triethanolamine Sodium thiocyanate Sodium formate or Calcium formate Sodium nitrite or Calcium nitrite Calcium nitrate Aluminates Silicates
iv)	Water-reducing	Lignosulfonic acids and their salts Hydroxylated carboxylic acids and their salts Polysaccharides Melamine polycondensation products Naphthalene polycondensation products Polycarboxylates
v)	High range water-reducing : Normal and Retarding	Melamine sulfonate polycondensation products Naphthalene sulfonate polycondensation products Polycarboxylates
vi)	Shrinkage reducing	Polyoxyalkylene alkyl ether Propylene glycol
vii)	Corrosion inhibiting	Amine carboxylates aminoester organic emulsion Calcium nitrite Organic alkydicarboxylic

		Chromates Phosphates Hypophosphites Alkalis Fluorides
viii)	Permeability-reducing admixture	
	Non-hydrostatic conditions (PRAN)	Long-chain fatty acid derivatives (stearic, oleic, caprylic capric) Soaps and oils (tallows, soya-based) Petroleum derivatives (mineral oil, paraffin,) Fine particle fillers (silicates, talc)
	Hydrostatic conditions (PRAH)	Crystalline hydrophilic polymers (latex, water-soluble, or liquid polymer).

7.1.1.5 Fibres

Fibers may be added to concrete for special applications to enhance select properties. But, such fibers shall be approved by the competent authority on the recommendation of a duly constituted competent peer review committee.

In any case, the presence of fibers shall not be taken advantage of in the estimation of strength to meet the requirements of limit state of safety specified in **9.3**.

7.1.2 Grades of concrete

7.1.2.1 The provisions of this standard shall be admissible to the grades of concrete listed in Table 10.

7.1.2.2 Concrete of grades lower than those given in Table 10 shall be used only as lean concrete. meant to be a leveling course, foundation for masonry boundary walls and other small temporary RC construction of single storey, provided the strength of such concrete is sufficient to resist the loads appearing on the concrete.

7.1.2.3 For concrete using mineral admixtures and those using high early strength cements, the properties of setting time and time dependent strength gain are different from those of standard and ordinary concretes. Cognizance of such modified properties shall be taken in deciding de-shuttering time, curing period and early age loading.

7.1.2.4 The minimum grade of plain concrete (PC), reinforced concrete (RC) and prestressed concrete (PSC) shall be taken. (to be discussed)

7.1.3 Concrete Mix Proportioning

7.1.3.1 Mix proportion

7.1.3.1.1 The mix proportions shall be selected to ensure required:

- a) workability of the fresh concrete,
- b) strength, durability and surface finish of hardened concrete, and
- c) any other specific requirement.

7.1.3.1.2 The determination of the proportions of ingredients of concrete to attain the properties of concrete as mentioned above shall be made by:

- a) designing the concrete mix such concrete shall be called 'Design mix concrete', or
- b) adopting predetermined concrete mix; such concrete shall be called 'Nominal mix concrete'.

7.1.3.1.3 Design mix concrete shall be preferred to nominal mix. When the former cannot be used for any reason in projects with grades of M20 or lower; the latter may be used with the permission of engineer-in-charge.

Table 10 Grades of Concrete
(Clause 7.1.2)

Group	Grade Designation	Specified Characteristic Compressive Strength based on strengths of 150mm cubes at 28 days (in MPa)
(1)	(2)	(3)
Ordinary Concrete	M10	10
	M15	15
	M20	20
Standard Concrete	M25	25
	M30	30
	M35	35
	M40	40
	M45	45
	M50	50
	M55	55
	M60	60
High Strength Concrete	M65	65
	M70	70
	M80	80
	M90	90
	M100	100

7.1.3.1.4 Information required for concrete mix design

In specifying a particular grade of concrete for concrete mix design, the following information shall be included:

- a) Grade designation;
- b) Type and grade of cement;
- c) Minimum and maximum cement content;
- d) Type of fine aggregate (whether natural sand, crushed stone, gravel sand, mixed sand or manufactured);
- e) Maximum nominal size of aggregate;
- f) Type of coarse aggregate (whether angular, sub-angular or gravel with some crushed particles, rounded gravel or manufactured);
- g) Maximum water-binder ratio;
- h) Exposure conditions as per **9.5.3**;
- j) Type, extent and condition of use of mineral or chemical admixture, (if to be added);
- k) Maximum temperature of concrete at the time of placing;
- l) Method of placing;
- m) Workability requirement at the time of placement;
- n) Level of quality assurance (such as normal, high or very high);
- o) Transportation time;
- p) Design service life;
- q) Durability test requirements as applicable as per specification, if any; and
- r) Any other specific requirement like early age strength requirements etc.

7.1.3.2 Design mix concrete

The design mix concrete up to M 60 shall meet the requirements given in **7.1.3.2.1** to **7.1.3.2.6**.

7.1.3.2.1 The constructor shall carry out the mix design and the mix so designed (not the method of design) shall be approved by the engineer-in-charge, within the limitations of parameters and other stipulations laid down by this standard. If sought by the engineer-in-charge, supporting data (including graphs showing strength versus water-cement ratio for range of proportions, complete trial mix proportioning details) shall be provided to substantiate the choice of cement content, fine and coarse aggregate content, water, mineral admixtures, chemical admixtures, etc.

7.1.3.2.2 The mix shall be designed to produce the densest possible concrete, following the approved procedures of mix designs and trials.

7.1.3.2.3 The grade of concrete shall have the required workability, durability characteristics and characteristic strength not less than the values given in Table 10.

7.1.3.2.4 The target mean strength $f_{ck,targ et}$ of concrete mix shall be estimated as:

$$f_{ck,targ et} = \text{Max}[f_{ck} + 1.65\sigma; f_{ck} + X]$$

where

f_{ck} = Characteristic compressive strength at 28 days (specified strength);

σ = Standard deviation as per Table 11; and

X = factor based on grade of concrete as per Table 11.

7.1.3.2.5 Mix design done may be considered adequate for works undertaken within an year, provided there is no change in source and the quality of the materials.

7.1.3.2.6 *Standard deviation*

The standard deviation for each grade of concrete shall be calculated, separately.

a) *Standard deviation based on test strength of sample*

- 1) *Number of test results of samples* — Total number of test strength of samples required to constitute an acceptable record for calculation of standard deviation shall be not less than 30. Attempts should be made to obtain the 30 samples, as early as possible, when a mix is used for the first time.
- 2) *In case of significant changes in concrete* — When significant changes are made in the production of concrete batches (for example changes in the materials used, mix design, equipment or technical control), the standard deviation value shall be separately calculated for such batches of concrete.
- 3) *Standard deviation to be brought up to date* — Calculation of standard deviation shall be brought up to date after every change of mix design. The standard deviation shall be checked every month, subject to availability of a minimum 30 test results to ensure that it is less than the value considered in mix design. If is found to be higher, necessary modification shall be done in the mix.

b) *Assumed standard deviation*

Where sufficient test results for a particular grade of concrete are not available, the value of standard deviation given in Table 11 may be assumed for the proportioning of mix in the first instance. As soon as the results of samples are available, actual calculated standard deviation (from at least 30 samples) shall be used and the mix proportioned suitably. However, when adequate past records for a similar grade exist and justify to the designer a value of standard deviation different from that shown in Table 11, it shall be permissible to use that value. If recycled aggregates are used as part replacement of aggregates, the standard deviation values given in Table 11 shall be increased by 1.0.

Table 11 Assumed Standard Deviation
[Clauses 7.1.3.2.4 and 7.1.3.2.6 (b)]

Grade of Concrete	Assumed Standard Deviation (in MPa)	X (in MPa)
(1)	(2)	(3)
M10, M15	3.5	5.0
M20, M25	4.0	5.5
M30 to M60	5.0	6.5
M65 to M80	6.0	8.0
M90, M100	8.0	10.0

7.1.3.3 Nominal mix concrete

In general, nominal mix concrete shall be avoided. When constrained to use it, the use of nominal mix concrete shall be restricted to works of minor nature in which the strength of concrete is not critical. Nominal mix concrete shall be permitted only for making concretes of grades M20 or lower. M20 concrete shall be used for RCC only if considered acceptable from the point of view of durability.

The mix proportions of materials for nominal mix concrete shall be as specified in Table 12. The nominal mix specified therein shall be measured by weight and shall not be measured by volume. Also,

- Cement content of the mix specified in Table 12 for any nominal mix shall be proportionately increased in case quantity of water in a mix has to be increased to overcome difficulties of placement and compaction, so that the water content as specified is not exceeded.
- Quantity of water required from durability point of view may be less than those given in Table 12. Suitable dose of water reducing admixture/ superplasticizers may be used to achieve required workability without increasing water content as given in Table 12. But, in no case, water content shall exceed the values given in Table 12.
- The proportion of the fine aggregates to coarse aggregates shall be adjusted to from upper limit to lower limit progressively as the grading of fine aggregates becomes finer and the maximum size of coarse aggregate becomes larger. Graded coarse aggregate shall be used.

Table 12 Proportions for Nominal Mix Concrete
(Clause 7.1.3.3)

Grade of Concrete	Maximum Quantity of Dry Aggregates by Mass per 50 kg of Cement (taken as sum of Individual Masses of Fine and Coarse Aggregates) (in kg)	Proportion of Fine Aggregates to Coarse Aggregates (by Mass)	Maximum Quantity of Water per 50 kg of Cement (in liters)
(1)	(2)	(3)	(4)
M 5	800	Generally, 1:2, but subject to upper and lower limits of 1:1½ and 1:2½	60
M 7.5	625		45
M 10	480		34
M 15	330		32
M 20	250		30

7.1.4 Physical Properties

The salient physical properties of concrete shall be taken as specified hereunder.

7.1.4.1 Modulus of elasticity

The short-term static characteristic value of modulus of elasticity E_{ck} of concrete (to be used in the analysis and design of concrete structures) shall be taken as:

$$E_{ck} = 10000 f_{ck}^{0.3}$$

7.1.4.1.1 The above equation is valid for concretes made with quartzite and granite aggregates only. For limestone and sandstone aggregates, the value should be reduced by 10 percent and 30 percent respectively. For basalt aggregates, the value should be increased by 20 percent.

7.1.4.1.1 Partial safety factor γ_M for material or factor of safety κ_M shall not be applied to E_{ck} given above.

7.1.4.2 Poisson's ratio

The poisson's ratio ν of uncracked concrete with different aggregates shall be taken as below:

Type of Aggregate used in concrete	Poisson's Ratio ν
(1)	(2)
Quartzite and Granite	0.12

Limestone	0.15
Basalt	0.18
Sandstone	0.20

7.1.4.3 Coefficient of thermal expansion

The coefficient of thermal expansion (α) of concrete with different aggregates shall be taken as below:

Type of Aggregate used in concrete	Coefficient of Thermal Expansion (α) of Concrete ($\times 10^{-5} / ^\circ\text{C}$)
(1)	(2)
Quartzite	1.30
Sandstone	1.20
Basalt	0.95
Granite	0.95
Limestone	0.95

7.1.4.4 Creep coefficient

The creep co-efficient $\varphi_{t,t_0}(t)$ shall be defined as:

$$\varphi_{t,t_0}(t) = \frac{\varepsilon_{cc}(t)}{\varepsilon_{ci}(t_0)},$$

where

$\varepsilon_{cc}(t)$ = Creep strain at time $t > t_0$ (it does not include the instantaneous elastic strain in concrete at the time of loading);

$\varepsilon_{ci}(t_0)$ = Initial strain at loading; and

t_0 = Age of concrete at the time of loading.

7.1.4.4.1 The coefficient given above shall not be applicable to light weight concretes.

The above estimates of $\varphi_{t,t_0}(t)$ shall be considered for normal weight concretes of grades M20 to M100, if the compressive stress does not exceed $0.36f_{ck}$ at the age of loading.

7.1.4.4.2 The creep coefficient $\varphi_{t,t_0}(t)$ in concrete members at time t shall be determined depending on the level of creep expected in structures. For this purpose, structures shall be classified into three levels of creep as given below:

a) Level 1

The following members and structures are placed under this level:

- 1) Reinforced concrete beams, frames and slabs with spans up to 20 m, and cantilever beams up to 2.5 m span;
- 2) Plain concrete footings with a reasonably uniform distribution of loads at their base;
- 3) Reinforced concrete retaining walls with heights up to 30 m; and
- 4) All structures having design service life upto 75 years or made using concrete of grades between M20 and M60 or having overall height less than 30 m.

b) *Level 2*

The following members and structures are placed under this level:

- 1) Prestressed beams or slabs of spans up to 20 m, and cantilever beams up to 2.5 m span;
- 2) Buildings of heights up to 100 m;
- 3) Box-girder, cable-stayed or arch bridges with spans up to 80 m;
- 4) Ordinary tanks, silos and pavements, and
- 5) All structures having design life up to 75 years.

c) *Level 3*

The following members and structures are placed under this level:

- 1) Prestressed, box girder, cable-stayed or arch bridges with span more than 80 m;
- 2) Buildings of heights more than 100 m;
- 3) Gravity, arch or buttress dams; cooling towers; large roof shells; nuclear containments and vessels; and large offshore structures; and
- 4) All structures having design service life more than 75 years or made using concrete of grades higher than M60 or having overall height less than 30 m.

7.1.4.4.3 $\varphi_{t,t_0}(t)$ shall be estimated as given below for different levels of creep. When self-compacting concrete is used, $\varphi_{t,t_0}(t)$ shall be increased by 10 percent for all levels of creep.

a) *Level 1*

For structures sustaining level 1 creep, $\varphi_{t,t_0}(t)$ shall be taken as given below:

<i>Age at the time of loading (t_0)</i>	<i>Creep Coefficient [$\varphi_{t,t_0}(t)$]</i>
7 days	2.8
28 days	2.1
365 days	1.3

b) *Level 2*

- 1) For structures sustaining level 2 creep, $\varphi_{t,t_0}(t)$ shall be taken as per Table 13. The values specified in Table 13 are for exposure categories C2, C3 and C4 as defined in 9.5.4.3.2. For exposure category C1, values given in Table 13 shall be multiplied by 1.2.

Values in Table 13 correspond to 60 percent relative humidity and for 75 years design life.

Table 13 Improved method of assessing Creep Coefficient $\varphi_{t,t_0}(t)$ considering size of Member and grade of concrete
[Clause 7.1.4.4.3(b)]

Grade of concrete	Age at the time of loading t_0 (in days)	Creep Coefficient $\varphi_{t,t_0}(t)$ for different sizes of members		
		100 mm – 300 mm	300 mm	600 mm
(1)	(2)	(3)	(4)	(5)
M25 to M45	1	4.61	3.96	3.65
	7	3.22	2.76	2.55
	28	2.48	2.13	1.96
	90	1.98	1.70	1.57
	365	1.51	1.30	1.20
M50 to M65	1	3.28	2.86	2.66
	7	2.29	2.00	1.86
	28	1.76	1.54	1.43
	90	1.41	1.23	1.14
	365	1.07	0.94	0.87
M70 and higher	1	2.46	2.18	2.05
	7	1.72	1.52	1.43
	28	1.32	1.17	1.10
	90	1.06	0.94	0.88
	365	0.80	0.71	0.67

- 2) Where end results are not sensitive to precise values of creep coefficients, the values given in Table 13 may be considered for design of structures with normal weight concrete provided that compressive stress does not exceed $0.36f_{ck}$ at the age of loading and mean temperature of concrete lie between 10°C and 20°C with seasonal variation between -20°C to 40°C. For temperature greater than 40°C, the

creep coefficients given in Table 13 may be increased by 10 percent, in the absence of accurate data.

c) *Level 3*

For structures sustaining level 3 creep, $\varphi_{t,t_0}(t)$ shall be estimated as:

$$\varphi_{t,t_0}(t) = \varphi_0 \beta_{t_0}(t),$$

where

φ_0 = Notional creep coefficient to which the creep co-efficient reaches logarithmically in 75 years, and

$\beta_{t_0}(t)$ = Coefficient describing development of creep with time

$$= \left[\frac{\log(t - t_0)}{4.437} \right]^{\min\left(\frac{RH}{75.1}\right)}$$

in which

t = Age of concrete (in days) at the time when creep is being estimated, and

t_0 = Age of concrete (in days) at the time of loading.

The notional creep coefficient φ_0 shall be estimated as:

$$\varphi_0 = \varphi_{RH} \beta(f_{cm}) \beta(t_0),$$

where

φ_{RH} = factor account for effect of relative humidity on notional creep coefficient

$$= \begin{cases} 1 + \frac{1 - RH/100}{0.1 \sqrt[3]{h_0}} & 45 \text{ MPa} \leq f_{ck} \\ \left[1 + \left[\frac{1 - RH/100}{0.1 \sqrt[3]{h_0}} \right] \alpha_1 \right] \alpha_2 & 45 \text{ MPa} > f_{ck} \end{cases},$$

$\beta(f_{cm})$ = Factor accounting for effect of concrete strength on φ_0

$$= \frac{16.8}{\sqrt{f_{ck} + 8}}, \text{ and}$$

$\beta(t_0)$ = Factor accounting for effect of age of concrete at loading on φ_0

$$= \frac{1}{0.1 + t_0^{0.2}}, \text{ and}$$

in which

RH = Relative Humidity of the ambient environment (in percent),

h_0 = Notional size (in mm) of the member = $2A_c/u$,

$$\alpha_1 = \left(\frac{45}{f_{ck} + 8} \right)^{0.7}, \text{ and}$$

$$\alpha_2 = \left(\frac{45}{f_{ck} + 8} \right)^{0.2},$$

wherein

A_c = Cross-sectional area of the member, and

u = Perimeter of the member in contact with the atmosphere.

7.1.4.5 Shrinkage strain

The total shrinkage of concrete depends upon the constituents of concrete, size of the member and environmental conditions.

7.1.4.5.1 The total shrinkage strain is composed of two components, the autogenous shrinkage strain and the drying shrinkage strain. The value of the total shrinkage strain ϵ_{cs} is given by:

$$\epsilon_{cs} = \epsilon_{cd} + \epsilon_{ca}$$

where,

ϵ_{cs} = total shrinkage strain

ϵ_{cd} = drying shrinkage strain

ϵ_{ca} = autogenous shrinkage strain

7.1.4.5.2 The autogenous shrinkage strain develops during hardening of concrete; the major part develops in the early days after casting. Autogenous shrinkage is a linear function of concrete strength. It should be considered specifically when new concrete is cast against hardened concrete. The development of autogenous shrinkage with time may be taken as:

$$\epsilon_{ca}(t) = \beta_{as}(t) \cdot \epsilon_{ca}$$

where $\beta_{as}(t) = 1 - \exp(-0.2\sqrt{t})$, t is age of concrete in days.

7.1.4.5.2.1 In the absence of accurate field/laboratory data, the values given below may be considered in design.

Grade of Concrete	Autogenous Shrinkage Strain ($\epsilon_{ca} \times 10^{-6}$)
M 30	35
M 35	45

M 45	65
M 50	75
M 60	95

7.1.4.5.3 The drying shrinkage strain develops slowly as it is a function of migration of the water through the hardened concrete.

The final value of the drying shrinkage strain $\epsilon_{cd,o}$ may be taken as equal to $k_h \epsilon_{cd}$. The development of the drying shrinkage strain in time may be taken as:

$$\epsilon_{cd}(t) = \beta_{ds}(t, t_s) k_h \epsilon_{cd}$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04 \sqrt{h_0^3}}$$

where

- t = age of the concrete at the moment considered, in days;
 t_s = age of the concrete at the beginning of drying shrinkage, in days; normally this is at the end of curing; and
 h_0 = notional size of the cross section, in mm
 = $2A_c / u$

k_h is a coefficient depending on the notional size h_0 , as given below:

h_0 (in mm)	k_h
100	1.0
200	0.85
300	0.75
500	0.70

7.1.4.5.3.1 Values of ϵ_{cd} may be taken from the table given below. These values are expected mean values with a coefficient of variation of about 30 percent.

f_{ck} (in MPa)	Unrestrained Drying Shrinkage Values ($\epsilon_{cd} \times 10^{-6}$) for Relative Humidity	
	50 percent	80 percent
(1)	(2)	(3)
25	535	300
50	420	240

75

330

190

NOTE – The values for the other designated grades may be obtained by interpolation.

7.1.4.6 Strength

The basic strengths of concrete shall be taken as specified hereunder, under direct compression, direct tension and bending tension. Implications of increase beyond 28 days in the strengths of concrete given hereunder shall be accounted for in design, when the same is detrimental or causes brittle actions to determine the behaviour.

7.1.4.6.1 Compressive strength

The characteristic compressive strength f_{ck} (in MPa) of concrete under direct compression shall be considered. The design should be based on 28-day characteristic strength of concrete, unless there is evidence to justify a higher strength for a particular structure due to age.

Mean compressive strength f_{cm} which is required to be adopted during construction, shall be estimated as:

$$f_{cm} = \text{Max}[f_{ck} + 1.65\sigma; f_{ck} + X],$$

where σ is standard deviation and X a constant specified in Table 11. In particular, when designing concrete mix, the *target strength* shall be taken as f_{cm} .

7.1.4.6.2 Direct tensile strength

The direct tensile strength f_{ct} of concrete shall be obtained by carrying out the direct tension test as specified in IS 516 (Part 1/Sec 1). In the absence of test data, f_{ct} can be approximately calculated by the following expression:

$$f_{ct} = \begin{cases} 0.80 f_{ct,sp} \\ 0.54 f_{cr} \end{cases},$$

where $f_{ct,sp}$ is the split tensile strength of concrete obtained by carrying out the split cylinder test as specified in IS 516 (Part 1/Sec 1), and f_{cr} is the modulus of rupture of concrete under bending tension by carrying out the flexural tension test as specified in IS 516 (Part 1/Sec 1).

7.1.4.6.3 Split tensile strength

The split tensile strength $f_{ct,sp}$ of concrete shall be obtained by carrying out the split cylinder test as specified in IS 516 (Part 1/Sec 1). In the absence of test data, the design value of $f_{ct,sp}$ can be approximated by:

$$f_{ct,sp} = 0.50\sqrt{f_{ck}}$$

When mean split tensile strength $f_{ct,sp,m}$ is required to be adopted during construction, it shall be estimated as:

$$f_{ct,sp,m} = \text{Max}[f_{ct,sp,m} + 1.65\sigma; f_{ct,sp,k} + 0.55],$$

where σ is standard deviation of split tensile strength.

In the absence of data, the mean value of $f_{ct,sp,m}$ may be taken as 1.4 times the characteristic value $f_{ct,sp}$.

7.1.4.6.4 Modulus of rupture

The modulus of rupture f_{cr} of concrete under bending tension shall be obtained by carrying out the bending tension test as specified in IS 516. In the absence of test data, the design value of f_{cr} can be approximated by:

$$f_{cr} = 0.7\sqrt{f_{ck}},$$

When mean flexural tensile strength $f_{cr,m}$ is required to be adopted during construction, it shall be estimated as:

$$f_{cr,m} = \text{Max}[f_{cr} + 1.65\sigma; f_{cr} + 0.55],$$

where σ is standard deviation of flexural tensile strength.

In the absence of data, the mean value $f_{cr,m}$ may be taken as 1.4 times the characteristic value f_{cr} .

7.1.4.6.5 Increase in strength with age

The design of new structures shall be based on 28 days characteristic strength of concrete only, unless there is an evidence to justify a higher strength for a particular structure due to age. Such cases shall be governed by the following:

- a) When increased strength is to be used, the rate of increase of compressive strength with age shall be based on actual investigations considering the curing procedure adopted at site and expected ambient temperatures. The actual strength achievable (or achieved) at time other than 28 days strength, but not at more than 91 days in case of slow setting concretes, can be chosen to base the design and construction

choices, if found more appropriate. This decision shall be based on achievement of early or delayed strength, and the age at which the first design load, apart from the self-weight, is expected to be resisted by the structure.

- b) For evaluation of strength or load carrying capacity and for retrofitting of existing structures, strength at ages other than 28 days may be used after ascertaining the actual strength, sustained load effect, state of cracking and fatigue effects, for which specialist literature may be referred.

7.1.4.7 Workability

The concrete mix proportions chosen should be such that the concrete is of adequate workability for the placing conditions of the concrete and can properly be compacted with the means available. Suggested ranges of workability of concrete measured in accordance with IS 1199 are given below:

<i>Sl No.</i>	<i>Placing Conditions</i>	<i>Degree of Workability</i>	<i>Slump</i>	<i>Compacting factor</i>
(1)	(2)	(3)	(4)	(5)
i)	1) Blinding concrete 2) Pavements using paving trains	Very Low	Slump shall not be relied upon (IS 1199).	0.75 -0.80
ii)	Mass concrete Lightly reinforced sections in slabs, beams, walls and columns Floors Hand placed pavements Canal Lining Strip footings	Low	25 – 75	0.8 – 0.90
iii)	Heavily reinforced sections in slabs, beams, walls and columns	Medium to high	75 – 100	0.90-0.95
iv)	Slip-form work, Pumped concrete, thin members	Medium to high	75 – 150	0.95-0.98
v)	Trench fill	High	100 – 150	0.95-1.0
vi)	Tremie concrete	Very High	Measurement of workability by determination	Not to be used

			of flow will be appropriate (as per IS 9103).	
vii)	Roller compacted concrete	Very very low	Zero slump concrete. Water content as per OMC for Max wet density	
NOTE 1 For most of the placing conditions, internal vibrators (needle vibrators) are suitable. 2 The diameter of the needle shall be determined based on the density and spacing of reinforcement bars and thickness of sections. 3 For tremie concrete, vibrators are not required to be used. 4 For workability of self compacting concrete, refer 12.5.1				

7.2 Reinforcing Steel

Reinforcing steel used in reinforced and prestressed concrete structures shall comply with the requirements specified hereunder. Further,

- a) All reinforcement shall be free from loose mill scales, loose rust and coats of paints, oil, mud or any other substances. Sand blasting is recommended to clean reinforcement.
- b) No reinforcement bar shall be coated with any material such as epoxy coated bars. Any such coated bar shall not be used in concrete structures.
- c) No reinforcement shall be welded.

7.2.1 Grades of steel reinforcing bars

7.2.1.1 The grade of the steel bars to be used at site shall be as given in the structural drawings. No change shall be made at site, unless it is approved specifically by the structural designer and owner.

7.2.1.2 The reinforcing steel bars used shall conform to IS 1786. The provisions of this standard shall be applicable to reinforcing steels of grades given in Table 14.

Table 14 Designated Grades of steel permitted as Reinforcing Steel Bars
(Clause 7.2.1.2)

Sl No.	Grade of steel	Specified Characteristic Yield Strength f_y (in MPa)
(1)	(2)	(3)
i)	Fe 250	250
ii)	Fe 415	415
iii)	Fe 500	500
iv)	Fe 550	550

7.2.2 Physical Properties

The salient physical properties of reinforcing steel bars shall be taken as specified hereunder.

7.2.2.1 Modulus of elasticity

The modulus of elasticity E_s of reinforcing steel of all grades shall be taken as 20,000 MPa.

7.2.2.2 Poisson's ratio

The poisson's ratio ν of reinforcing steel of all grades shall be taken as 0.3.

7.2.2.3 Coefficient of thermal expansion

The coefficient α of thermal expansion of reinforcing steel of all grades shall be taken as $1.2 \times 10^{-5}/^{\circ}\text{C}$.

7.2.2.4 Strength

7.2.2.4.1 The yield strength of steels for which there is no clearly defined yield point, shall be taken as the 0.2 percent proof stress, which represents that strength results in a residual plastic strain of 0.0020 when unloaded from it. For steels having definite yield points, the stress corresponding to the lower yield point shall be considered as the yield (tensile) strength. And, the ultimate (tensile) strength of steels shall be taken as the maximum stress that the steel will resist before rupture. Unless otherwise stated, strength refers to characteristic strength.

7.2.2.4.2 The following are the acceptance criteria for steel reinforcement:

- Actual yield strength (that is 0.2 percent proof strength) of reinforcement steel bars shall not exceed 1.2 times the specified characteristic yield strength f_y , and
- The ratio of the actual ultimate tensile strength and actual yield strength of reinforcing steel shall be at least 1.15, but not more than 1.25.

7.2.2.5 Elongation

All reinforcing steel shall comply with the elongation requirements specified in IS 1786.

7.3 Prestressing Materials**7.3.1 Prestressing Steel**

Prestressing steel shall be used only as specified hereunder:

- a) Bonded tendons in new concrete structures where it is required to resist applied loads,
- b) Bonded tendons (as internal prestressing, if possible) or unbonded tendons (as external prestressing, but with due precautions) in existing concrete structures where it is required to increase resistance of the structure, and
- c) Unbonded tendons, where it is required to resist effects of temporary loads during the construction of new concrete structures and to be removed after the said stage of construction.

7.3.1.1 Tendons

Prestressing tendons are made of single or group of wires or strands or bars. Three types of prestressing steels are permissible, namely plain or indented wires, stress-relieved multi-ply strands, and high tensile steel bars. These shall conform to the requirements given hereunder in addition to the Indian Standards listed in Table 15.

Table 15 Types of Prestressing Steel
(Clause 7.3.1.1)

Type	Name	Reference Standard
(1)	(2)	(3)
Wires	Plain cold drawn stress-relieved wires	IS 1785 (Part 1)
	Indented hard-drawn stress-relieved wires	IS 6003
Strands	Stress relieved multi-ply strands of Normal Relaxation	IS 6006
	Stress-relieved multi-ply strands of Low Relaxation	IS 14268
Bars	High tensile steel bars	IS 2090

a) *Wires*

The wires used in prestressing shall comply with the following requirements:

- 1) The minimum elongation of wires at fracture shall be as specified in Table 16 and elongation shall be measured over a gauge length of 200 mm.
- 2) The 1,000 hour relaxation (tested at an initial stress of 0.7 times the ultimate tensile strength at 20°C) shall not be more than 5 percent of 0.7 times the UTS.
- 3) For acceptance of test results from a lot, the value calculated, as arithmetic mean minus 0.6 of the range of test results, shall be more than the minimum tensile strength and minimum elongation at fracture specified in Table 16.
- 4) 0.2 percent proof stress shall not be less than 85 percent of minimum tensile strength specified in Table 16.
- 5) The wires conforming to Indian Standards can be provided with protective coatings, like galvanizing or epoxy coating, carried out in specialized manufacturing units. But, if the manufacturing processes affect any of the mechanical and physical properties, such properties shall be considered in design calculations.

Table 16 Minimum Elongation needed in Hard Drawn Stress Relieved Wires
[Clause 7.3.1.1(a)]

Type	Diameter (in mm)	Minimum Tensile Strength (in MPa)	Minimum Elongation at Fracture (in percent)
(1)	(2)	(3)	(4)
Plain Wires	4	1715	3.0
	5	1570	4.0
	7	1470	4.0
	8	1375	4.0
Indented Wires	4	1715	3.0
	5	1570	4.0

b) *Strands*

The strands used in prestressing shall comply with the following requirements:

- 1) The minimum elongation of the strands at fracture shall be as specified in Table 17. The elongation shall be measured over a gauge length of 600 mm. The elongation measured immediately before fracture of any of the constituent wires over the said gauge length shall not be less than 3.5 percent of the gauge length.
- 2) The 1,000 hour relaxation (tested at an initial stress of 0.7 times the UTS at 20°C) shall not be more than 5 percent of 0.7 times the UTS in normal

- relaxation steel and 2.5 percent of 0.7 times the UTS in low relaxation steel.
- 3) For acceptance, all samples tested from a batch shall meet requirement of minimum breaking load and proof load as specified in Table 17.
 - 4) The strands conforming to Indian Standards can be provided with protective coatings, like galvanizing or epoxy coating, carried out in specialized manufacturing units. But, if the manufacturing processes affect any of the mechanical and physical properties, such properties shall be considered in design calculations.

Table 17 Minimum Breaking Load and Proof Load needed in Stress Relieved Strands
[Clause 7.3.1.1(b)]

Class	Designation	Nominal Area (in mm ²)	Normal Relaxation		Low Relaxation	
			Breaking Load (in kN)	0.2 percent Proof Load (in kN)	Breaking Load (in kN)	0.2 percent Proof Load (in kN)
I	11.1 mm 7 ply	70.0	124.54	105.86	120.1	108.00
	12.7 mm 7 ply	92.9	166.18	139.6	160.1	144.1
	15.2 mm 7 ply	139.0	226.86	192.83	240.2	216.2
II	11.1 mm 7 ply	74.2	137.89	117.21	137.9	124.1
	12.7 mm 7 ply	98.8	183.71	156.11	183.7	165.3
	15.2 mm 7 ply	140.0	261.44	222.23	260.7	234.6

c) *Bars*

The bars used in prestressing shall comply with the following requirements:

- 1) The minimum elongation at fracture of the high tensile steel bars shall be as specified in Table 18. The elongation shall be measured over a gauge length of $5.65\sqrt{A}$, where A is the area of cross-section of steel bar. The elongation measured immediately before fracture shall not be less than 10 percent of the gauge length.
- 2) The 1,000 hour relaxation (tested at an initial stress of 0.7 times the UTS at 20°C) shall not be more than 49 MPa.
- 3) For acceptance of test results from a lot, a value calculated as arithmetic mean minus 0.6 of the range of test results shall be more than the minimum strength and elongation as specified in Table 18.

Table 18 Minimum Elongation, Specified Tensile Strength and Proof Strength in High Tensile Steel Bars
[Clause 7.3.1.1(c)]

Diameter (in mm)	Minimum Elongation (in percent)	Minimum Specified Tensile Strength (in MPa)	Minimum Proof Strength
(1)	(2)	(3)	(4)
10, 12, 16, 20, 22, 25, 28, 32	?? (to be discussed)	980	85 percent of specified tensile strength

7.3.1.2 Physical properties

The salient physical properties of prestressing steel tendons shall be taken as specified hereunder.

7.3.1.2.1 Modulus of elasticity

The modulus of elasticity E_p of prestressing steel shall be taken as per manufacturer's test reports for the lot. The value of E_p used in design shall conform to the manufacturer's test value subject to complying with the acceptance criteria. The maximum and minimum value of modulus of elasticity obtained from acceptance tests shall be within ± 2.5 percent of the values given in manufacturer test report.

When values are not available from the manufacturer, tests shall be performed to ascertain the same. In general, the values shall be within $+ \dots \%$ of the values indicated hereunder:

$$E_p = \begin{cases} 210,000 \text{ MPa} & \text{for Plain Cold Drawn Wires} \\ 200,000 \text{ MPa} & \text{for High Tensile Steel Bars (rolled or heat treated)} \\ 195,000 \text{ MPa} & \text{for Strands} \end{cases} .$$

7.3.1.2.2 Poisson's ratio

The poisson's ratio ν of prestressing steel of all grades shall be taken as 0.3.

7.3.1.2.3 Coefficient of thermal expansion

The coefficient α of thermal expansion of prestressing steel of all grades shall be taken as $1.2 \times 10^{-5} / ^\circ\text{C}$.

7.3.1.2.4 Strength

The tensile yield stress of steels for which there is no clearly defined yield point, shall be taken as the 0.2 percent proof stress. Strength refers to characteristic strength f_y , and shall mean that the value of 0.2 percent proof stress of the reinforcing steel wires, bars or strands below which not more than 5 percent of the test results are expected to fall.

7.3.1.2.5 Stress relaxation

The stress relaxation shall be obtained from the Manufacturers Test Certificates, and verified independently through tests, if required. In absence of actual test data, the design value of stress relaxation up to 30 years shall be taken as three times the relaxation value at 1,000 hours, which is tested at an initial stress of 0.7 times the UTS at 20°C. For initial stress other than 0.7 times the UTS, the values given in Table 19 shall be used. For test durations less than 1,000 hours, the maximum values of relaxation loss may be taken as per Table 20.

Table 19 Stress relaxation for different Initial Stresses (expressed as percent of initial stress tested at 1,000 hours at 20°C ± 2°C)
(Clause 7.3.1.2.5)

Sl No.	Initial Stress (in MPa)	Relaxation Loss (in percent)	
		Normal Relaxation Steel	Low Relaxation Steel
(1)	(2)	(3)	(4)
i)	$\leq 0.5 f_p$	0	0
ii)	$0.6 f_p$	2.5	1.25
iii)	$0.7 f_p$	5.0	2.50
iv)	$0.8 f_p$	9.0	4.50

Table 20 Stress relaxation up to 1,000 hours (expressed as percent of initial stress tested at 1,000 hours at 20°C ± 2°C)
(Clause 7.3.1.2.5)

Sl No.	Time (in hours)	Loss of the initial stress at 1000 hours (in percent)	
		Normal Relaxation Steel	Low Relaxation Steel
(1)	(2)	(3)	(4)
i)	1	34	37
ii)	5	44	47
iii)	20	55	57

iv)	100	70	72
v)	200	78	79
vi)	500	90	90
vii)	1000	100	100

The stress relaxation $\rho(t)$ at time t (hours) other than at 1,000 hours shall be estimated as:

$$\rho(t) = \rho_{1000} \left(\frac{t}{1000} \right)^k,$$

where

ρ_{1000} = stress relaxation value at 1000 hours as per Table 20, and

$$k = \left(\frac{\rho_{1000}}{\rho_{100}} \right)$$

$$= \begin{cases} 0.155 & \text{Normal Relaxation Steel} \\ 0.143 & \text{Low Relaxation Steel} \end{cases}$$

In pre-tensioned members, stress relaxation shall be considered owing to temperature increase during their curing, which induced thermal strain in members. Hence, for capturing early age relaxation, arising from initial temperatures higher than 40°C (as in steam curing), an equivalent time t_{eq} (hours) shall be added to the time t (hours) after tensioning in the relaxation time to account for the effects of the heat treatment on the loss in prestress in the relaxation of the prestressing steel. t_{eq} shall be estimated as:

$$t_{eq} = \frac{1.14^{(T_{\max} - 20)}}{T_{\max} - 20} \left[\sum_{i=1}^n \{ (T_{\max} - 20) \Delta t_i \} \right],$$

where

$T_{\max}$ = Maximum temperature (in °C) during the curing at elevated temperatures,

Δt_i = Time interval for which the Temperature (in °C) is maintained, and

n = Total number of time intervals considered.

7.3.1.3 Untensioned steel

In general, untensioned steel shall conform to the requirements as per 9.3.4. Requirements of cover and spacing between bars shall apply as specified in 9.2.1.5 and 9.2.2.5, and of provisions for assembly of reinforcement as specified in 10.5.3.

7.3.2 Sheathing Ducts and Joints

The sheathing ducts shall:

- a) be made of either mild steel or HDPE conforming to relevant Indian Standards, having a test certificate furnished by the manufacturer;
- b) be in as long lengths as practical from handling and transportation considerations without getting damaged;
- c) be watertight at its internal joints when bent to the minimum radius of bending required;
- d) conform to the requirements specified in ____ and ____, and the tests specified in ____ shall be performed as part of additional acceptance tests for prestressing systems employing corrugated HDPE sheathing ducts and are not meant for routine site testing purposes and
- e) have smooth internal surfaces.

The joints in sheathing ducts shall:

- a) Be joined by any of the following methods, subject to satisfactory pressure tests, before prestressing:
 - 1) Corrugated threaded sleeve couplers, which can be tightly screwed to the outside of the sheathing ducts;
 - 2) Welding using electric roaster machine or mirror machine; and
 - 3) Heat shrink couplers; and
- b) Be leak-tight against water pressure of 0.05 MPa for 5 minutes as per test procedure given in Annex.

7.3.2.1 *Metallic ducts*

Unless specified otherwise, the material of metal sheathing shall be:

- a) Cold rolled cold annealed (CRCA) mild steel intended for mechanical treatment and surface refining, but not for quench hardening or tempering;
- b) Clean and free from rust, and normally of bright metal finish;
- c) Galvanized or used with lead coated mild steel strips when used in aggressive environment; and
- d) Of thickness not less than 0.3mm, 0.4mm and 0.5mm for sheathing ducts having internal diameter up to 50mm, 75mm and 90 mm, respectively; for ducts of larger diameters, the thickness shall be as recommended by prestressing system supplier.

7.3.2.2 *Corrugated High Density Poly-Eurythane Ducts*

Unless specified otherwise, the material of HDPE sheathing shall:

- a) Be made of high density polyethylene with more than 2 percent carbon black to provide resistance to ultraviolet degradation;
- b) Of manufactured wall thickness of at least 2.0 mm, 2.5 mm, 3 mm, and 4 mm for circular ducts of internal diameter 50 mm, 85 mm, 100 mm and 125 mm, respectively, and of at least 2.0 mm for flat ducts of any size. Linear interpolation may be done for any intermediate values.
- c) Have tolerance for duct diameter shall be larger of ± 1 percent and ± 1 mm, and for wall thickness -0/+0.5 mm;
- d) Corrugated on both sides for internally bonded tendons; and
- e) Transmit 40 percent of ultimate tendon strength from the tendon to the surrounding concrete over a length not greater than 16 times duct diameter.

7.3.2.3 Tests on mild steel and corrugated HDPE sheathing ducts

The following tests shall be performed on ducts to ascertain their suitability:

7.3.2.3.1 Mild steel sheathing ducts

All tests specified below shall be carried out on the same sample and in the order given below. At least 3 samples shall be tested for one lot of supply (not exceeding 7,000m in length).

a) Workability Test

A test sample 1,100 mm long shall be soldered to a fixed base plate with a soft solder (see Fig. 2). Then, it shall be bent to a radius of 1,800 mm alternately on either side to complete 3 cycles. Thereafter, the sealing joints shall be visually inspected to verify that no failure or opening has taken place.

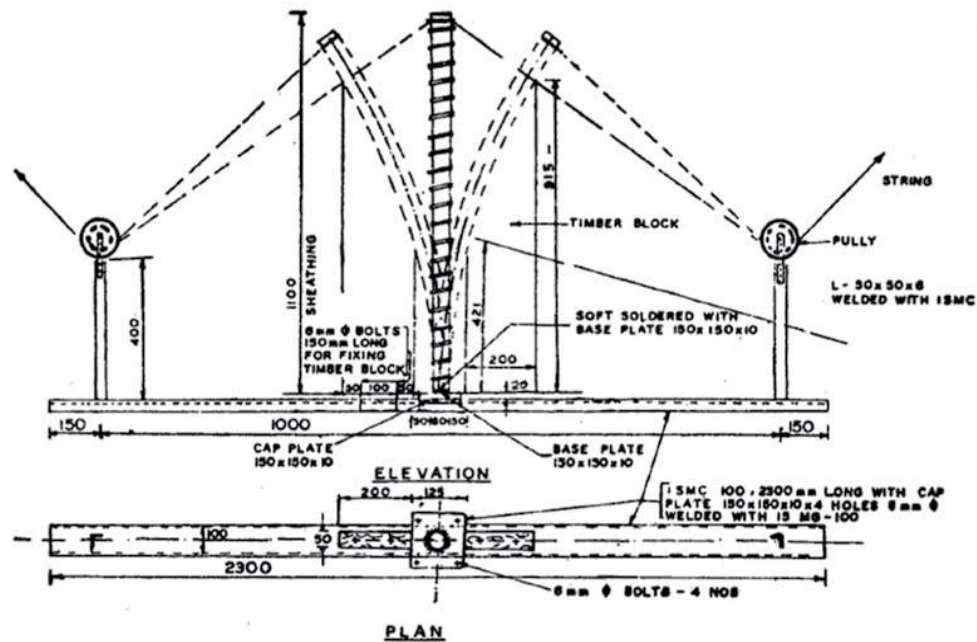


FIG. 2 TEST SET-UP OF WORKABILITY TEST OF A MILD STEEL SHEATHING DUCT

b) *Transverse Load Rating Test*

A test sample (500 mm long) shall be placed on a horizontal support, so that the sample is supported at all points of outward corrugations (see Fig. 3), and loaded gradually in increments (as per Table 21) at the centre of the supported portion through a circular contact surface of 12 mm diameter. Couplers shall be placed so that the load is applied approximately at the centre of two corrugations (see Fig. 3).

The sample is said to be acceptable, if the permanent deformation is less than 5 percent of the original diameter.

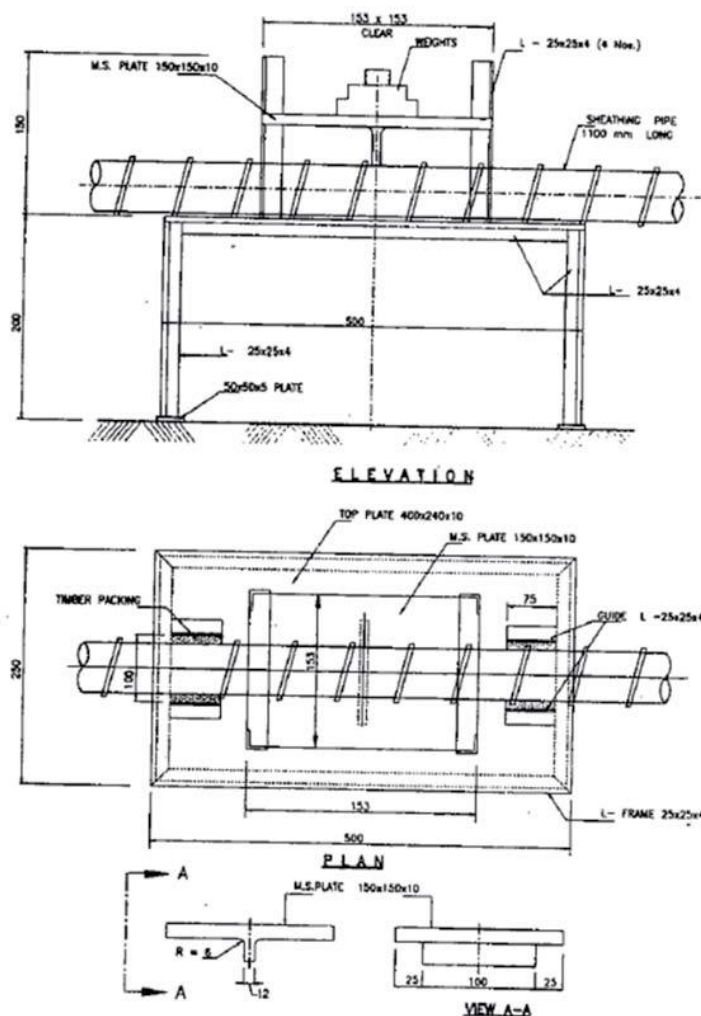


FIG. 3 TRANSVERSE LOAD TEST SET-UP

Table 21 Load to be applied Transversely to test the Capacity of Ducts
[Clause 7.3.2.3.1(b)]

Diameter (mm)	Transverse Load (kN)
$25 \leq \phi < 35$	250
$35 \leq \phi < 45$	400
$45 \leq \phi < 55$	500
$55 \leq \phi < 65$	600
$65 \leq \phi < 75$	700
$75 \leq \phi < 85$	800
$85 \leq \phi < 90$	1,000

c) Tension Load Test

A test sample (1,100 mm long) shall be hung from a vertical support, so that the sample is supported at all points of outward corrugations (see Fig. 4). The hollow core is filled with a wooden circular piece having a diameter of 95 percent of the inner diameter of the sample to maintain circular profile during test loading (see Fig. 4). And, the duct is loaded gradually in increments (as per Table 22) at the bottom end. Couplers shall be screwed on the duct (see Fig. 4).

The sample is said to be acceptable, if there is no deformation of the joints or slippage of couplers.

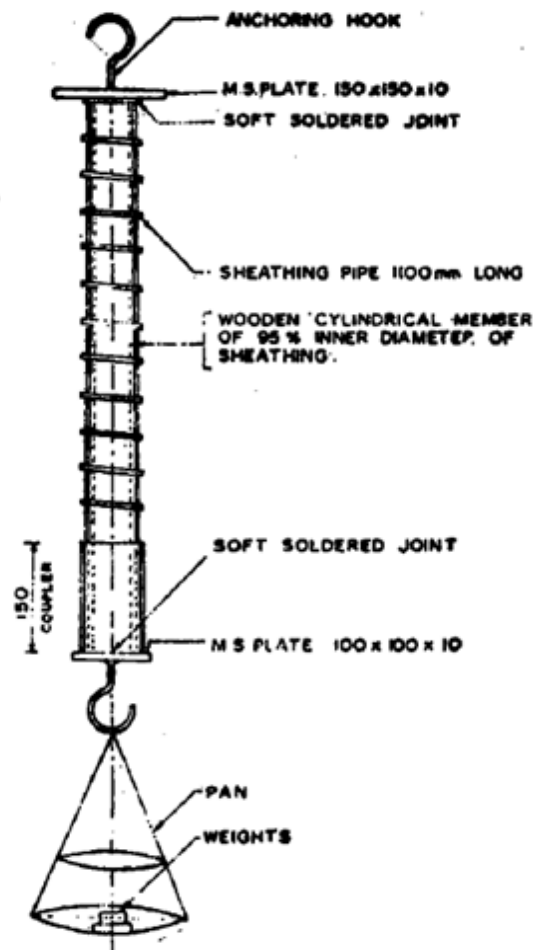


FIG. 4 AXIAL LOAD TEST SET-UP

Table 22 Load to be applied Axially to test the Capacity of Ducts
(Clause 7.3.2.3)

Diameter (mm)	Axial Load (kN)
$25 \leq \phi < 35$	300
$35 \leq \phi < 45$	500
$45 \leq \phi < 55$	800
$55 \leq \phi < 65$	1,100
$65 \leq \phi < 75$	1,400
$75 \leq \phi < 85$	1,600
$85 \leq \phi < 90$	1,800

d) *Water Loss Test*

A sample (1,100 mm long) shall be sealed at one end, the other end is connected to a system capable of applying a pressure of 0.05 MPa, and the sample is filled with water (see Fig. 5). The water pressure is maintained at that pressure (using a hand pump and pressure gauge, or a stand pipe system) (see Fig. 5).

The sample is said to be acceptable, if the water loss does not exceed 1.5 percent of the volume of the sample.

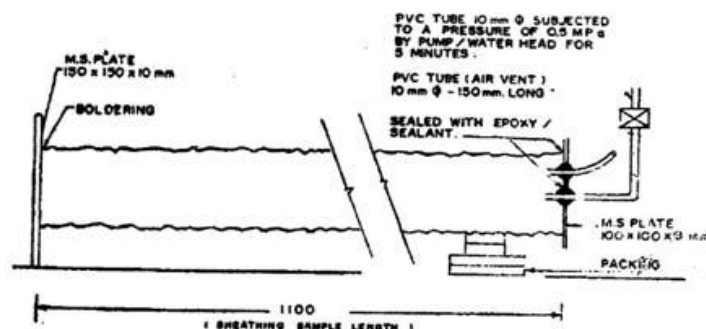


FIG. 5 WATER LOSS TEST SET-UP

The actual volume V_p of water filled in the sample is estimated as:

$$V_p = V_a - V_b,$$

where

V_a = Pre-measured quantity of water in measuring cylinder before filling the sample, and

V_b = Balance quantity of water left in the cylinder after completely filling of the test sample.

The relative profile volume ρ_v shall be estimated as:

$$\rho_v = V_p - \frac{\left(\frac{\pi \phi^2 L}{4} \right)}{\pi \phi}$$

where

L = Length of duct specimen, and

ϕ = Internal nominal diameter.

7.3.2.4 Sheathing duct joints

7.3.2.4.1 The sheathing ducts shall be of the spiral corrugated type. For major projects, the sheathing ducts shall be manufactured preferably at the project site utilizing appropriate machines. With such an arrangement, long lengths of sheathing ducts may be used with consequent reduction in the number of joints and couplers. Where sheathing duct joints are unavoidable, such joints shall be made cement slurry tight by the use of corrugated threaded sleeve couplers which can be tightly screwed on to the outer side of the sheathing ducts. A heat-shrink coupler also could be used, if suitable.

7.3.2.4.2 The sleeve coupler (*see* Fig. 6) shall not have a length of the coupler less than 150mm, but should be increased up to 200 mm wherever practicable. The joints between the ends of the coupler and the duct shall be sealed with adhesive sealing tape to prevent penetration of cement slurry during concreting. The couplers of adjacent ducts shall be staggered, wherever practicable. As far as possible, couplers shall not be located in curved zones. The corrugated sleeve couplers are being manufactured conveniently using the sheath making machine with the next higher size of die set.

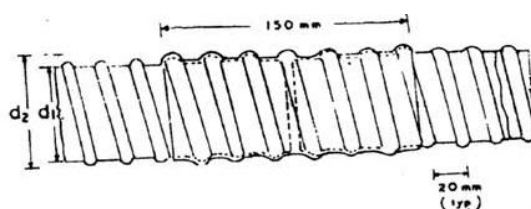


FIG. 6 TYPICAL DETAILS OF SLEEVE COUPLER

7.3.2.4.3 The heat-shrink coupler (*see* Fig. 7) is supplied in the form of bandage rolls, which can be used for all diameters of sheathing ducts. The bandage is coated on the underside with a heat sensitive adhesive so that after heating the bandage material shrinks in to the sheathing and ensures formation of a leak proof joint, without the need for extra taping or support in the form of corrugated sleeve couplers. A soft gas flame is used to heat the coupler.

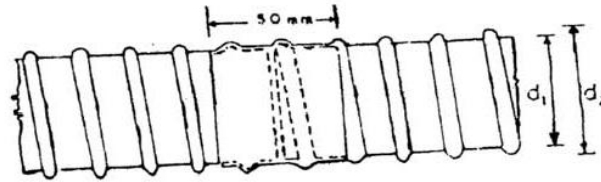


FIG. 7 TYPICAL DETAILS OF HEAT SHRINK COUPLER

7.3.2.4.3.1 Corrugated HDPE sheathing ducts

A summary of all the requirements, methods of verification, and acceptance criteria for the polymer duct system together with the components and material is given in **Table 21**.

7.3.3 Mechanical Couplers

7.3.3.1 Fixed or movable type mechanical couplers shall:

- a) Be placed to not adversely affect the load carrying capacity of the member;
- b) Meet the specifications laid down by the manufacturer for minimum concrete cover over couplers and reinforcement for bursting and spalling, but subjected to acceptance tests similar to those specified in **clause** for anchorages;
- c) Be located away from intermediate supports;
- d) Not be used in more than on 50 percent of the tendons at any cross-section;
- e) Be spaced such that the distance between any two successive sections at which cables are coupled is not closer than 1.5m in structural members whose depth is less than 2.0m, but not closer than 3.0m when depth is more than 2.0m;
- f) Meet the requirements of strength of individual anchorages as specified in 9.2.2.7; and
- g) Be able to transfer full force of tendon from one to another.

7.3.3.2 Also, the anchorage and stressing of second tendon shall not disturb the anchorage of the first tendon when fixed couplers are used.

7.4 Prestressing Systems

Prestressing systems manufactured by specialist manufacturers shall be compatible with the standard prestressing wires/strands used. The stressing equipment, de-stressing/restressing facilities, and grouting arrangements shall be compatible with the tendons.

Prestressing system proposed to be adopted should be well established (nationally and internationally), tested and approved. For any new or lesser used systems, specialist literature shall be referred for deciding the acceptance criteria.

7.4.1 Pre-tensioned Systems

7.4.1.1 Anchorages

Anchorages shall:

- a) Enable the full design strength to be developed in the tendons when used with appropriate lengths in pre-tensioned tendons and have sufficient (see Fig. 8):

- 1) Transmission length L_{pt} over which the prestressing force P_0 is transmitted fully to concrete,
- 2) Dispersion length L_{disp} over which the prestressing force P_0 is dispersed gradually to concrete, where

$$L_{disp} = \sqrt{L_{pt}^2 + d^2}, \text{ and}$$

- 3) Anchorage length L_{bpd} over which the design tendon force P_{pd} is developed in concrete.
- b) Meet the requirements specified in **Clause _____**
 - c) Not be permitted in top surface of the deck, and
 - d) Be introduced only after examining their consequences, if needed temporarily only during construction.
 - e) When tendons are anchored:
 - 1) at a construction joint or within a concrete member (whether on an external rib, within a pocket or entirely inside the member), the minimum residual compressive stress shall be checked to be at least 3 MPa in the direction of the anchored prestressing force. If the minimum residual stress is lesser, reinforcement shall be provided to cater to the local tension developed near the anchor beyond the tendon that is terminated. This check is not required, if the tendon is coupled at the anchorage considered.
 - 2) in the deck slab and soffit slab, the slab shall be thickened locally or provided with blisters so that the minimum concrete cover to anchorage is 200 mm.

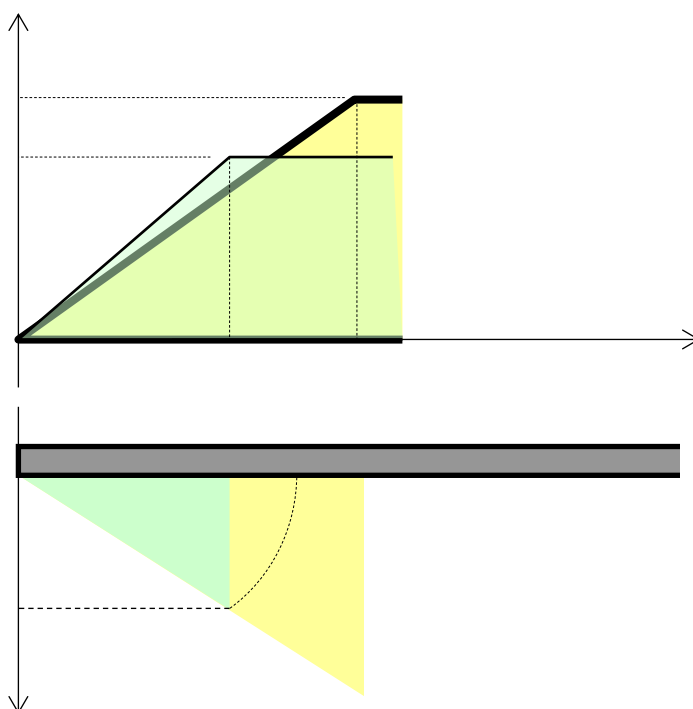


FIG. 8 LENGTH PARAMETERS IN TRANSFER OF PRESTRESS IN PRE-TENSIONED ELEMENTS

7.4.1.2 Transfer of prestress

- a) The prestress shall be assumed to be transferred to concrete by a constant bond stress f_{bpt} , given by

$$f_{bpt} = \eta_t \eta_b f_{ctd}$$

where

η_t = Coefficient accounting for type of tendon and bond with concrete

$$= \begin{cases} 2.7 & \text{Indented Wires} \\ 3.0 & \text{3 and 7 wire strands} \\ \text{From Actual Tests} & \text{Others} \end{cases}$$

η_b = Coefficient accounting for quality of bond with concrete

$$= \begin{cases} 1.0 & \text{Good bond conditions} \\ 0.7 & \text{Others} \end{cases}$$

$$f_{ctd}(t) = \text{Design tensile stress at time of release time } t$$

$$= 0.7 \left(\frac{0.5 \sqrt{f_{ck}}}{\gamma_M} \right),$$

- b) The tendons shall be provided with a design transmission length L_{pd} , given by

$$L_{pd} = \begin{cases} 0.8L_{bpt} & \text{Local stress at release} \\ 1.2L_{bpt} & \text{Higher for Limit State of Safety (shear, anchorage, etc.)} \end{cases}$$

where

$$L_{bpt} = \text{Basic transmission length} = \alpha_1 \alpha_2 \phi \left(\frac{f_{pm0}}{f_{pmt}} \right),$$

$$\alpha_1 = \begin{cases} 1.00 & \text{Gradual Release} \\ 1.25 & \text{Sudden Release} \end{cases},$$

$$\alpha_2 = \begin{cases} 0.25 & \text{Tendons with circular cross-section} \\ 0.19 & \text{3- and 7-wire strands} \end{cases},$$

ϕ = Nominal diameter of tendon, and

f_{pm0} = Tendon stress just after release.

- c) The concrete stress shall be assumed to be linearly distributed outside the dispersion length L_{disp} , and
- d) The stress in concrete may be taken different from the above, if adequately justified and if the transmission length is modified accordingly.

7.4.1.3 Anchorage of tensile force for the ultimate limit state:

- a) The anchorage of tendons shall be checked at sections where the tensile stress in concrete exceeds $0.5\sqrt{f_{ck}}$. The tendon force shall be calculated at a cracked section, including the effect of shear. When the tensile stress in concrete is less than $0.5\sqrt{f_{ck}}$, no anchorage check need to be performed.
- b) The bond strength for anchorage in the ultimate limit state is:

$$f_{bpd} = \eta_t \eta_b f_{ctd}, \text{ and}$$

where

η_t = Coefficient accounting for type of tendon and bond with concrete

$$= \begin{cases} 1.4 & \text{Indented Wires} \\ 1.2 & \text{7 wire strands} \\ \text{From Actual Tests} & \text{Others} \end{cases}$$

η_b = Coefficient accounting for quality of bond with concrete

$$= \begin{cases} 1.0 & \text{Good bond conditions} \\ 0.7 & \text{others} \end{cases}$$

$f_{ctd}(t)$ = Design tensile stress at time of release time t

$$= 0.7 \left(\frac{0.5 \sqrt{f_{ck}}}{\gamma_M} \right),$$

- c) The design tensile strength of concrete shall be limited to 3 MPa in concretes of higher strengths.
- d) The total anchorage length L_{bpd} for anchoring a tendon carrying a stress f_{pd} is (see Fig. 9):

$$L_{bpd} = L_{pt2} + \alpha_2 \phi \left(\frac{f_{pd} - f_{pd\infty}}{f_{bpd}} \right),$$

where

L_{pt2} = Upper value of design transmission length L_{pd} ($= 1.2L_{bpt}$),

f_{pd} = Stress in tendon given by (i) above, and

$f_{pd\infty}$ = Stress in tendon after all losses.

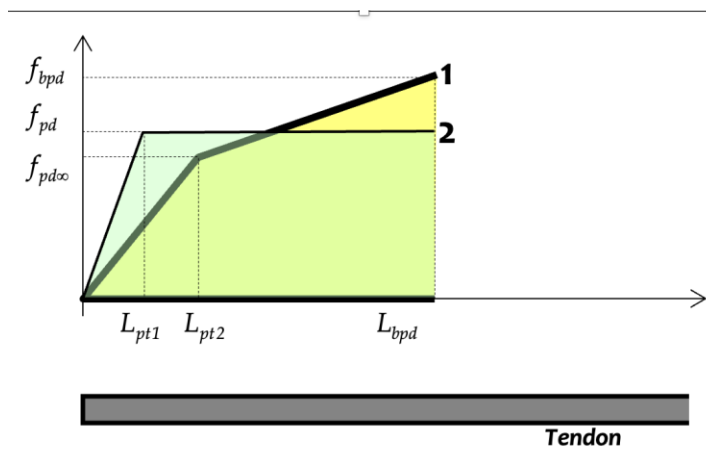


FIG. 9 STRESSES IN ANCHORAGE ZONE OF PRE-TENSIONED MEMBERS: (1) AT RELEASE OF TENDONS, AND (2) AT ULTIMATE LIMIT STATE

- e) When reinforcing and pre-tensioned steels are used together within the same zone of concrete, the anchorage capacities of each shall be calculated separately and added for design verification.

7.4.1.4 *Deviators*

7.4.1.4.1 A deviator shall withstand both longitudinal and transverse forces applied by tendon and transmit them to the structure; and ensure that the radius of curvature of the prestressing tendon does not cause any overstressing or damage to it.

7.4.1.4.2 In a deviation zone,

- a) tubes forming the sheaths shall be able to sustain the radial pressure and longitudinal movement of the prestressing tendon, without damage and without impairing its proper functioning; and
- b) radius of curvature of the tendon shall not be less than 40 times the diameter of wire/strand.
- c) designed angle of deviation of tendon shall be limited to 0.01 radians when no deviator is used. But, the forces developed by the change of angle shall be considered in design.

7.4.2 *Post-tensioned Systems*

7.4.2.1 The anchorages in post tensioned system shall be any one of the following types:

- a) Anchorages (partially or fully embedded in concrete) in which the prestressing force of tendons is transferred within the body of the prestressed element by combination of bearing, friction and wedge action; and
- b) Anchorages (externally mounted) in which the prestressing force of tendons is transferred to concrete through an externally mounted bearing plate.

7.4.2.2 The anchorage shall meet the following requirements:

- a) The anchorage may consist of any device patented or otherwise, which complies with the requirements laid down in **9.2.2.8.1** to **9.2.2.8.3**. Proprietary anchorages shall be handled and used strictly in accordance with the manufacturer's instructions and recommendations.
- b) The anchorage shall be capable of holding (with nominal slip) the prestressing tendon carrying a load corresponding to average of the design initial prestressing load and the ultimate strength of the prestressing tendon.

- c) The anchorage shall transfer effectively and distribute, as evenly as possible, the entire force from the prestressing tendon to the concrete without inducing undesirable secondary or local stresses.
- d) The anchorage shall be safe and secure against static load, dynamic load and impact load.
- e) The anchorage shall have provision for introducing a suitable protective medium, such as cement grout, for the protection of the prestressing steel, unless alternate arrangements are made.

7.4.2.3 The anchorage shall be capable of:

- a) Holding and transferring a force not less than 95 percent of the actual mean tensile ultimate strength of the tendon, without failure of any part of the anchorage-tendon assembly.
- b) Withstanding not less than 20 lakh cycles of fatigue load less than 60 percent of nominal UTS of tendons it is expected to hold, without suffering more than 5 percent breakage of wires or strands at the load frequency of not more than 500 cycles per minute.

7.5 Prestressing Grout

Unbonded tendons placed either in ducts embedded in concrete or located outside the member shall be protected from corrosion by suitable fillers. Grouting by cement, wax, and nuclear grade (low sulphur) grease are some of the options. For materials other than cement or such long-life permanent materials, arrangements shall be made for inspection and refilling or replacement of grouting materials.

Post-tensioned tendons shall be bonded to the concrete of prestressed member using a *grout*, whose properties comply with requirements specified in Table 23. Also, the grout placed in the duct shall meet the requirements specified in Table 24.

Table 23 Typical properties of Post-Tensioned Grouts in Steel Ducts*(Clause 7.5)*

Sl No.	Parameter	Value	Test Method
(1)	(2)	(3)	(4)
1	<i>Efflux Time (s)</i>		Marsh Cone Test
	(a) Immediately after mixing	15 - 20	
	(b) 30 minutes after mixing	18 - 23	
	(c) 180 minutes after mixing	25 - 30	
2	<i>Spread Diameter (mm)</i>		Spread Test
	(a) immediately after mixing	155 - 160	
	(b) 30 minutes after mixing	145 - 150	
	(c) 180 minutes after mixing (mm)	130 - 135	
3	<i>Bleed Water Volume (ml)</i>		Inclined Tube Bleed Test
	(a) Standard	0	
	(b) Wick-induced	0	
	(c) Pressure-induced	0	
	(d) Inclined tube	0	
4	<i>Volume of Soft Grout (ml)</i>	0	Visual Observation and Extraction by Scraping
5	<i>Setting Time (hours)</i>		?? (to be discussed)
	(a) Initial	7 - 10	
	(b) Final	15 - 17	
6	<i>Change in length (%)</i>		?? (to be discussed)
	(a) In sealed condition on 28 days	0 - 0.016	
	(b) 28 days	0 - 0.030	
7	<i>Cube Compressive Strength (MPa)</i>		?? (to be discussed)
	(a) 3 days	> 20	
	(b) 7 days	> 35	
	(c) 28 days	> 40	

Table 24 Specifications for Grouts to be used in Post-tensioned Systems
(Clause 7.5)

Sl No.	Parameter	Acceptance criteria	Lab Tests	Field (Site) Tests
(1)	(2)	(3)	(4)	(5)
i)	Efflux Time $T_{e,0}$ (s) immediately after mixing	≤ 25		✓
ii)	Efflux time $T_{e,30}$ (s) 30 minutes after mixing	$1.2T_{e,0} \geq T_{e,30} \geq 0.8T_{e,0}$ $T_{e,30} \leq 25$	✓	
iii)	Efflux time $T_{e,180}$ (s) 180 minutes after mixing	$1.4T_{e,0} \geq T_{e,180} \geq 0.8T_{e,0}$ $T_{e,180} \leq 25$	✓	
iv)	Spread diameter $D_{s,0}$ (mm) immediately after mixing	≥ 140		✓
v)	Spread diameter $D_{s,30}$ (mm) 30 minutes after mixing	$1.2D_{s,0} \geq D_{s,30} \geq 0.8D_{s,0}$ $D_{s,30} \geq 140$	✓	
vi)	Spread diameter $D_{s,180}$ (mm) 180 minutes after mixing	$1.4D_{s,0} \geq D_{s,180} \geq 0.6D_{s,0}$ $D_{s,180} \geq 140$	✓	
vii)	Standard bleed volume BV_{Standard} (%)	≤ 0.10		✓
viii)	Wick-induced bleed volume BV_{Wick} (%)	≤ 0.30		✓
ix)	Pressure-induced bleed volume $BV_{\text{Pressure, 350}}$ (%)	≤ 0.10	✓	
x)	Incline tube bleed volume BV_{Inclined} (%)	≤ 0.30	✓	
xi)	Prototype tendon grouting bleed volume $BV_{\text{Prototype grouting}}$ (%)	≤ 0.30	✓	
xii)	Change in length ΔL (%)	-0.02% to 0.02 on 28 days	✓	
xiii)		≤ 0.01 after 1 day	✓	
xiv)	Volume of soft grout $V_{\text{softgrout}}$	0	✓	✓

Factory made coated wires or strands embedded in *polyethylene ducts* with suitable fill also are acceptable. When such specialized materials and techniques of prestressing are employed, manufacturer's recommendations shall be complied with.

Grout is a homogenous mixture of cement and water. It may contain chemical admixtures which modify the properties of grout in its fluid state. These recommendations cover the cement grouting of post tensioned tendons of prestressed concrete members.

7.5.1 *Constituent Materials*

Cement grouts shall be made with cement, water and admixtures, whose specifications are as hereunder:

- a) *Water*: Only clean potable water free from impurities shall be used. No water from sea, creeks or effluents shall be used.
- b) *Cement*: The same cement as used in construction of prestressed elements, shall be used in the preparation of the grout.
- c) *Admixtures*: Acceptable admixtures conforming to IS 9103 may be used, if tests show that their use improves the properties of grout, that is, increases fluidity, reduces bleeding, entrains air and expands the grout. Admixtures shall not contain chlorides, nitrates, sulphides, sulphites or any chemical that is likely to damage the steel or the grout. When an expanding agent is used, the total unrestrained expansion shall not exceed 10 percent. Aluminum powder shall not be used as an expanding agent.
- d) *Sand*: It is not recommended to use sand for grouting of prestressing tendons.

7.5.2 *Use of Grout Colloidal Mixer*

The grout shall be continuously mixed in a colloidal mixer with a minimum speed of 1000 RPM, and with a travel discharge not exceeding 15 m/s.

7.5.3 *Properties of Grout*

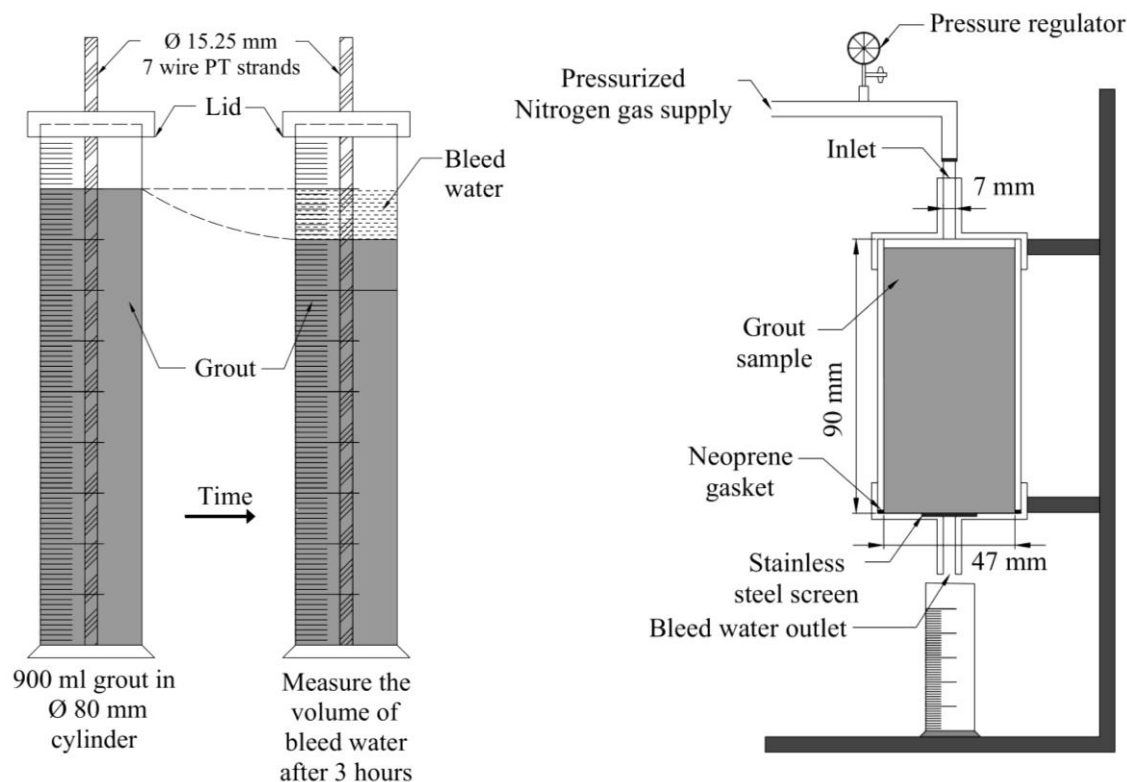
Before using any grout, the required tests shall be conducted in advance to ascertain the mix of the grout. The properties of the mix shall be tested periodically during the construction of a prestressed member, whose specifications are specified hereunder:

- a) *Water-Cement Ratio*: It should be as low as possible, consistent with the required workability. This ratio shall not exceed 0.45.
- b) *Deleterious Materials*: No chloride or sulphate shall be added separately to the grout. When constituent materials of the grout contain chlorides, sulphates or sulphites, their proportion by weight shall not exceed the following limits:

- 1) Chlorides (Cl^-) : 0.1 percent
 - 2) Sulphate (SO_3) : 4.0 percent
 - 3) Sulphide-ions (S_2^-) : 0.01 percent
- c) *Temperature*: The temperature of the grout (after accounting for the ambient temperature of the structure) shall not exceed 25°C .
 - d) *Compressive Strength*: The compressive strength from 100 mm cubes of the grout shall not be less than 17 MPa at 7 days. The cubes shall be cured in a moist atmosphere for the first 24 hours and subsequently in water.
 - e) *Setting Times*: The initial setting time of grout shall be more than three hours and less than 12 hours. And, the final setting time shall not be less than 24 hours.
 - f) *Bleeding*: Bleeding in a grout kept at rest for three hours shall not exceed 0.3 percent of the initial volume of grout, as ascertained from the ____ specified in IS ____.
 - g) *Volume Change*: The volume change of grout kept at rest for 24 hours shall be within the range -0.5 percent and 5.0 percent of the original volume, as ascertained from the ____ specified in IS ____.
 - h) *Fluidity*: Fluidity shall be in the range ____, as ascertained from the Standard Flow Cone Test specified in IS ____.

7.5.4 Tests for Ascertaining Properties of Grout

The test set-ups of two tests employed in the evaluation of properties of the grout shall be taken shown in Fig.10 and 11.



10A

10B

FIG. 10 SCHEMATIC REPRESENTATION OF THE TEST METHODS TO MEASURE THE BLEED WATER: (A) WICK-INDUCED BLEED TEST, AND (B) PRESSURE-INDUCED BLEED TEST

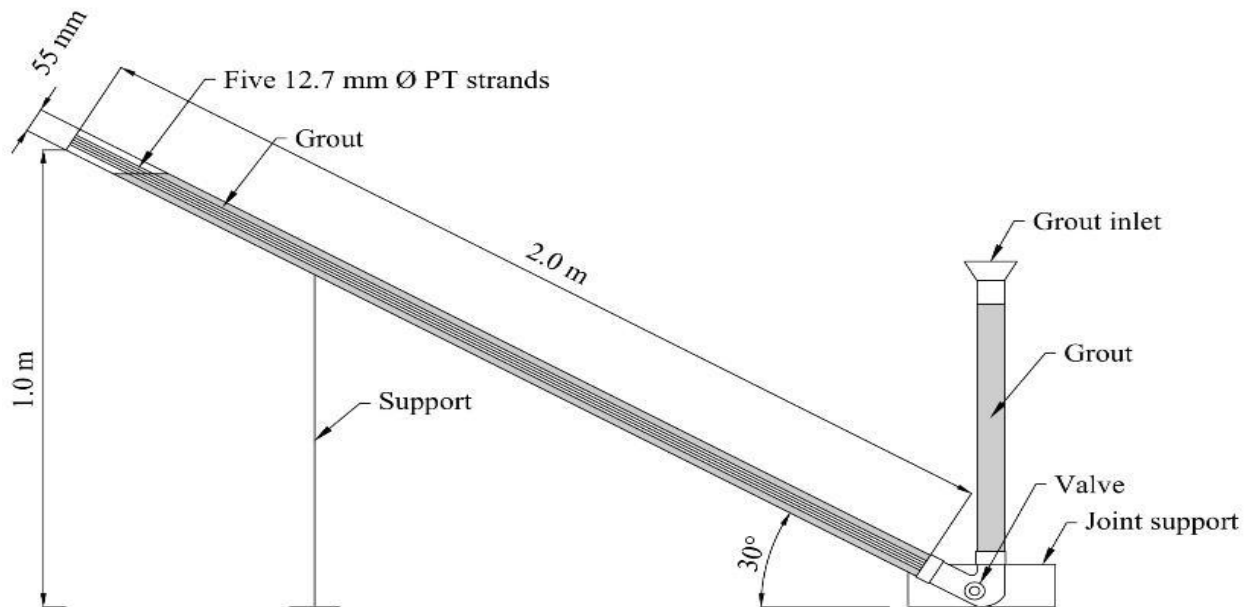


FIG. 11 SCHEMATIC REPRESENTATION OF THE INCLINED TUBE-BLEED TEST SETUP

7.6 FRP Bars

The provisions of this standard are not applicable for members reinforced with FRP bars. FRP bars shall not be used for normal reinforced or prestressed concrete structures. FRP bars can be used only in specific low risk structures where specifically required from Durability considerations and permitted by engineer in charge. In such cases, for structural design, specialist literature need to be consulted, because design provisions of this standard are not applicable to FRP bars in view of difference in physical properties of these bars.

8 STRUCTURAL ANALYSIS

8.1 General

Two methods of structural analysis are admissible.

8.1.1 Linear Analysis

- a) Methods of linear analysis can be used for design of new concrete structures (and their members) and for assessment of existing structures for examining compliance with standardized requirements using initial elastic properties which are suitably modified with partial safety factors specified in this standard.
- b) Linear structural analysis can be performed using
 - 1) 3D structural analysis using a validated standard software.
 - 2) 2D structural analysis which can be done by hand calculations using planar vertical frames in simple structures that satisfy the following:
 - i) The structure is a residential building;
 - ii) The structure is symmetrical in plan and elevation and subjected to symmetrical loads.
 - iii) The structure has a rectangular plan with plan area 400 m² and less;
 - iv) Plan aspect ratio of the structure is 3 or less;
 - v) The structure has height of 10 m or less;
 - vi) The structure has regular structural grid along both plan directions and these grids are parallel to each other along each direction;
 - vii) The sizes of similar members do not vary by more than 10 percent that is, total depths of all beams are within 10 percent of the smallest one, and the plan dimensions of vertical members along each plan direction are within 10 percent of the smallest one; and
 - viii) The effects of minimum plan eccentricity between center of mass and center of resistance at each mass level of the structure are accounted for separately and explicitly as specified in relevant Indian Standards.
- c) It shall be performed by any of the following methods:
 - 1) Conventional Method as per 8.2,
 - 2) Strut and Tie Method as per 8.3, and
 - 3) Simplified Estimates as per 8.4.

8.1.2 Nonlinear Analysis

8.1.2.1 Nonlinear analysis shall be used to assess new concrete structures (and their members) under the action of loads corresponding to Set 2 Accidental Load Combinations and Set 3 Extreme Load Combinations as specified in 6.4 and Table 2), and existing concrete structures (and their members) under the action of loads corresponding to all three sets of load combinations (specified in 6.4 and Table 2).

8.1.2.2 The actual inelastic properties with no partial safety factors applied to the characteristic strengths and strains are used.

8.1.2.3 Standard structural analysis software (publicly validated), only after undertaking a detailed sensitivity analysis of the parameters used to define the inelastic stress-strain or load-deformation characteristics of the materials and/or members shall be used.

8.2 Conventional Method

Structures shall be analyzed by the linear elastic (first-order) analysis method to estimate internal forces and deformations produced by design loads applied on them using spatial (3D) analytical models with material and cross-sectional properties as specified in this standard, under the action of loads combined as per Table 3.

8.2.1 Modelling

3D models shall be employed in the structural analysis with geometry, material and boundary conditions of the structure captured accurately to reflect the true conditions of the structure, except as permitted otherwise in **8.2.1.1(a)**. Member-end rotations may be released during analysis only when the same is reflected during design of members.

8.2.1.1 Member flexibility

The geometry, material properties and boundary conditions of connectivity between members or members and foundation, shall be taken as specified hereunder in order to capture the flexibility of the structure.

a) Geometry (3D Model)

Members of monolithic concrete structures shall be modelled with their centrelines, and assumed to be rigidly connected to each other. The members shall be considered to span between the centrelines of the end members, if

$$\left(\frac{D_1 + D_2}{2} \right) < \left(\frac{L_{centerline}}{10} \right),$$

where D_1 and D_2 are dimensions of the end members into which the member is framing at its ends, and $L_{centerline}$ is the span of the member measured between the centrelines of the end members. Else, the member shall be considered to be flexible up to $D_1/6$ and $D_2/6$ from the faces of the two end members, and the rest of the portions of the member within the joint region treated as rigid. This shall be considered along all three directions of the structure. At least the natural discretisation shall be adopted to model the structure. If the member geometry is non-prismatic or if concentrated loads are applied within the span of the member,

additional nodes required shall be considered, including at the points of application of concentrated loads and moments.

In indeterminate structures subjected to load combinations corresponding to strength, serviceability, robustness and integrity criteria specified in Tables 3 to 6, respectively, the relative stiffness of members shall be based on one of the following:

- 1) The second moment of area of their cracked transformed cross-sections, where the transformed cross-section shall be considered as area of uncracked part of concrete and area of steel reinforcement bars transformed into equivalent concrete area using the modular ratio m given by:

$$m = \frac{300}{f_{ck}},$$

- 2) The prescriptive values of cracked moments of inertia of members as specified in Table 25, where the estimation of the gross moment of inertia I_g of a beam shall consider effects of:
 - i) both beam and a part of the slab (equal to the effective flange width as specified in 7.7.1.4), when slabs are not modeled explicitly; and
 - ii) only beam, when slabs are modeled explicitly and when load combinations considered include earthquake load.

Table 25 Effective Section Properties of members to be used in Structural Analysis

[Clause 8.2.1.1(a)(2)]

Sl No.	Type of structural member	Normalized Effective Cross-sectional Properties					
		For strength design, robustness check and integrity check			For serviceability check		
		Axial Area A_{ae} / A_{ag}	Shear Area A_{se} / A_{sg}	Second Moment of Area I_e / I_g	Axial Area A_{ae} / A_{ag}	Shear Area A_{se} / A_{sg}	Second Moment of Area I_e / I_g
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Slabs	1.0	0.40	0.25	1.0	0.7	0.35
ii)	Beams	1.0	0.40	0.35	1.0	0.7	0.50
iii)	Columns						
	(a) Compression	1.0	0.40	0.70	1.0	0.7	0.90
	(b) Tension	0	0.40	0.70	0	0.7	0.90
iv)	Structural						

	Walls						
	(a) Compression	1.0	0.40	0.70	1.0	0.7	0.90
	(b) Tension	0	0.40	0.70	0	0.7	0.90

When safety of a concrete structure depends on the torsional strength of certain members that is, the members are required to resist primary torsion, the torsional stiffness of such members shall be considered in both analysis and design. Torsional stiffness of such members shall be included in structural analysis; torsional rigidity GC of such members shall be taken as:

$$GC = 0.4G_c C_{eff},$$

where

$$G_c = \text{shear modulus of concrete} = \frac{E_c}{2(1+\nu_c)}, \text{ and}$$

$$C_{eff} = 0.4C,$$

in which

E_c = Modulus of Elasticity of concrete, as specified in 7.1.4.1, and

ν_c = Poisson's ratio of concrete, as specified in 7.1.4.2,

C = Torsional Constant = $0.5K$,

wherein

K = St. Venant Torsional Constant of the gross plain concrete section, of the rectangular web without considering the flange, if any

$$= \alpha D b^3,$$

b = Smaller dimension of the member,

D = Larger dimension of the member, and

α = Factor α to estimate the Saint Venant Torsional Constant K of concrete members given below:

$\frac{D}{b}$	1.0	1.2	1.5	2.0	2.5	3.0	4.0	5.0
α	0.14	0.17	0.20	0.23	0.25	0.26	0.28	0.29

b) Material Property

In the estimation of deflections in structures, the *Modulii* of Elasticity E_c , E_s and E_p of concrete, reinforcing steel and prestressing steel shall be taken as specified in **7.1.4.1, 7.2.2.1 and 7.3.1.2.1**, respectively.

c) *Boundary Conditions*

When modelling, the connection between members shall be considered to be rigid while those between column and column bases shall be considered to be:

- 1) flexible, if the column terminates in to an individual footings resting of flexible soil (where the flexibility shall be reflected by elastic springs, whose translational and rotational stiffnesses shall be estimated considering the flexibility of the footing and the modulus of subgrade reaction of the soil underneath), and
- 2) rigid, if the column terminates into a raft.

8.2.1.2 *Soil-Foundation system flexibility*

The flexibility of foundation of the structure and soil underneath or adjoining shall be accounted for, especially in large structures resting on:

- a) Open foundations (other than piles) resting on soft soil sites (that is, of Site Class D or E), and
- b) Deep foundations (other than wells and caissons) resting on medium and hard soil sites (that is, of Site Class A, B or C),

project site shall be classified as site classes A, B, C, D or E as per IS 1893 (Part 1).

Even if the soil is soft, the simple structures identified in **8.1.1(b)(2)** are exempt from the above requirement.

a) *Foundation Flexibility*

The flexibility of the structural elements composing the foundation of the structure shall be modeled considering their geometry and material properties.

b) *Soil flexibility*

When soils are soft, ground improvement methods shall be employed. The flexibility offered by the soil underneath on which the foundation of the structure rests and the soil adjoining the structure shall be modeled in one of the following ways, namely as:

1) Elastic springs, when only flexibility of soil mass needs to be considered

i) When foundation element is rigid:

Lumped translational springs shall be employed along both vertical and horizontal directions, to account for both normal and shear resistances offered by the soil. Further, rotational springs shall be provided to reflect the rotational restraint offered under the foundations.

ii) When foundation element is flexible:

Distributed translational springs shall be employed along both vertical and horizontal directions, to account for both normal and shear resistances offered by the soil. Modulus of Subgrade Reaction of soil layers shall be considered as specified in IS 1893 (Part 1) while arriving at the stiffness of the said translational and rotational springs.

2) Elastic soil mass, when flexibility and inertia of soil mass needs to be considered

Soil underneath foundations shall be discretized into 3-D soil blocks. The material properties by the Shear Modulus G and Poisson's Ratio ν shall be taken as specified in IS 1893 (Part 1). Caution shall be exercised when modelling the dynamic behaviour of the structure-foundation-soil system, to not introduce spurious reflections arising out of the reflection of waves at the solid boundary generated at the base rock on which the soil rests; or the following shall be considered as boundary condition at supports of structures:

- i) Settlement of foundations, if any, owing to long-term consolidation of soil shall be modeled as specified displacement at the boundary; and
- ii) Loss of contact at foundations, if any, shall be modeled using localized soil springs distributed along the full contact area to reflect the flexibility of soil and of the foundation, to capture loss of contact in soil spring, when they are in tension. Further, linear analysis shall be used only if factor of strength against overturning is more than 1.5, eccentricity of

loading is less than $e = M_D/P_D < L/3$, and area of contact with soil below is more than 50 percent of total area of footing,

else nonlinear analysis shall be performed.

8.2.1.3 Effect of Prestress

When a concrete structure is prestressed (partial or full), structural analysis of the whole structure shall be performed considering:

- Force effects of prestressing in determinate structures, and
- Both force and displacement effects of prestressing in indeterminate structures,

to account for impact of prestressing on adjoining elements.

In indeterminate structures, a Finite Element Model (FEM) shall be prepared of the member and the structure, where forces are transferred between the tendon elements (reflecting the prestressing tendons) and fiber elements (reflecting the concrete adjoining the tendons) along the full length of the member through the bond elements (reflecting the grout).

When the members are expected to respond nonlinearly, the input $\sigma - \varepsilon$ curves of the constitutive materials of the said finite elements shall be nonlinear; the analysis shall be nonlinear necessarily.

8.2.2 Loading

Loads shall be applied as per the load combinations specified in 6.4 as per:

- Table 3 when designing to meet strength criteria,
- Table 4 when checking compliance with the serviceability criteria,
- Table 5 when checking compliance with the robustness criteria,
- Table 6 when checking compliance with the integrity criteria, and
- Table 3, Table 4, Table 5 and Table 6 (as applicable), when checking compliance with the restorability criteria.

8.2.3 Analysis

The stability index Q_s at storey s shall be estimated as:

$$Q_s = \frac{\sum_{c=1}^{N_{cs}} (P_{Dc} \Delta_{Dc})}{\left(\sum_{c=1}^{N_{cs}} H_{Dc} \right) h_s},$$

where

P_{Dc} = design axial force in column c in the storey considered,

Δ_{Dc} = lateral deflection at top of column c with respect to its bottom, in the storey considered along the principal plan direction considered,

H_{Dc} = lateral shear force at top of column c in the storey considered along the principal plan direction considered,

h_s = height of storey s considered, and

N_{cs} = number of columns in the storey s considered.

In the above, P_{Dc} , Δ_{Dc} and H_{Dc} are based on results from linear elastic analysis under the action of loads considered as per the combinations of loads given in 6.4 for strength criteria (Table 3).

The largest value of stability index $Q_{\max}$ amongst those of all N_s storeys of the structure shall be taken as:

$$Q_{\max} = \text{Max}[Q_s, \forall s \in [1, N_s]].$$

The method of analysis of structures applicable shall be as per Table 26, depending on the value of $Q_{\max}$ for the purpose stated therein.

Table 26 Method of structural analysis applicable to the three sets of load combinations

(Clause 8.2.3)

Sl No.	Load Combination	Method of Analysis		
		<i>Linear Analysis</i> if $Q_{\max} < 0.04$	<i>Approximate Geometric Nonlinear Analysis</i> , if $0.04 < Q_{\max} < 0.10$	<i>Geometric and Material Nonlinear Analysis</i> , if $0.10 < Q_{\max}$
(1)	(2)	(3)	(4)	(5)
i)	Basic	Yes for designing the structure	Yes for designing the structure	No, but redesign the Structure to reduce Q_i to be less than 0.10
ii)	Accidental	No	No	Yes for assessing

				the structure
iii)	Extreme	Yes for designing the structure	Yes for designing the structure	Yes for assessing the structure

8.2.3.1 Linear Analysis

8.2.3.1.1 Linear analysis may be performed of concrete structural systems, when:

- Stability index $Q_i < 0.04$, wherein Q_i is estimated as per 8.2.3 under the combinations of loads given in 6.4 for strength criteria (Table 3); and
- Slenderness ratios kL/r of all columns satisfy:

$$\frac{kL}{r} \leq \begin{cases} 20 & \text{Columns NOT braced against side - sway} \\ 40 & \text{Columns braced against side - sway} \end{cases},$$

where

k = Effective length factor of the column,

L = Length of the column, and

r = Radius of gyration

$$= \sqrt{\frac{I_g}{A_g}},$$

in which I_g and A_g are the gross second moment of area and gross area of cross-section of the column section, respectively. r for a column of rectangular cross-section may be taken as 0.30 times the dimension of cross-section in the considered direction of deformation, and of circular cross-section as 0.25 times the diameter of circular cross-section.

8.2.3.1.2 Linear analysis may be performed using the four methods, namely:

- Frame Method shall be used when structures consist largely of members that have one dimension much larger than their other two.
- Finite Element Method shall be used when structures consist of members that are stocky and structures have only structural walls and slabs. The appropriateness of the results from this analysis shall be ensured before considering them in design. Stress concentration at points of application of concentrated loads, re-entrant angles, and openings in slabs, shall be addressed in the design stage.

- c) Hybrid Method shall be used when structures consist of mixed structural systems and members of uneven geometry (such as, deep beams and/or beam-column joints with similar sizes). Some members may be modelled as frame members by their centerline geometries (such as, beams and columns) and others by 3-dimensional finite elements (such as, beam-column joints, structural walls, and service cores in buildings). All requirements related to modelling of the structure specified in the Frame and Finite Element Methods shall be applicable here.
- d) Strut & Tie Method shall be used when structures consist of complex load paths, non-uniform stress distribution in non-prismatic members and junctions of members of concrete structures. This method is meant to perform local analysis, and shall not be used as a substitute for overall global structural analysis of the structure.

8.2.3.1.3 In each of methods mentioned above, it shall be ensured that:

- a) Geometry (such as areas of cross-sections and moments of inertia) of members is consistent with **8.2.1.1(a)**, and
- b) Modulus of elasticity is considered as specified in **7.1.4.1**, **7.2.2.1** and **7.3.1.2.1**.
- c) Mass is represented accurately, when dynamic actions are to be assessed.
- d) Boundary conditions reflect the true restraint at the supports.
- e) Effects of shear deformations are considered when estimating the stiffness of members whose L/D ratio is less than 10. This requirement shall not be applicable to slabs.

8.2.3.2 *Approximate Geometric Nonlinear Analysis*

Geometric nonlinear analysis shall be performed of laterally flexible concrete structural systems (like frame structures), if any, when $0.04 < Q_i < 0.10$, wherein Q_i is estimated as per **8.2.3** under the combinations of loads given in **6.4** for strength criteria (Table 3). The approximate geometric nonlinear analysis may be performed to estimate the additional bending moments (due to $P-\Delta$ effects) on columns as per **8.3.1.7**, and used in the design of columns.

8.2.3.3 *Nonlinear Analysis*

Material and geometric nonlinear analysis shall be performed of laterally flexible concrete structural systems, if any, when $Q_i > 0.10$, wherein Q_i is estimated as per **8.2.3**. Nonlinearities accruing in the structure shall be considered under the action of the loads specified in the load combinations mentioned in **6.4**, when checking for the compliance of the structure with robustness, integrity and restorability criteria.

- a) *Progressive Collapse Analysis*

- 1) When designing important and critical structures for the integrity criteria, progressive collapse analysis shall be performed to estimate the design strength and deformation demands on members, especially the key members of the primary (vertical and lateral) load resisting system, to prevent:
 - i) Local collapse involving formation of plastic hinges of a few adjoining members resulting in that local portion of the structure to become unstable under the inelastic condition and unrestricted deformation under the applied loads,
 - ii) Global collapse involving formation of plastic hinges of all members of the structure (in general) resulting in the whole structure to become unstable under the inelastic condition and unrestricted overall deformation under the applied loads.
- 2) Progressive Collapse Analysis shall involve:
 - i) Identifying all key elements of the primary load resisting system of the structure,
 - ii) Preparing the analytical model removing one key element at a time,
 - iii) Performing linear structural analysis of the residual structure to estimate the design strength demand on the remaining members under the applied gravity loads (namely dead load and live load, and, if applicable, snow load) specified in Set 1 Basic Load Combination given in Table 2.
 - iv) Repeating the analysis removing another key element, until all key elements are removed one at a time.
 - v) Listing all combinations of stress-resultants (P , V , M and T) on each member of the structure from each of the above analyses.
 - vi) These combinations of P , V , M and T shall be used to design the members.

b) *Blast Analysis*

Structural analysis of a structure subjected to blast loading shall be nonlinear response history analysis method; the time increment shall be in milli-seconds. It shall account for material and geometric nonlinearities. Also, the stiffness and strength characteristics of the material as per the nonlinear stress-strain curve of the constituent materials.

Two analyses shall be performed, namely:

- 1) Analysis 1 Nonlinear analysis of the full structure, and

- 2) Analysis 2 Nonlinear analysis of the modified structure, with a few critical members removed, which are closest to the blast. The critical members to be removed shall depend on the outcome of the Analysis 1. Also, the blast pressure shall be applied on the remaining surfaces after removing the critical members. This analysis shall be performed as per progressive collapse.

8.3 Strut and Tie Method

Strut & tie analysis shall be performed as specified hereunder.

8.3.1 Modelling

8.3.1.1 A member or local region shall be represented by a strut and tie model, which consists of a traditional truss made up of struts and ties connected at nodes, capable of transferring the factored loads applied on them. The truss can represent:

- a) B-region (beam or bernoulli region) of the member, or
- b) D-region (disturbed or discontinuity region) of the structure including the junctions of members, the junction of members and supports, or parts of the members where the loads are applied along with other part of the adjoining B-regions.

8.3.1.2 Further,

- a) At each node, at least three members meet, that is, at least three struts, two struts and one tie, one strut and two ties, or three ties.
- b) The ties shall be permitted to cross struts, without touching each other. But, the struts shall cross or overlap only at the nodes.
- c) When determining the geometry of the truss, the dimensions of the struts, ties, and nodal zones shall be taken into account.
- d) The angle θ between the axes of any strut and any tie meeting at a node shall not be less than 25° . For this purpose:
 - 1) The compressive force may be assumed to spread in the strut at a slope of 2 longitudinal to 1 transverse to the axis of the strut, and
 - 2) The tensile force shall be considered to pass straight parallel to the axis of the tie.
- e) The area of each face of a node shall not be less than the cross-sectional area of the strut or tie measured perpendicular to it, and the shape of each face of the node shall be similar to the shape of the projection of the end of the struts onto the corresponding faces of the nodes.
- f) *Geometric Properties*
The geometric properties of struts and ties shall be taken as hereunder:

- 1) The cross-sectional area A_s of a strut shall be estimated as a product of its width b_s and depth D_s of confined concrete within the transverse reinforcement shall be taken as:
 b_s = Breadth of the member under compression, and
 D_s = 0.25 times length of the member under compression.
- 2) The cross-sectional area A_T of the ties made of steel reinforcing bars shall be taken as the sum of the cross-sectional areas of all bars in the member transformed to equivalent area of concrete as specified in **8.2.1.1(a)**.

g) *Material Properties*

The material properties (such as Modulus of Elasticity, Poisson's Ratio, compressive strength, and tensile strength, as applicable) shall be taken as hereunder:

- 1) For struts, the properties shall be taken as that of the concrete is confined by appropriate closely spaced closed-loop transverse steel reinforcement; and
- 2) For ties, it shall be taken as that of the longitudinal steel reinforcement bars.

8.3.2 Loading

The loads and load combinations shall be considered as given in **6.4** and specified in Tables 3, 5 and 6 for strength design, robustness check and integrity check, respectively.

8.3.3 Analysis

The traditional linear truss analysis shall be performed under the action of the applied loads, and the forces estimated in the struts and ties along with the support reactions.

8.4 Simplified Estimates

Unless more accurate estimates are available (from computer-based analyses) as per **8.2.1.1(b)**, provided all conditions mentioned herein are satisfied, the following simplified estimates shall be used of:

- a) Bending moments and shear forces of continuous concrete beams and bending moments of concrete slabs (namely 2-way solid slabs and flat slabs spanning in two directions) as specified in **8.7.1.2** and **8.7.3**. These simplified estimates shall stand as the lower bound values for the results from the conventional method and strut and tie method of structural analysis;
- b) Effective lengths of concrete compression members as specified in **8.8.2**; and
- c) Deflections and crack widths in concrete beams as specified in **8.10.2**.

In continuous beams where vertical supports are not cast monolithically with the horizontal members and individual, small sized, concrete structures that have simple and symmetric structural system, instead of performing the detailed structural analysis by the conventional method, the bending moment and shear force coefficients specified hereunder shall be deemed to be sufficient. When these coefficients are used, redistribution of bending moments shall not be permitted.

8.5 Moment Frame Structures

When a concrete structure is made of a moment frame, the simplified estimates provided in 8.6 and 8.7 for beams and slabs, respectively, shall be used.

8.6 Beams

Beams are member with axial stress f_{ax} less than $0.1f_{ck}$ under design loads arising out of load combination specified in 6.4 and Tables 3, 5 and 6.

a) Load transfer from rectangular slab panels to perimeter supports

The uniformly distributed load on the solid slab spanning in two directions at right angles shall be assumed to be transferred to perimeter supports as per the influence area generated by the intersection of the 45° lines as shown in Fig. 12.

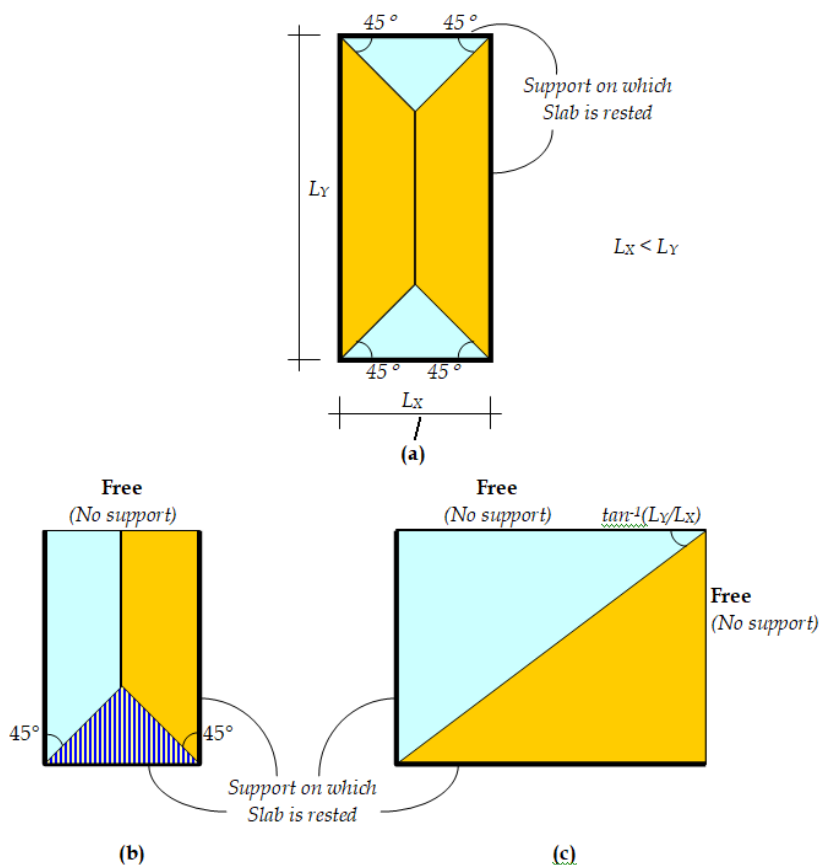


FIG. 12 UNIFORMLY DISTRIBUTED LOAD ON SHADED AREA OF SOLID SLAB IS ASSUMED TO BE TRANSFERRED TO THE ADJOINING SUPPORT: SLAB SUPPORTED ON (A) ALL FOUR SIDES, (B) THREE SIDES, AND (C) TWO ADJOINING SIDES

b) Design moment and shear coefficients

- 1) The bending moment and shear force coefficients in continuous beams at any floor or roof level shall be taken as specified hereunder, provided:
 - i) The structure has a simple and symmetric structural system,
 - ii) The structure is subjected to relatively uniform gravity loads (namely DL and IL) on its spans,
 - iii) The structure has stability index $Q_i < 0.04$ under gravity loads, and
 - iv) The beam is not a deep beam, whose ratio of effective span l_e to overall depth D , is:

$$\frac{l_e}{D} \leq \begin{cases} 2.0 & \text{Simply Supported Beam} \\ 2.5 & \text{Continuous Beam} \end{cases}.$$

When any one of these conditions is not satisfied, detailed conventional method of analysis shall be employed.

Unless more exact estimates are made for beams of uniform cross-section, which support substantially uniformly distributed loads over three or more spans, which do not differ by more than 15 percent of the longest span length, the design bending moments and design shear forces may be estimated using the coefficients given in Tables 27 and 28, respectively. The coefficients given in Table 27 shall be multiplied by W_e to obtain the design bending moments, and those given Table 28 shall be multiplied by W to obtain the design shear forces, where W is the total design load on the span and l_e the effective span, or such other restraining moment as may be shown to be applicable.

Table 27 Design Bending Moment Coefficients at Select Locations

[Clause 8.6(b)]

<i>Type of Load</i>	<i>Span Moments</i>		<i>Support Moments</i>	
	<i>Near middle of End Span</i>	<i>At middle of Interior Span</i>	<i>At support of End Span</i>	<i>At supports of Interior Span</i>
DL and IL (Fixed)	$+\frac{1}{12}$	$+\frac{1}{16}$	$-\frac{1}{10}$	$-\frac{1}{12}$
IL (Not Fixed)	$+\frac{1}{10}$	$+\frac{1}{12}$	$-\frac{1}{9}$	$-\frac{1}{9}$

Table 28 Design Shear Force Coefficients at Select Locations

[Clause 8.6(b)]

<i>Type of Load</i>	<i>At End Support</i>	<i>At Support near the End Support</i>		<i>At Other Interior Supports</i>
		<i>Support closer to End Span</i>	<i>Support farther from End Span</i>	
DL and IL (Fixed)	0.40	0.60	0.55	0.50
IL (Not Fixed)	0.45	0.60	0.60	0.60

- 1) For moments at supports where two unequal spans meet or when the spans are not equally loaded, the average of the two values for the negative moments at the

support may be taken for design. Where coefficients given in Table 27 are used for estimation of bending moments, redistribution shall not be permitted.

- 3) Where a member is built into a masonry wall that offers only a partial restraint, the member shall be designed to resist a negative moment of $M_e/24$ at the face of the support. For such a condition, the coefficient given in Table 28 to estimate the design shear force at the end support may be increased by 0.05.

8.7 Slabs

Rectangular slabs shall be treated as one-way slabs, when they satisfy any of the following conditions:

- a) They cantilever from only one edge;
- b) They are supported on opposite edges; and
- c) They are supported on more than two opposite edges, but have a plan aspect ratio l_y/l_x of more than 2.

All other slabs (including circular and polygon plan shaped slabs supported on all edges, or at a point in the center) shall be treated as two-way slabs.

8.7.1 Solid Slabs

The provisions of 8.6 for beams apply to slabs also. In addition, the provisions provided hereunder are applicable.

8.7.1.1 One-way slabs

Slabs spanning in one direction and continuous over supports shall be treated as a one-way slab, and analyzed according to the provisions applicable to continuous beams.

8.7.1.2 Two-way slabs

When the load on the slab is transferred to supports are perpendicular to each other in plan or when there are only two supports but on opposite edges of the slab, which are not parallel to each other, the slab shall be treated as a two-way slab.

8.7.1.3 Slabs monolithic with supports

Slabs (except flat slabs) constructed monolithically with supports shall be analyzed by taking such slabs either as continuous over the supports and capable of free rotation, or as members of a continuous system with the supports, taking into account the stiffness of such supports. If such supports are formed due to beams that justify fixity at the support of slabs, then the effects on the supporting beam, such as bending of the web in

the transverse direction of the beam and twisting about the longitudinal direction of the beam, wherever applicable, also shall be considered in the analysis of the beam.

For the purpose of estimation of moments in slabs in a monolithic structure, it will be sufficiently accurate to assume that members connected to the ends of such slabs are fixed in position and direction at the far ends from their connections with the slabs.

8.7.1.4 Slabs carrying concentrated load

If a solid slab supported on two opposite edges, carries concentrated loads, the maximum bending moment caused by the concentrated loads shall be assumed to be resisted by an effective width b_e of slab (measured parallel to supporting edge) given by:

- a) For a single concentrated load, the effective width b_e shall be estimated as:

$$b_e = \left[kx \left(1 - \frac{x}{l_e} \right) + a \right] \leq b$$

where

a = width of contact area of concentrated load from nearer support measured parallel to supported edge;

b = actual width of the slab;

k = Constant as per Table 29, as a function of b/l_e ,

x = Distance of centroid of concentrated load from the nearer support, and

l_e = Effective span.

When the concentrated load is placed near the unsupported edge of a slab, b_e shall be estimated as:

$$b_e = \text{Min} \left[\left\{ kx \left(1 - \frac{x}{l_e} \right) + a \right\}; b; 0.5 \left\{ kx \left(1 - \frac{x}{l_e} \right) + a \right\} + y_0 \right],$$

where y_0 is the distance of the concentrated load from the unsupported edge.

Table 29 k for Simply Supported and Continuous Slabs

[Clause 8.7.1.4(a)]

b/l_e	k	
	Simply supported slab	Continuous slab
0.1	0.40	0.40
0.2	0.80	0.80
0.3	1.16	1.16

0.4	1.48	1.44
0.5	1.72	1.68
0.6	1.96	1.84
0.7	2.12	1.96
0.8	2.24	2.08
0.9	2.36	2.16
1.0	2.48	2.24
1.1	2.58	2.29
1.2	2.64	2.36
1.3	2.72	2.40
1.4	2.78	2.45
1.5	2.84	2.49
1.6	2.88	2.53
1.7	2.92	2.56
1.8	2.96	2.58
0.9	2.99	2.60
≥2.0	3.00	2.60

- b) For two or more concentrated loads placed in a line in the direction of the span, the bending moment per meter width of slab shall be calculated separately for each load according to its appropriate b_e of slab estimated as in (a), and added together for design calculations.
- c) For two or more concentrated loads placed not in a line in the direction of the span, if b_e of slab owing one load does not overlap with b_e of slab owing to another load (both estimated as in (a) above), then the slab for each load can be designed separately. If b_e of slab of one load overlaps with that of the adjacent load, the overlapping portion of the slab shall be designed for the combined effect of the two loads.
- d) For cantilever solid slabs, b_e shall be estimated as:

$$b_e = (1.2a_1 + a) \leq \frac{l_{cfe}}{3},$$

where

a_1 = Distance of concentrated load from face of the cantilever support,

a = Width of contact area of concentrated load measured parallel to supporting edge, and

l_{cfe} = Length of cantilever slab measured parallel to fixed edge.

When the concentrated load is placed near the unsupported edge of a cantilever slab, b_e shall be estimated as:

$$b_e = \text{Min} \left[\{(1.2a_1 + a)\}; \frac{l_{cfe}}{3}; 0.5 \left\{ kx \left(1 - \frac{x}{l_e} \right) + a \right\} + y_{cfe} \right],$$

where y_{cfe} is the distance of the concentrated load from the extreme end measured in the direction parallel to the fixed edge.

- e) For slabs other than solid slabs, b_e shall depend on the ratio of transverse and longitudinal flexural rigidities of the slab. Where this ratio is close to 1.0, b_e estimated for solid slabs may be used. And, as this ratio decreases, proportionately smaller value shall be taken.
- f) Any other recognized method of structural analysis may be used for slabs with the approval of the engineer-in-charge, as an alternative to the values specified above.

8.7.1.5 Slabs spanning in two directions at right angles

The bending moments in slabs spanning in two directions at right angles shall be analysed by:

- a) Any acceptable theory or by using coefficients given in ____, when carrying uniformly distributed load, and
- b) Any acceptable method approved by engineer-in-charge, when carrying concentrated load.

When $l_y/l_x > 2$, the slabs shall be designed as one way slabs, else as two way slabs.

8.7.1.6 Analysis of two way slabs

- a) *Two Way Slabs restrained against torsion at corners*

When the corners of a slab are prevented from lifting, the coefficients of bending moment in the slab shall be estimated as specified hereunder.

- 1) Maximum bending moments M_x and M_y per unit width and maximum shear force V_x and V_y per unit width along shorter and longer spans, respectively, for the nine cases of continuity (Fig. 13) of the edges of rectangular slabs shall be estimated as:

$$M_x = \alpha_x w l_x^2$$

$$M_y = \alpha_y w l_y^2$$

$$V_x = \alpha_x w L_x$$

$$V_y = \alpha_y w l_y$$

where

α_x, α_y = short and long span bending moment coefficients, respectively, as per Table 30;

β_x, β_y = short and long-span shear force coefficients, respectively, as per Table 31;

w = total uniformly distributed design load per unit area based on load combinations specified in Table 3; and

l_x, l_y = Lengths of shorter and longer spans, respectively [Fig. 12(a)].

- 2) Slabs shall be considered in each direction to be composed of middle strips and edge strips (Fig. 14); the middle strip is three-quarters of the width and each edge strip one-eighth of the width.
- 3) The maximum moments estimated in (1) above shall apply only to middle strips, and no redistribution shall be made.

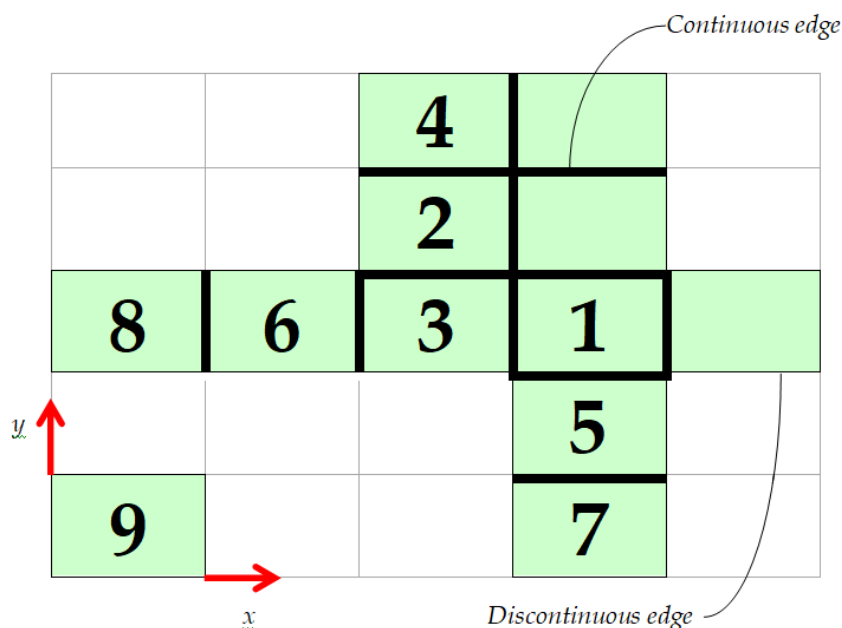


FIG. 13 NINE CASES OF CONTINUITY OF SLABS

Table 30 Bending Moment Coefficients for Rectangular Slab Panels supported on Four Sides with provision for Torsion at corners

[Clause 8.7.1.6(a)]

Case	Moment considered	Short Span Coefficient α_x								α_y
		l_y/l_x								All l_y/l_x
		1.00	1.10	1.20	1.30	1.40	1.50	1.75	2.00	
1.	Interior Panels									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2.	One Short Edge Discontinuous									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3.	One Long Edge Discontinuous									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4.	Two Adjacent Edges Discontinuous									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5.	Two Short Edges Discontinuous									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	0.045
	Positive moment at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6.	Two Long Edges Discontinuous									
	Negative moment at continuous edge	-	-	-	-	-	-	-	-	-
	Positive moment at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7.	Three Edges Discontinuous - One Long Edge Continuous									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	0.057
	Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8.	Three Edges Discontinuous - One Short Edge Continuous									
	Negative moment at continuous edge	-	-	-	-	-	-	-	-	-
	Positive moment at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9.	Four Edges Discontinuous									
	Negative moment at continuous edge	-	-	-	-	-	-	-	-	-
	Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

- 4) When support moments estimated using Table 30 for adjacent panels differ significantly, the following procedure shall be adopted to adjust them:
- Estimate a Total Moment as the sum of mid-span moment and average of support moments (neglecting signs) for each panel.
 - Treat the value from (i) above as fixed end moments.
 - Distribute the fixed end moments across the supports in proportion to relative stiffness of adjacent spans, and arrive at New Support Moments.
 - Estimate New Mid-Span Moment in a span as Total Moment of that span obtained in (i) above minus smaller of New Support Moments obtained in (iii) above of that span (neglecting signs).

If the resulting New Support Moments are significantly greater than those from Table 30, the tension steel over the supports shall be extended further, as given below:

- i) Consider the span moment to be varying parabolically between supports, with the maximum value as New Mid-Span Moment obtained in (iv) above.
- ii) Determine the points of contra-flexure of the New Support Moments [obtained from (iii) above] with the Total Moment [obtained from (i) above].

b) *Two Way Slabs not restrained against Torsion at corners (Simply Supported Slabs)*

When the corners of a slab are prevented from lifting, the slab shall be designed as specified hereunder.

- 1) When simply supported slabs do not have adequate provision to resist torsion at corners and to prevent the corners from lifting, the Maximum bending moments M_x and M_y per unit width in a slab spanning along shorter and longer spans, respectively, shall be estimated as:

$$M_x = \alpha_x w l_x^2$$

$$M_y = \alpha_y w l_x^2$$

where

α_x, α_y = short- and long-span bending moment coefficients, respectively, as

per Table 32;

w = total uniformly distributed design load per unit area; and

l_x, l_y = lengths of shorter and longer spans, respectively.

Table 31 Shear Force Coefficients for Rectangular Slab Panels supported on Four Sides with provision for Torsion at corners

[Clause 8.7.1.6(a)]

Case	Moment considered	Short Span Coefficient β_x								β_y
		l_y / l_x								All
		1.00	1.10	1.20	1.30	1.40	1.50	1.75	2.00	l_y / l_x
1.	Interior Panels									
	Continuous Edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33
2.	One Short Edge Discontinuous									
	Continuous Edge	0.36	0.39	0.42	0.44	0.45	0.47	0.50	0.52	0.36
	Discontinuous Edge	-	-	-	-	-	-	-	-	0.24
3.	One Long Edge Discontinuous									
	Continuous Edge	0.36	0.40	0.44	0.47	0.49	0.51	0.55	0.59	0.36
	Discontinuous Edge	0.24	0.27	0.29	0.31	0.32	0.34	0.36	0.38	-
4.	Two Adjacent Edges Discontinuous									
	Continuous Edge	0.40	0.44	0.47	0.50	0.52	0.54	0.57	0.60	0.40
	Discontinuous Edge	0.26	0.29	0.31	0.33	0.34	0.35	0.38	0.40	0.26
5.	Two Short Edges Discontinuous									

	Continuous Edge	0.40	0.43	0.45	0.47	0.48	0.49	0.52	0.54	-
	Discontinuous Edge	-	-	-	-	-	-	-	-	0.26
6.	Two Long Edges Discontinuous									
	Continuous Edge	-	-	-	-	-	-	-	-	0.40
	Discontinuous Edge	0.26	0.30	0.33	0.36	0.38	0.40	0.44	0.47	-
7.	Three Edges Discontinuous – One Long Edge Continuous									
	Continuous Edge	0.45	0.48	0.51	0.53	0.55	0.57	0.60	0.63	-
	Discontinuous Edge	0.30	0.32	0.34	0.35	0.36	0.37	0.39	0.41	0.29
8.	Three Edges Discontinuous – One Short Edge Continuous									
	Continuous Edge	-	-	-	-	-	-	-	-	0.45
	Discontinuous Edge	0.29	0.33	0.36	0.38	0.40	0.42	0.45	0.48	0.30
9.	Four Edges Discontinuous									
	Discontinuous Edge	0.33	0.36	0.39	0.41	0.43	0.45	0.48	0.50	0.33

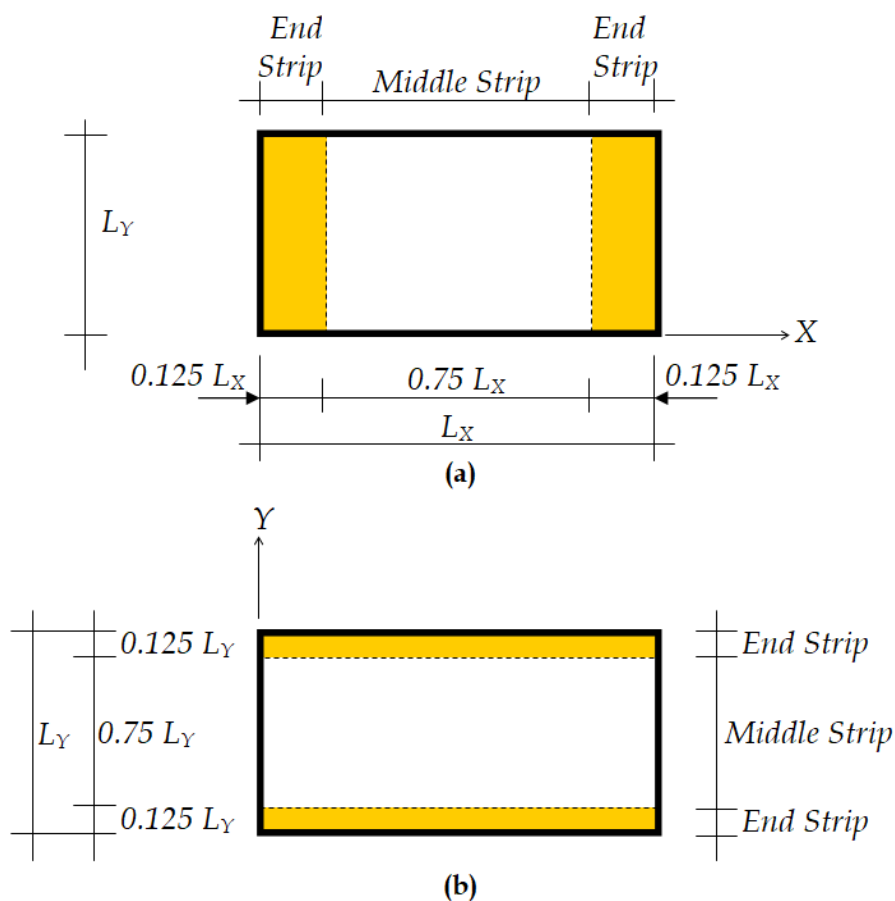


FIG. 14 MIDDLE AND EDGE STRIPS OF A SLAB FOR BENDING ALONG:
(A) X-DIRECTION, AND (B) Y-DIRECTION

**Table 32 Bending Moment Coefficients for Rectangular Slab Panels
Simply-supported on four sides**

[Clause 8.7.1.6(b)]

l_y/l_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

8.7.2 Ribbed, Hollow Or Voided Slabs

The bending moments and shear forces due to design loads on continuous slabs shall be obtained by the rigorous method specified in **9.3.3.1(e)**.

8.7.3 Flat Slabs**8.7.3.1 General**

The coefficients given hereunder are for slabs subjected to gravity loads only. Values provided hereunder cannot be added numerically with those obtained by other methods of analysis. The effects of lateral loads on flat slabs shall be estimated by detailed nonlinear analysis (using hybrid analysis specified in **8.1.3.1**).

The term flat slab means a RC slab without beams or with beams not meeting the requirements of l/d ratio specified in **8.8.1**, but with or without drops, which is supported by columns with or without flared column heads or capitals (Fig. 15). It may be solid slab or may have recesses formed on the soffit, so that the soffit comprises a series of ribs in two directions. The recesses may be formed by removable or permanent filler blocks.

8.7.3.1.1 For the purpose of this clause, the following definitions shall apply:

- Column strip* - a strip of slab having a width of $0.25l_2$, but not greater than $0.25l_1$ on each side of the column centerline, where l_1 is the span in the direction moments are being determined, measured centre to centre of supports and l_2 the span transverse to l_1 measured centre to centre of supports.
- Middle strip* - a strip of slab bounded on each of its opposite sides by the Column Strip.
- Panel strip* - a part of a slab bounded on each of its four sides by the centerline of a column, or centerlines of adjacent spans.

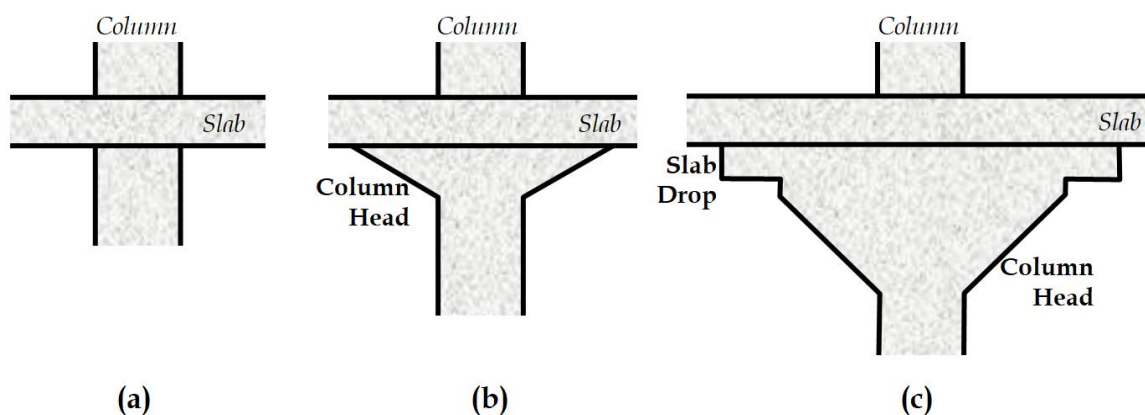


FIG. 15 DIFFERENT GEOMETRIES OF FLAT SLABS (A) SLAB WITHOUT DROP AND COLUMN WITHOUT COLUMN HEAD, (B) SLAB WITHOUT DROP AND COLUMN WITH COLUMN HEAD, AND (C) SLAB WITH DROP AND COLUMN WITH COLUMN HEAD

8.7.3.2 Geometric proportioning

8.7.3.2.1 Thickness of flat slab

The minimum thickness of slab shall be 125 mm. The thickness of a flat slab shall be controlled by considerations of span to effective depth ratios given by 8.2.3.1.

For slabs with drops conforming to 8.7.3.2.2, the ratios of longer span to effective depth given in 8.2.3.1 shall be applied directly, else the ratios shall be multiplied by 0.9. For this purpose, only the longer span shall be considered.

8.7.3.2.2 Drop

When drops are provided, they shall be rectangular in plan, and have a length in each direction not less than one-third of the panel length in that direction. For exterior panels, the width of drops at right angles to the non-continuous edge and measured from the centerline of the columns shall be equal to one-half the width of drop for interior panels.

8.7.3.2.3 Column heads

When column heads or capitals are provided, the portion of a column head that lies within the largest right circular cone or pyramid that has a vertex angle of 90° and can be included entirely within the outlines of the column and the column head, shall be considered for design purposes (see Fig. 15).

8.7.3.3 *Determination of bending moment***8.7.3.3.1** *Methods of analysis*

Flat slab systems shall be analysed by the computer-based finite element method. In the absence of results from such detailed analysis, it shall be permissible to analyse such systems by one of the following methods:

- a) The direct design method as specified in (8.7.3.4) and
- b) The equivalent frame method as specified in (8.7.3.5)

subject to the limitations of the methods mentioned therein.

8.7.3.3.2 *Bending moments in panels with marginal beams*

When the slab is supported by a marginal beam of depth more than 1.5 times the thickness of the slab or by a wall, then:

- a) The total load to be carried by the beam or wall shall comprise those loads applied directly on the wall or beam plus a uniformly distributed load equal to one-quarter of the total load on the slab, and
- b) The bending moments on the half-column strip adjacent to the beam or wall shall be one-quarter of the bending moments for the first interior column strip.

8.7.3.3.3 *Transfer of bending moments to columns*

$$\alpha = \frac{1}{1 + \frac{2}{3} \sqrt{\frac{a_1}{a_2}}},$$

where

a_1 = Overall dimension of the critical section for shear in the direction perpendicular to the axis about which moment acts, and

a_2 = Overall dimension of the critical section for shear in the direction of the axis about which moment acts (see Fig. 16).

For resisting the bending moment arising in the slab, a slab width between lines that are $1.5D$ on each side of the column or capital shall be considered effective, where D is the larger of: (i) thickness of slab, (ii) thickness of drop panel, or (iii) width of column resisting the bending moment. Concentration of reinforcement over column head by closer spacing or additional reinforcement may be used to resist the bending moment at this section.

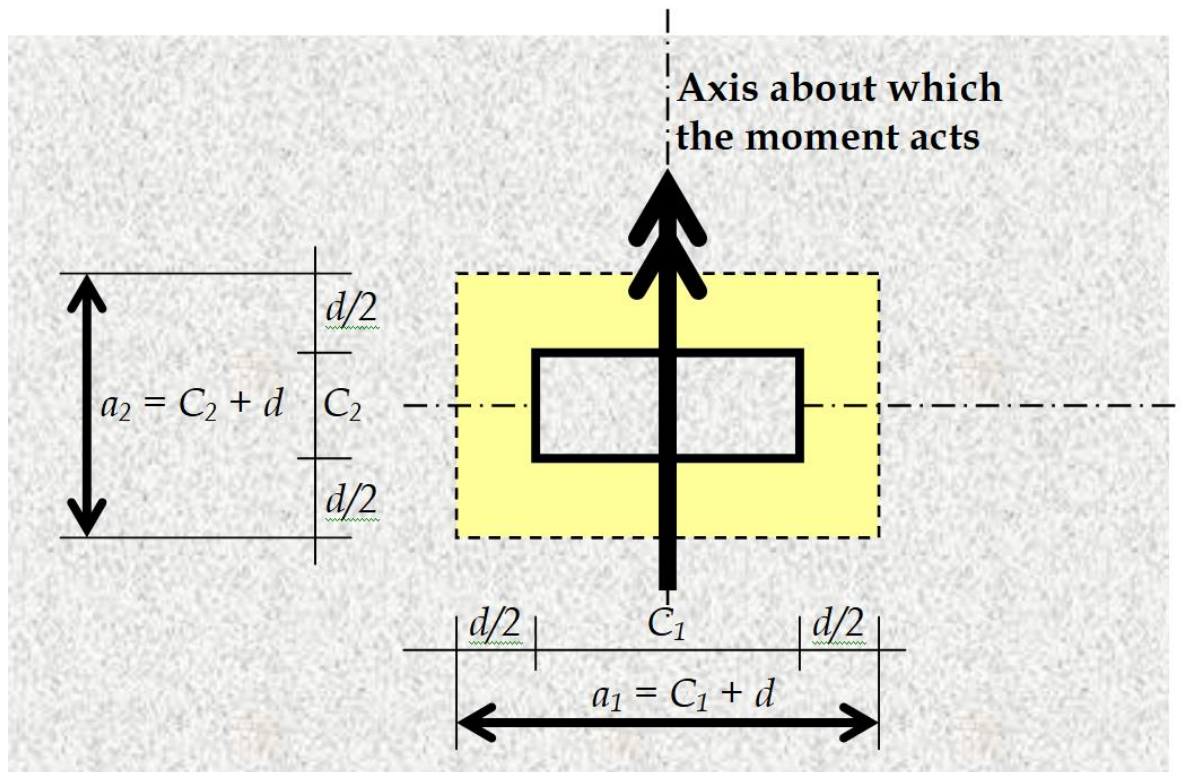


FIG. 16 GEOMETRY AT TRANSFER OF UNBALANCED BENDING MOMENT FROM FLAT SLAB TO COLUMN; THIS IS APPLICABLE FOR TRANSFER OF UNBALANCED BENDING MOMENT FROM FOOTING TO COLUMN ALSO

8.7.3.4 Direct design method

8.7.3.4.1 The direct design method shall be used to analyse and design flat slab structural system only when the flat slab system fulfils the following conditions:

- a) The structure is subjected to gravity loads that are uniformly distributed over the panels. When the system is required also to resist lateral loads arising from wind, earthquake ground shaking, blast, impact or other lateral load actions, additional

RC structural walls or other bracing elements shall be provided in the structure. And, such elements providing basic lateral resistance to the structure shall be designed to resist the entire design lateral load given by 6.4.

- b) There shall be at least three continuous spans in each direction.
- c) The lengths of successive spans in each direction do not differ by more than one-third of longer span length; end spans may be shorter but not longer than interior spans.
- d) The panels shall be rectangular, and the ratio of longer span length to shorter span length within a panel shall not be greater than 2.0.
- e) Columns are not offset from the grid lines by more than 10 percent of the span length in the direction of the offset, notwithstanding the provision in (b) above; and
- f) The design live load shall not exceed three times the design dead load.

8.7.3.4.2 The following steps shall be followed:

- a) Determine total design moment (M_0) of a span (that is, the isostatic moment of a simply supported beam) along each direction. The absolute sum of the positive and average negative bending moments in each direction, determined for a strip bounded laterally by the centerline of the panel on each side of the centerline of the supports, shall be taken as:

$$M_0 = \frac{Wl_n}{8},$$

where

W = design load on area l_2l_n ;

l_n = clear span extending from face to face of columns, capitals, brackets or walls, but not less than $0.65l_1$;

l_1 = length of span in the direction of M_0 ; and

l_2 = length of span transverse to l_1 .

While determining M_0 , the following need to be considered:

- 1) Circular supports shall be treated as square supports of the same area.
- 2) When the transverse span of the panels on either side of the centerline of supports varies, l_2 shall be taken as the average of the transverse spans.
- 3) When an exterior panel is being considered, the distance from the exterior edge to the centerline of panel shall be substituted for l_2 in calculation of M_0 .

- b) Determine negative and positive design moments of a span, as fractions of M_0 , along each direction.

- 1) In an interior span, the design negative moment M_0^- (considered to act at the face of rectangular supports, or circular supports treated as square supports having the same area) and design positive moment M_0^+ (considered to act at the mid-span) shall be distributed in the following proportion:

$$M_0^- = 0.65M_0, \text{ and}$$

$$M_0^+ = 0.35M_0$$

And, in an exterior span, the design negative moments M_0^{-EXT} M_0^{-EXT} (considered to act at the face of rectangular supports, or circular supports treated as square supports having the same area) at the exterior and interior supports, and design positive moments M_0^+ (considered to act at the mid-span) shall be distributed in the following proportion:

$$M_0^{-EXT} = \frac{0.65}{1 + \frac{1}{\alpha_c}},$$

$$M_0^+ = 0.63 - \frac{0.28}{1 + \frac{1}{\alpha_c}}, \text{ and}$$

$$M_0^{-EXT} = 0.75 - \frac{0.10}{1 + \frac{1}{\alpha_c}},$$

where α_c , the ratio of flexural stiffness of exterior columns to that of the slab at a joint taken in the direction moments are being determined, is given by:

$$\alpha_c = \frac{\sum_{i=1}^{N_{cj}} K_{ci}}{K_s}$$

in which K_{ci} is the flexural stiffness of column i meeting at the joint and K_s the flexural stiffness of the slab, expressed as moment per unit rotation.

- 2) These design moments can be modified by up to 10 percent, so long as sum of the modified values of M_0^{-EXT} , M_0^{-EXT} and M_0^+ for the panel in the direction considered is not less than the corresponding total design moment M_0 specified above.

- 3) When the negative moments at an interior support are different from the two spans framing into it, the section shall be designed to resist the larger of the two interior negative design moments, unless an analysis is made to distribute the unbalanced moment in accordance with the stiffness of the adjoining parts.
- c) Apportion negative and positive design bending moments to column strips and middle strips across the panel width, along each direction.

The bending moments at the critical (support or mid-span) cross-section shall be distributed to the column and middle strips as specified in **8.3.1.2(c)(1)**.

- d) Determine moments in columns
 - 1) Columns built integrally with the flat slab system shall be designed to resist moments arising from loads on the slab system.
 - 2) Unless a general analysis is made, at an interior support, the column members above and below the flat slab together shall be designed to resist total design moment M_c for the column given by:

$$M_c = 0.08 \frac{(w_{DL} + 0.5w_{IL})l_2l_n^2 - w'_{DL}l_2'^2l_n}{1 + \frac{1}{\alpha_c}}$$

where

w_{DL} , w_{IL} = design dead and imposed loads per unit area, respectively;

l_2 = length of span transverse to the direction of M_c ;

l_n = length of clear span in the direction of M_c measured between the face of supports;

α_c is as specified in **7.7.3.4.2(b)(1)**; and w'_{DL} , l_2' and l_n' refer to the shorter span.

M_c shall be distributed to the column sections above and below the flat slab in proportion to their stiffness.

- e) *Effects of Pattern Loading*

When the ratio of imposed load to dead load exceeds 0.5:

- 1) The sum of flexural stiffness of columns above and below the flat slab, $\sum_{i=1}^{N_{cj}} K_{ci}$ shall be such that α_c is not less than the minimum value $\alpha_{c,min}$ specified in Table 33; or

- 2) If $\sum_{i=1}^{N_{cj}} K_{ci}$ does not satisfy (1) above, the positive design moments of the panel shall be multiplied by the coefficient β_s given by:

$$\beta_s = 1 + \left[\frac{2 - \frac{w_{DL}}{w_{IL}}}{4 + \frac{w_{DL}}{w_{IL}}} \right] \left(1 - \frac{\alpha_c}{\alpha_{c,min}} \right).$$

Table 33 Minimum Permissible values of $\alpha_{c,min}$

[Clause 8.7.3.4.2(e)]

$\frac{w_{IL}}{w_{DL}}$	$\frac{l_2}{l_1}$	$\alpha_{c,min}$
0.5	0.5 to 2.0	0.0
1.0	0.50	0.6
	0.80	0.7
	1.00	0.7
	1.25	0.8
	2.00	1.2
2.0	0.50	1.3
	0.80	1.5
	1.00	1.6
	1.25	1.9
	2.00	4.9
3.0	0.50	1.8
	0.80	2.0
	1.00	2.3
	1.25	2.8
	2.00	3.0

8.7.3.5 Equivalent frame method

The method shall be used when the flat slab system does not meet the requirements laid down for the use of the direct design method or when the structure having the flat slab is required to resist lateral loads.

8.7.3.5.1 Assumptions

The bending moments and shear forces in a flat slab system may be determined by this method, which considers the building with flat slab system as a continuous 3-dimensional frame model and makes the following assumptions:

- a) The building is made up of equivalent frames with vertical and horizontal members at all levels. Each frame consists of a row of equivalent columns, bounded laterally by the centerline of the panel on each side of the centerline of the columns. Frames adjoining an exterior panel are bounded by the edge and the centerline of the adjoining panel.
- b) Each such planar frame is analyzed in its entirety, for vertical loading alone. Each floor thereof and the roof, is analyzed separately with its columns assumed fixed at their remote ends. When slabs are analyzed separately to determine the bending moment at a given column, the slab is fixed at a support two panels distant there from, provided the slab continuous beyond the point.
- c) When determining relative stiffness of members, the moment of inertia of a slab or column may be assumed to be that of the gross cross-section of the concrete alone.
- d) Variations of moment of inertia along the axis of the slab owing to provision of drops is considered. In a recessed or coffered slab, which is made solid in the region of the columns, the stiffening effect may be ignored, provided the solid part of the slab does not extend more than $0.15 l_e$, into the span measured from the centerline of the columns. The stiffening effect of flared column heads is ignored.

8.7.3.5.2 Loading pattern

8.7.3.5.2.1 When the loading pattern is known, the structure shall be analyzed for the load concerned.

8.7.3.5.2.2 When imposed load/dead load ratio is variable, but does not exceed 0.75, or the nature of the imposed load is such that all panels are loaded simultaneously, the maximum moments shall be assumed to occur at all sections when full design imposed load is on the entire slab system.

8.7.3.5.2.3 For other conditions of imposed load/dead load ratio and when all panels are not loaded simultaneously,

- a) Maximum positive moment near mid-span of a panel shall be assumed to occur when three-quarters of full design imposed load is on the panel and on alternate panels thereafter; and
- b) Maximum negative moment in the slab at a support may be assumed to occur when three-quarters of full design imposed load is on the adjoining panels only.

8.7.3.5.2.4 In no case shall the design moment be taken to be less than those occurring with full design imposed load on all panels.

8.7.3.5.3 Negative design moment

8.7.3.5.3.1 At an interior support, the critical section for negative moment, in both column strip and middle strip, shall be taken at the face of rectilinear supports, but in no case at a distance greater than $0.175 l_1$ from the centre of the column, where l_1 is the length of the span in the direction moments are being determined, measured center-to-center of supports.

8.7.3.5.3.2 At an exterior support provided with bracket or capital, the critical section for negative moment in the direction perpendicular to the edge shall be taken at a distance from the face of the bracket or capital not greater than one-half the projection of the bracket or capital beyond the face of the bracket or capital.

8.7.3.5.3.3 Circular or regular polygon shaped bracket or capital shall be treated as square bracket or capital having the same area.

8.7.3.5.4 Modification of Maximum Moment

Moments determined above may be reduced in such proportion that the numerical sum of the positive and average negative moments is not less than the value of total design moment M_o specified in **8.7.3.5**.

8.7.3.5.5 Distribution of bending moment across the panel width

8.7.3.5.5.1 The column strip shall be designed to resist:

- a) 75 percent of total negative moment in the panel at that support, at an interior support; and
- b) 100 percent of total negative moment in the panel at that support, at an exterior support. Where the exterior support consists of a column or a wall extending for a distance equal to or greater than $0.75l_2$, where l_2 is the length of span transverse to the direction moments considered, the exterior negative moment shall be considered to be uniformly distributed across the length l_2 .
- c) 60 percent of total positive moment in the panel.

8.7.3.5.5.2 The middle strip shall be designed to resist the following:

- a) That portion of the total design moment M_o not resisted by the column strip.
- b) The sum of the moments assigned to its two half middle strips.

- c) The middle strip adjacent and parallel to an edge supported by a wall shall be designed to resist twice the moment assigned to half the middle strip corresponding to the first row of interior columns.

8.8 Columns and Struts

8.8.1 Definitions

A column shall have axial stress f_{ax} more than $0.1f_{ck}$ under design loads arising out of any load combination specified in 6.4.

Depending on slenderness ratio L_{eX}/D or L_{eY}/b , columns shall be classified as:

$$Column = \begin{cases} \text{Pedestal} & \text{Both } L_{eX}/D \text{ and } L_{eY}/b \text{ are } \leq 3 \\ \text{Short Column} & \text{Both } L_{eX}/D \text{ and } L_{eY}/b \text{ are } \leq 12, \text{ and at least more than } 3 \\ \text{Long Column} & \text{Either } L_{eX}/D \text{ or } L_{eY}/b \text{ is } > 12 \end{cases}$$

where

b = Cross-section dimension perpendicular to the minor axis,

D = Cross-section dimension perpendicular to the major axis,

L_{eX} = Effective length when bending about the major axis, and

L_{eY} = Effective length when bending about the minor axis.

A strut shall have only axial stress f_{ax} (and no bending stresses) under design loads arising out of any load combination specified in 6.4.

8.8.2 Effective Length of Compression Members

In the absence of more exact analysis based on detailed geometric nonlinear analysis of the whole structure as per principles of mechanics, the effective lengths l_{eX} and l_{eY} of a compression member (namely concrete columns, beam-columns and walls) for bending about its major and minor axes, respectively, shall be estimated by any of the following methods:

- Generalized method, or
- Prescriptive method,

under the action of loads corresponding to the basic load combinations specified for strength criteria in Table 3.

8.8.2.1 Generalized method

The effective length L_e of a compression member shall be estimated by the following procedure:

- a) Estimate normalized rotational restraints factor γ_1 and γ_2 at the member ends and normalized translational restraint factor T using:

$$\gamma_1 = \frac{1}{1 + R_1},$$

$$\gamma_2 = \frac{1}{1 + R_2}, \text{ and}$$

$$T = \frac{T_1 T_2 / (T_1 + T_2)}{EI/L^3},$$

where

$$R_1 = \frac{M_1/\theta_1}{EI/L},$$

$$R_2 = \frac{M_2/\theta_2}{EI/L},$$

$$T_1 = \frac{(V_s - V_1)/v_1}{EI/L^3}, \text{ and}$$

$$T_2 = \frac{(V_s - V_2)/v_2}{EI/L^3},$$

wherein

M_1 = bending moment induced at the top of the compression member,

M_2 = bending moments induced at the bottom of the compression member,

V_s = shear force in the structure in the storey in which the effective length of the member is being estimated,

V_1 = shear force at the top of the compression member in the storey in which the effective length is being estimated,

V_2 = shear force at the bottom of the compression member in the storey in which the effective length is being estimated;

These stress resultants shall be those corresponding to the basic load combination 15 given in Table 3.

- b) Read the effective length factor k from Fig. 17, 18, 19 and 20 and 21, using γ_1 , γ_2 and T estimated in (a) above, and the shear deformation factor β as:

$$\beta = \frac{12EI/L^3}{GA_s/L},$$

where

E = Modulus of elasticity of concrete as specified in **7.2.2.1**,

G = Shear modulus of concrete = $E/2(1 + \nu)$, where ν is Poisson's ratio as specified in **7.1.4.2**,

A_s = Effective area of the cross-section as specified in **8.2.1.1**,

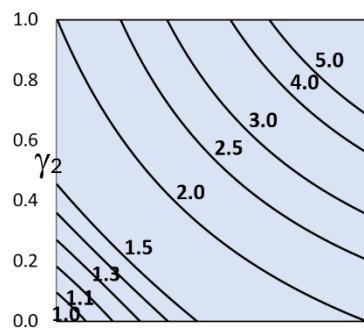
I = Effective second moment of area of the cross-section as specified in **8.2.1.1**,
and

L = Unsupported length of the compression member.

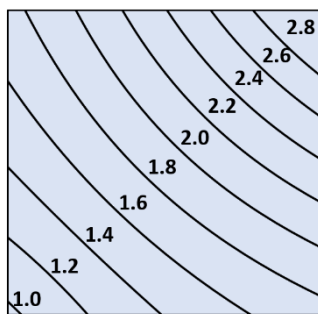
c) Estimate the effective length L_e as:

$$L_e = kL,$$

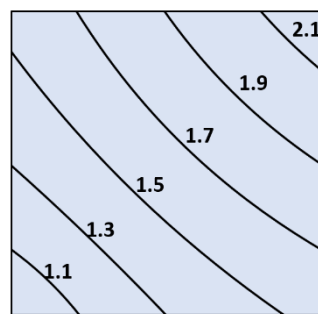
where k is the effective length factor, and L is the unsupported length.



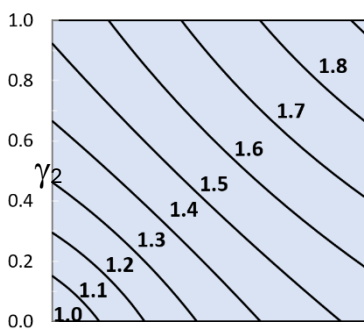
(a)



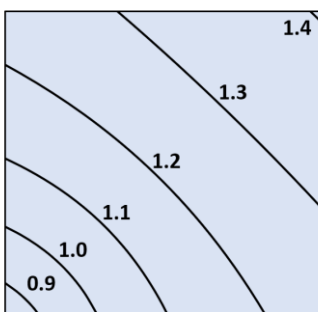
(b)



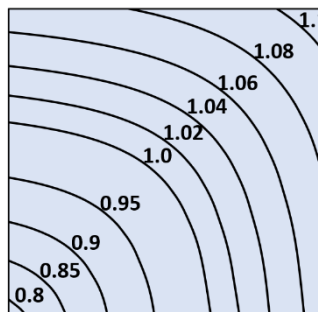
(c)



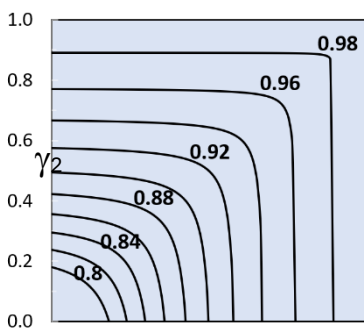
(d)



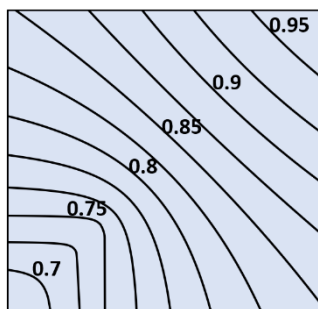
(e)



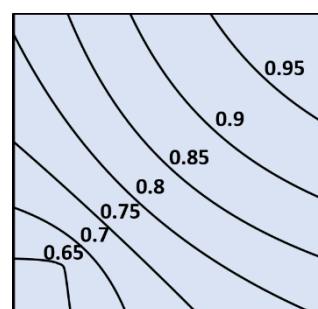
(f)



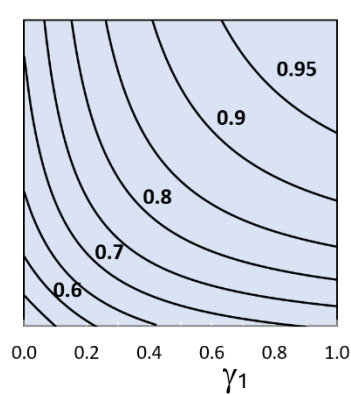
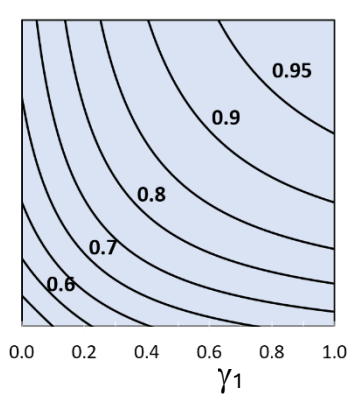
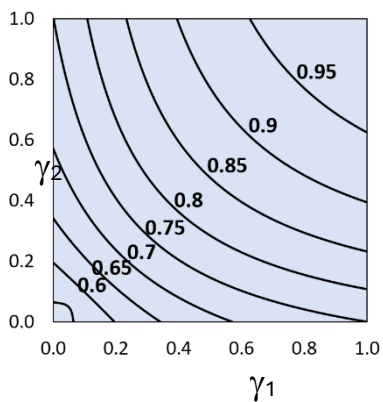
(g)



(h)



(i)



(j) (k) (l)

FIG. 17 k -factor charts considering only flexural deformations for $\beta=0$ with non-dimensional relative translational restraint T of: (a) $T=0$, (b) $T=1$, (c) $T=2$, (d) $T=3$, (e) $T=5$, (f) $T=8$, (g) $T=10$, (h) $T=15$, (i) $T=20$, (j) $T=30$, (k) $T=50$, and (l) $T=100$

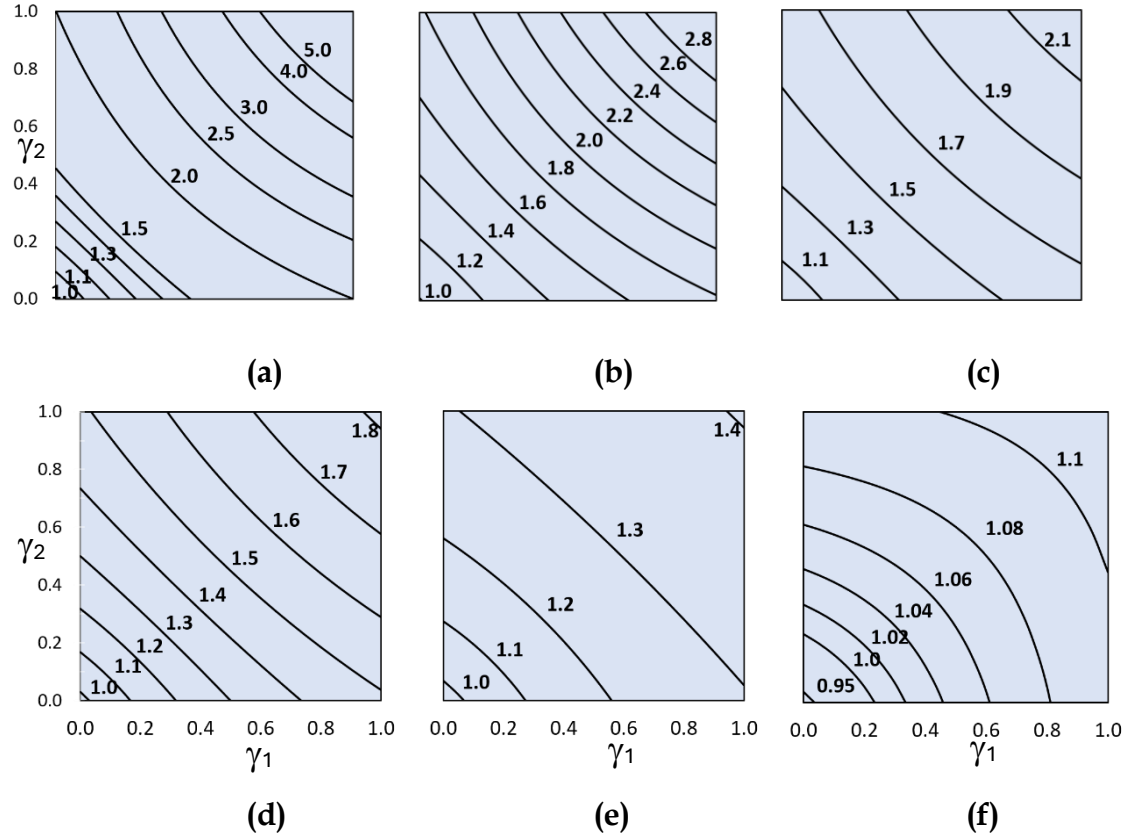


FIG. 18 k -factor charts considering both flexural and shear deformations for $\beta=1$ with non-dimensional relative translational restraint T of: (a) $T=0$, (b) $T=1$, (c) $T=2$, (d) $T=3$, (e) $T=5$, and (f) $T=8$

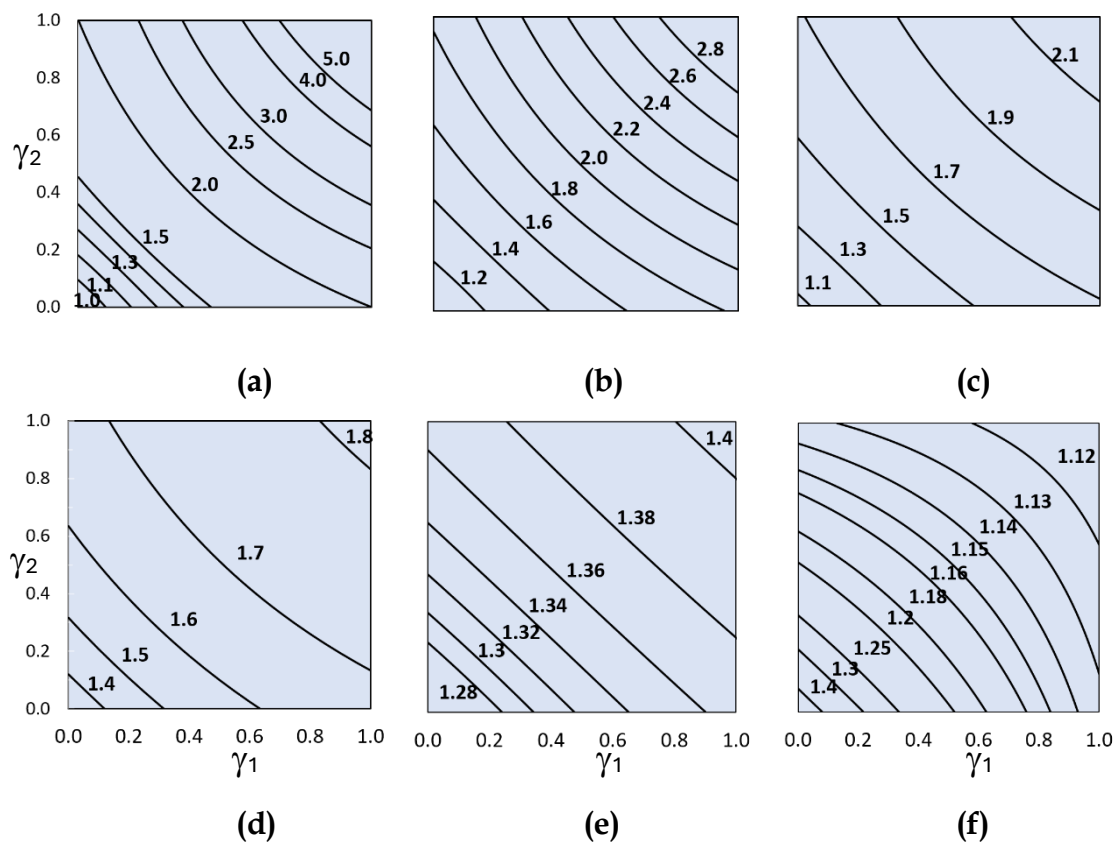


FIG. 19 k -factor charts considering both *flexural and shear deformations* for $\beta=2$ with *non-dimensional relative translational restraint* T of: (a) $T=0$, (b) $T=1$, (c) $T=2$, (d) $T=3$, (e) $T=5$, (f) $T=8$

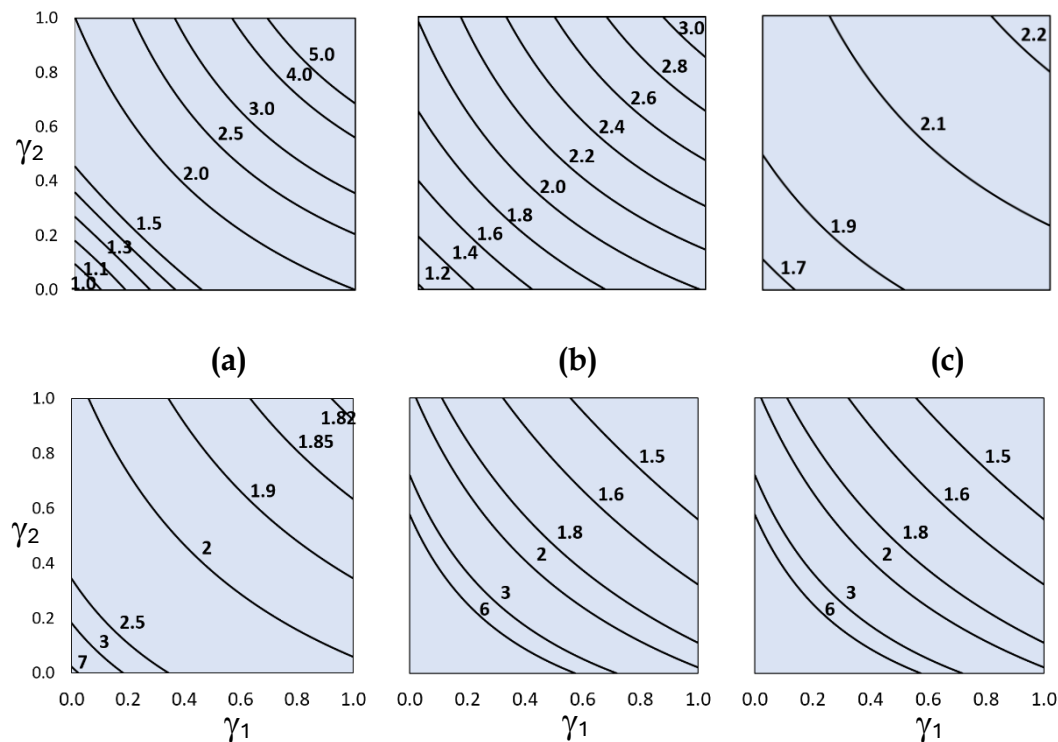


FIG. 20 k -factor charts considering both *flexural and shear deformations* for $\beta=5$ with *non-dimensional relative translational restraint* T of: (a) $T=0$, (b) $T=1$, (c) $T=2$, (d) $T=3$, (e) $T=5$, and (f) $T=8$

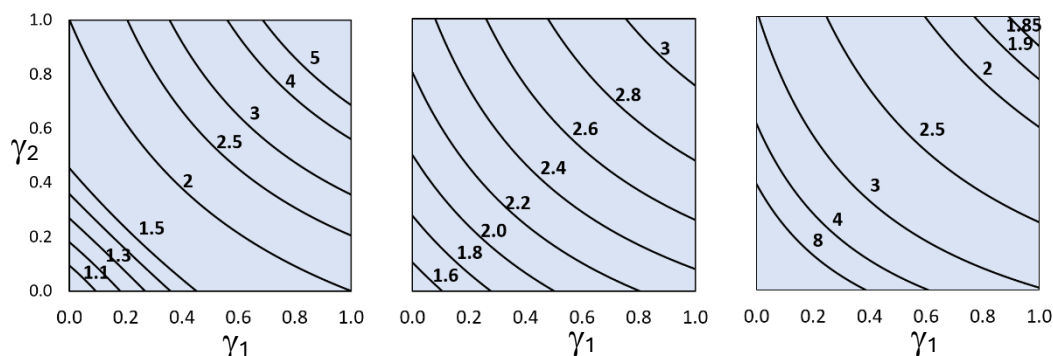


FIG. 21 k -factor charts considering both *flexural and shear deformations* for $\beta=8$ with *non-dimensional relative translational restraint* T of: (a) $T=0$, (b) $T=1$, and (c) $T=2$

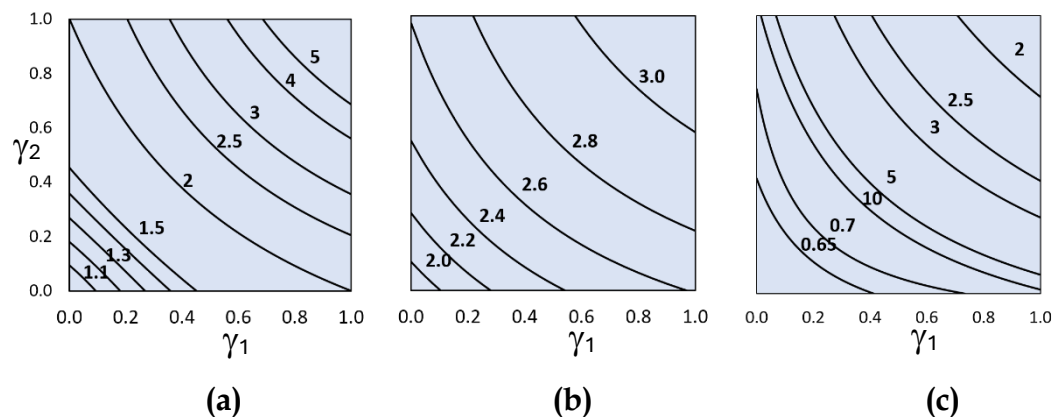


FIG. 22 k -factor charts considering both *flexural and shear deformations* for $\beta=10$ with *non-dimensional relative translational restraint* T of: (a) $T=0$, (b) $T=1$, and (c) $T=2$

8.8.2.2 Prescriptive method

Only two extreme cases of lateral restraint are considered. The restraint against sway or no-sway shall be determined by evaluating the stability index Q specified in 8.2.3.

8.8.2.2.1 Columns and beam-columns

The effective length L_e of a column and beam-column shall be estimated as per Table 34.

8.8.2.2.2 Structural wall

The effective height H_{we} of a structural wall shall be taken as given below:

- a) When the stability index Q at any storey of the structure is 0.04 or less, the structural wall shall be considered to be braced, and

$$H_{we} = 2H,$$

where

H = Height of the structural wall from the base of the structure to the roof.

- b) When the stability index Q at any storey of the structure is more than 0.04, the structural wall shall be considered to be unbraced, and if

- 1) restrained against rotation at both ends by floors or intersecting walls,
 $H_{we} = \text{Min}[0.75H_w; 0.75L_1]$

- 2) not restrained against rotation at both ends by floors or intersecting walls:

$$H_{we} = \text{Min}[H_w; L_1]$$

where

H_w = Unsupported height of the structural wall, and

L_1 = Horizontal distance between centers of lateral restraint.

Table 34 Effective lengths of Compression Members
 (Clause 8.8.2.2.1)

Sl No.	End Restraint of Member				Symbol	Effective Length l_{eX} or l_{eY}	
	End 1		End 2			Theoretical	Recommended
	Translation	Rotation	Translation	Rotation			
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Fully restrained	Fully restrained	Fully restrained	Fully restrained		$0.5l_u$	$0.65l_u$
ii)	Fully restrained	Fully restrained	Fully restrained	Not restrained		$0.7l_u$	$0.8l_u$
iii)	Fully restrained	Not restrained	Fully restrained	Not restrained		l_u	l_u
iv)	Not restrained	Fully restrained	Fully restrained	Fully restrained		l_u	$1.2l_u$
v)	Not restrained	Fully restrained	Fully restrained	Not restrained		$2l_u$	$2l_u$
vi)	Not restrained	Not restrained	Fully restrained	Fully restrained		$2l_u$	$2l_u$

NOTE — l_u is the unsupported length of compression member.

8.9 Redistribution of Moments

Redistribution of bending moments obtained from conventional analysis may be carried out only in continuous beams and statically indeterminate moment frames, provided the following conditions are satisfied:

- a) Equilibrium between external loads applied and internal forces induced is maintained.
- b) After redistribution of bending moments, the following are complied with:

- 1) The revised bending moment capacity provided at any section of a member is not less than 70 percent when limit state method of design is used (and 85 percent when allowable stress method of design is used of the numerically largest bending moment demand at that section obtained from the linear elastic analysis considering all combinations of loads specified in this standard);
- 2) The revised bending moment demand at any section of a member is not more than 30 percent when limit state method of design is used (and 15 percent when allowable stress method of design is used of the numerically largest bending moment demand at that section obtained from the linear elastic analysis considering all combinations of loads specified in this standard); and
- 3) At sections where the revised bending moment capacity provided M^* after redistribution is less than the numerically largest bending moment demand M_0 at that section obtained from the linear elastic analysis considering all combinations of loads specified in this standard, the following relationship is satisfied:

$$\frac{x_L}{d} + \frac{M_0 - M^*}{M_0} \leq 0.6$$

where

x_L = Limiting depth of neutral axis at the section, and

d = Effective depth at the section; and

- 4) In RC structures with moment frame as the structural system and more 4 storeys tall, the fraction $(M_0 - M^*)/M_0$ in 7.3.1.4 (2)(c) is less than 0.10.
- c) No redistribution of moments shall be permitted between beams and columns of frame members when:

- 1) Bending moment and shear forces coefficients under gravity loads are taken from simplified analysis of continuous beams and statically indeterminate moment frames,
- 2) Bending moments at the beam ends are completely released in structural analysis, and
- 3) Bending moments are estimated by nonlinear analysis.

8.10 Deflection and Crack Width in Concrete Members

Deflection and crack width shall be estimated in members under the action of all loads envisaged to appear on the structure. These shall be estimated under the action of service level loads acting as per combinations specified in 6.4.

8.10.1 Deflection

The deflection of a member at any time t due to loads applied transverse to it shall be estimated as specified hereunder.

8.10.1.1 Beams and slabs

The deflection so estimated shall not exceed the values given in 8.4.1.2.

Unless more exact estimates are made, the total transverse deflection Δ_D of beams under the action of loads corresponding to load combinations to meet serviceability criteria specified in **Table 4** shall be estimated as:

$$\Delta_D = \Delta_{DS} + \Delta_{DL,S} + \Delta_{DL,C} + \Delta_{DT},$$

where the respective terms are specified hereunder.

a) Short-term deflection Δ_{DS}

The short-term deflection due to immediate elastic deformation may be estimated by the usual linear elastic methods of deflection using the short-term modulus of elasticity of concrete E_c given by 7.1.4.1 and an effective moment of inertia I_e given by:

$$I_e = \frac{I_{cr}}{1.2 - \left(\frac{M_{cr}}{M} \right) \left(\frac{z}{d} \right) \left(1 - \frac{x}{d} \right) \left(\frac{b_w}{b_c} \right)},$$

where

I_{cr} = moment of inertia of the cracked section;

$$M_{cr} = \text{cracking bending moment of the section} = \frac{f_{cr} I_g}{y_t}$$

in which f_{cr} is the modulus of rupture of concrete, I_g the gross cross-section about the centroidal axis considering the Modulus of concrete for the area corresponding to that of steel reinforcement, and y_t the distance from centroidal axis of gross cross-section (considering the Modulus of concrete for the area corresponding to that of steel reinforcement) to the extreme face of concrete in tension;

M = maximum unfactored design bending moment,

z = lever arm between the centroid of compression area and extreme layer of steel,

d = effective depth of the cross-section,

b_w = breadth of the web of the cross-section,

b_c = breadth of the compression face of the cross-section,

The effective moment of inertia I_e shall be taken as bounded by:

$$I_{cr} \leq I_e \leq I_g,$$

where I_g is the gross moment of inertia of the cross-section.

For continuous beams, the total deflection shall be estimated using the equivalent value of I_{eq} obtained by the weighted averaging as per:

$$I_{eq} = k \left(\frac{I_{eL} + I_{eR}}{2} \right) + (1 - k) I_{eM},$$

where

I_{eL} = I_e at left support,

I_{eR} = I_e at right support,

I_{eM} = I_e at mid-span, and

k = Weighting factor given by Table 35.

Table 35 Weighting factor k to estimate equivalent I_{eq} of the member

[Clause 8.10.1.1(a)]

$\frac{M_L + M_R}{M_{FEM,L} + M_{FEM,R}}$	≤ 0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
k	0	0.03	0.08	0.16	0.30	0.50	0.73	0.91	0.97	1.0

where

M_L = bending moment at left support,

M_R	= bending moment at right support,
$M_{FEM,L}$	= fixed end moment (due to DL and IL) at left support, and
$M_{FEM,R}$	= fixed end moment (due to DL and IL) at right support.

b) Long-term deflection $\Delta_{DL,S}$ due to shrinkage

The deflection due to shrinkage may be computed from the following equation:

$$\Delta_{DL,S} = k_s \psi_s l^2,$$

where

$$k_s = \begin{cases} 0.500 & \text{Cantilever Members} \\ 0.125 & \text{Simply Supported Members} \\ 0.086 & \text{Members continuous at one end} \\ 0.063 & \text{Fully continuous Members} \end{cases}$$

$$\psi_s = \text{shrinkage curvature} = k_r \left(\frac{\epsilon_{cs}}{D} \right),$$

wherein

ϵ_{cs} = ultimate shrinkage strain induced in concrete (as per 7.1.4.5),

D = overall depth of the concrete member, and

$$k_r = \begin{cases} 0.72 \left(\frac{p_t - p_c}{\sqrt{p_t}} \right) \leq 1.0 & 0.25 \leq (p_t = p_c) < 1.0 \\ 0.65 \left(\frac{p_t - p_c}{\sqrt{p_t}} \right) \leq 1.0 & (p_t = p_c) \geq 1.0 \end{cases}, \text{ and}$$

where p_t and p_c are the percentages of reinforcing steel on the tension and compression faces, respectively, and l is the clear length of the member.

c) Long-term deflection $\Delta_{DL,C}$ due to creep

The creep deflection due to permanent loads may be obtained from the following equations:

$$\Delta_{DL,C} = \Delta_{DL,C\_Initial+Creep} - \Delta_{DL,C\_Initial}$$

where

$\Delta_{DL,C\_Initial+Creep}$ = initial deflection plus creep deflection due to permanent loads obtained using elastic analysis with effective modulus of elasticity E_{ce} of concrete taken as $E_{ce} = E_c / (1 + \theta)$, wherein E_c is given by 7.1.4.1 and θ , the creep coefficient, by 7.1.4.4; and

$\Delta_{LC\_Initial}$ = initial deflection due to permanent loads obtained using elastic analysis with E_c given by 7.1.4.1.

d) *Deflection Δ_{DT} due to temperature effects*

The deflection due to temperature effects (due to heat and not fire) may be calculated as:

$$\Delta_{DT} = k_s \psi_T l^2,$$

where

k_s = modifier depending on end rotational restraint as given in 7.3.3.1(a)(2),

ψ_T = temperature curvature = $k_r \left(\frac{\varepsilon_{DT}}{D} \right)$, and

l = clear length of the Member.

wherein

k_r = modifier depending on percentages of tension and compression steels as given in 7.3.3.1(a)(1),

ε_{DT} = design temperature strain imposed on concrete = $\alpha \Delta T$, and

D = overall depth of the concrete member,

in which

α = coefficient of thermal expansion as per 7.1.4.3, and

ΔT = differential temperature (°C) between the top and bottom faces of the beam.

8.10.1.2 Columns and walls

The lateral deflections of columns and walls shall be taken as obtained from detailed structural analysis of the structure.

8.10.2 Crack Width Calculation for Beams

8.10.2.1 The crack width w_c at the surface of the member shall be estimated as:

$$w_c = \frac{3d' \varepsilon_m}{1 + \frac{2(d'-c)}{(h-x)}},$$

where

d' = distance from point considered on the surface to the nearest surface of the outermost layer of longitudinal bar,

ε_m = average strain at the level of the outermost layer of longitudinal bar,

c = cover to the outermost layer of longitudinal bar,

h = overall depth of the member perpendicular to the axis of bending, and

x = depth of outermost compression face of member from neutral axis at the section considered,

provided that strain in the outermost layer of tension reinforcement is limited to $0.8f_y/E_s$.

8.10.2.2 The average steel strain ε_m shall be estimated on the basis of the following assumptions (see Fig. 23):

- The concrete and steel are considered to be fully elastic in both tension and in compression;
- The elastic modulus E_s (in MPa) of reinforcing steel is as per 7.2.2.1; and
- The elastic modulus E_c (in MPa) of concrete is as per 7.1.4.1 in both compression and tension.

In Fig. 23,

f_c = Compressive stress in the outermost fiber of concrete, and

f_s = Tensile stress in the outermost layer of longitudinal steel.

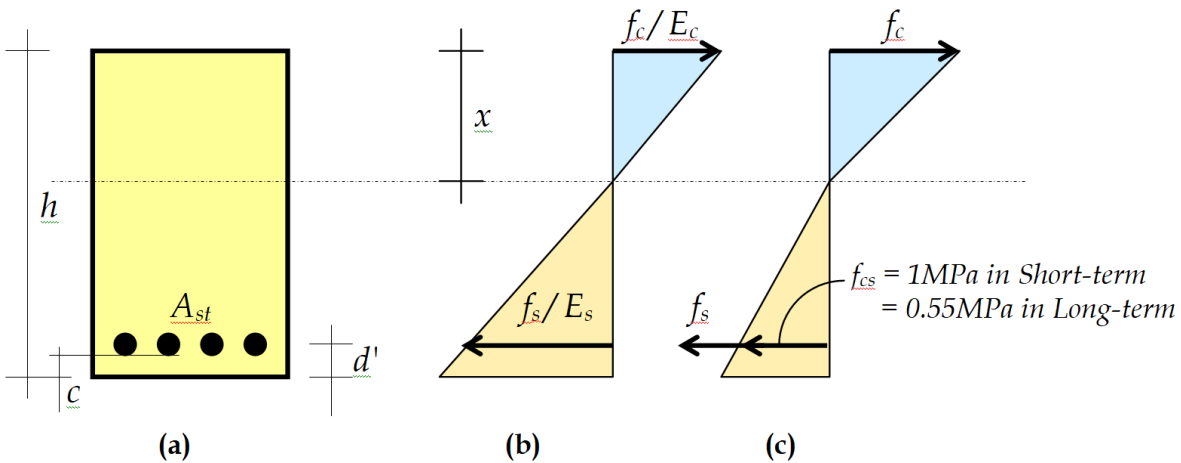


FIG. 23 STATE OF STRESS AT THE CRACKED SECTION WHERE CRACK WIDTH IS BEING ESTIMATED: (A) SECTION GEOMETRY, (B) STRAIN DISTRIBUTION, AND (C) STRESS DISTRIBUTION

8.10.2.3 Alternatively, ε_m may be approximated on the basis of a cracked section and then reduce this by an amount equal to the tensile force generated in concrete divided by the

steel area. The tensile force acting over the tension zone in concrete shall be estimated from a triangular distribution of stress in concrete, with zero at the neutral axis and 1 MPa at the level of the centroid of the tension steel in the short-term and 0.55 MPa in the long-term. For a rectangular tension zone, this provides a simple expression for ε_m :

$$\varepsilon_m = \varepsilon_1 - \frac{b(D-x)(a-x)}{3(d-x)E_s A_{st}}$$

where

- ε_1 = strain in steel at the level considered, calculated ignoring the stiffening of the concrete in the tension zone,
- b = width of the section at the centroid of the tension steel,
- a = distance from the compression face to the point at which the crack width is being calculated
 $= D - c$,
- d = effective depth of the section
 $= D - d'$, and
- A_{st} = area of tension reinforcement.

8.10.3 Temperature Stresses and Deformations

Structural analysis shall be performed by the finite element method to estimate the stresses and deformations induced by temperatures arising from the two load sub-combinations (given in **Table 2**) related to temperature, namely:

- a) *Temperature – Ambient Load (TAL) in Set 1 Basic Load Combination:*
This analysis shall be *linear* analysis; and
- b) *Temperature – Fire Load (TFL) in Set 2 Accidental Load Combination*
This analysis shall be nonlinear analysis, considering both *material* and *geometric* analysis.

9 STRUCTURAL DESIGN AND DETAIL

The aspects of structural design and detail specified hereunder are for estimating the design capacities of the members, connections and structure on:

- a) Strength capacities of members, namely axial force, shear force, bending moment and torsional moment capacities;
- b) Overall displacement capacity of the structure, and elastic deformation capacities of members, namely deflection and rotation under the action of combination of loads given in Table 3; and
- c) Strength capacities of joints and anchorages to transmit forces through them.

These design capacities are to be examined against the design demands (such as, axial force, shear force, bending moment, torsional moment, displacements, drifts and rotations) through structural analysis performed as specified in 8.

With respect to earthquake ground shaking:

- a) The capacities of RC members and RC structure to resist effects of earthquake ground shaking shall be considered as specified in IS 13920 (Parts 1 to 11) for different structures. Provisions of the said parts of IS 13920 shall be complied with in addition to those specified in this standard; in case of conflict, provisions of the said parts IS 13920 shall prevail; and
- b) The assessment of the capacity of RC members and the RC buildings to resist effects of earthquake ground shaking, when required, shall be considered as specified in IS 13935 (Parts 1 to 11) for different structures. Provisions of the said parts of IS 13935 shall be complied with to strengthen members and the structure when meeting the performance requirements laid down as the objective of strengthening of the structure before an earthquake or after it is damaged during an earthquake.

In the clauses hereunder, the word *bars* shall mean reinforcing steel bars conforming to IS 1786. When reinforcing bars made of other materials are used, detailed experimentation shall be performed to be satisfied with the behaviour of members made with such materials, and design calculations suitably modified by the designer at the request of the owner and to the satisfaction of the competent authority.

9.1 STABILITY

Under the action of the design characteristic loads, the structure as a whole shall be stable against the following:

- a) Overturning,
- b) Sliding,
- c) Floatation,
- d) Subsidence, and
- e) Elastic and inelastic stability of the structure and members.

The stability of the structure shall be checked for the temporary enabling-work structures also, which are employed during construction, under the action of the realistic loads appearing on them.

9.1.1 Overturning

A structure deemed to be stable against overturning shall satisfy:

$$M^{DO} < M^{RO},$$

where

M^{DO} = factored overturning moment demand that causes overturning of the structure as a whole

$$= \begin{cases} 1.2M_{DL}^{DO} + 1.4M_{IL}^{DO} + 1.5M_{EP}^{DO} + 1.5M_{WL}^{DO} & \text{with Wind Load} \\ 1.2M_{DL}^{DO} + 1.4M_{IL}^{DO} + 1.5M_{EP}^{DO} + 1.5M_{EL}^{DO} & \text{with Earthquake Load} \end{cases}, \text{ and}$$

in which

M_{DL}^{DO} = moment demand due to dead load specified in **6.1.1**,

M_{IL}^{DO} = moment demand due to imposed load specified in **6.1.2**,

M_{EP}^{DO} = moment demand due to earth pressure load specified in **6.1.4**, including the effects of earthquake shaking, when applicable,

M_{WL}^{DO} = moment demand due to wind load specified in **6.1.6**, and

M_{EL}^{DO} = moment demand due to earthquake load estimated using the overstrength lateral force $V_{\Omega EL}$ (considering the same ratio of distribution of forces along the height as considered in design) given by:

$$V_{\Omega EL} = \begin{cases} V_{BD}(1 + 0.75(R - 1)) & 1 \leq R < 3 \\ 2.5V_{BD} & 3 \leq R \leq 5 \end{cases}, \text{ and}$$

wherein

V_{BD} = design earthquake base shear of the structure, specified in IS 1893, and

R = elastic force reduction factor used in the estimation of V_B of the structure as specified in IS 1893; and

M^{RO} = unfactored restoring moment that helps in resisting overturning of the structure,

$$= 0.9M_{DL}^{DR} + 0.5M_{EP}^{DR} + M_A^{DR},$$

in which

M_{DL}^{DR} = restoring moment due to dead load defined specified in **6.1.1**,

M_{EP}^{DR} = restoring moment due to earth pressure load specified in **6.1.4**, if any, and

M_A^{DR} = restoring moment due to characteristic values of resistance offered by positive anchorages provided in the structure.

The anchorages or counterweights provided for overhanging members (during construction and service stages) shall be such that static equilibrium shall remain even when overturning moment is *doubled*.

9.1.1.1 Uplift at column bases

Uplift shall not be acceptable in columns resting on individual footings.

In other foundations, partial uplift may be admitted subject to safe bearing capacity not being exceeded as specified hereunder depending on the soil types:

- a) *Hard soil and rock* : Up to 15 percent area can uplift,
- b) *Medium hard soil* : Up to 10 percent area can uplift, and
- c) *Soft soil* : Not uplift permitted.

When the above requirements are violated, the structural system shall be revised to avoid such a condition. The footing of a column under axial tension shall be combined with those of other adjoining columns, such that:

- a) the combined footing does not have any uplift, and
- b) the structure meets all requirements of strength criteria.

9.1.2 Sliding

A structure deemed to be stable against sliding, under the most adverse combination of the design loads, shall satisfy:

$$F^{DS} < F^{RS},$$

where

F^{DS} = Factored sliding shear force demand that causes sliding of the structure as a whole

$$= \begin{cases} 1.5(F_{DL}^{DS} + F_{IL}^{DS} + F_{EP}^{DS} + F_{WL}^{DS}) & \text{with Wind Load} \\ 1.5(F_{DL}^{DS} + F_{IL}^{DS} + F_{EP}^{DS} + F_{EL}^{DS}) & \text{with Earthquake Load} \end{cases}$$

in which

F_{DL}^{DS} = sliding shear force demand due to dead load specified in Table 3,

F_{IL}^{DS} = sliding shear force demand due to imposed load specified in Table 3,

F_{EP}^{DS} = sliding shear force demand due to earth pressure load specified in Table 3, including the effects of earthquake shaking, when applicable,

F_{WL}^{DS} = sliding shear force demand due to wind load specified in Table 3,

F_{EL}^{DS} = sliding shear force demand due to earthquake load specified in Table 3,
estimated using the overstrength lateral force $V_{\Omega EL}$ given by:

$$V_{\Omega EL} = \begin{cases} V_{BD}(1 + 0.75(R - 1)) & 1 \leq R < 3 \\ 2.5V_{BD} & 3 \leq R \leq 5 \end{cases}, \text{ and}$$

wherein

V_{BD} = design earthquake base shear of the structure, specified in IS 1893,
and

R = elastic force reduction factor used in the estimation of V_B of the
structure as specified in IS 1893, and

F_{DL}^{RS} = sliding shear force resistance due to dead load specified in Table 3; and

F^{RS} = Design sliding shear force resistance that helps in resisting overturning of the
structure as a whole

$$= 0.9F_{DL}^{RS} + F_{SKA}^{RS} + 0.5ZF_{EP}^{RS},$$

in which

F_{SKA}^{RS} = sliding shear force resistance due to characteristic values of resistance
offered by positive shear keys and positive anchors provided in the
structure,

F_{EP}^{RS} = sliding shear force resistance (on horizontal surfaces) due to vertical earth
pressure load specified in Table 3, and

Z = earthquake zone factor as specified in Table 2 of IS 1893 (Part 1)
corresponding to the earthquake zones in which the site is located.

The resistance shall be based on positive mechanisms. The resisting force provided by
effects of friction:

- shall be ignored when considering earthquake loads on the members in the parts
of the structure that are above ground, but
- may be considered when dealing with foundations and parts of the structure that
are buried in soil.

9.1.3 Floatation

A structure deemed to be stable against floatation shall satisfy:

$$M^{DF} < M^{RF},$$

where

M^{DF} = overturning moment demand that causes overturning of the structure as a whole

$$= \begin{cases} 1.2M_{DL}^{DF} + 1.4M_{IL}^{DF} + 1.5M_{WL}^{DF} & \text{with Wind Load} \\ 1.2M_{DL}^{DF} + 1.4M_{IL}^{DF} + 1.5M_{EL}^{DF} & \text{with Earthquake Load} \end{cases}$$

in which

M_{DL}^{DF} = overturning moment demand due to dead load specified in Table 3,

M_{IL}^{DF} = overturning moment demand due to imposed load specified in Table 3,

M_{WL}^{DF} = overturning moment demand due to wind load specified in Table 3, and

M_{EL}^{DF} = overturning moment demand due to earthquake load specified in Table 3; and

M^{RF} = restoring moment that helps in resisting overturning of the structure as a whole

$$= 0.9M_{DLB}^{RF} + M_A^{RF},$$

in which

M_{DLB}^{RF} = restoring moment due to that part of dead load specified in Table 3, which accounts for effects of buoyancy force on the structure, and

M_A^{RF} = restoring moment due to characteristic values of resistance offered by positive anchorages provided in the structure, when provided.

The restoring moment provided by imposed load shall be ignored.

9.1.4 Subsidence

The provisions of this standard apply to concrete structures that are rested on competent ground. Subsidence shall be considered when evaluating the competence of the ground to hold concrete structures on it without becoming unstable under the action of the loads prescribed in 6. The factors causing subsidence of the ground include:

- a) Liquefaction during strong earthquake shaking;
- b) Asperities (or holes) in ground, either natural or filled ground; and
- c) Sudden downward movement during lateral spreading under strong earthquake shaking.

9.2 BASIC QUANTITIES

The calculations related to structural design shall be performed using quantities defined as hereunder. Also, limits specified hereunder shall be applicable for the limits on reinforcements.

9.2.1 Effective Dimensions

In the absence of more accurate determination, the provisions given hereunder shall be used for the quantities commonly used in structural design.

9.2.1.1 Effective span

9.2.1.1.1 Beams and slabs

The effective span l_e of a slab or beam shall be taken as:

a) Simply Supported Member

$$l_e = \begin{cases} \text{Min}[l_{clear} + d; l_{cc}] & \text{Not integrally built with its supports} \\ l_{cc} & \text{Resting on rocker and roller bearings} \end{cases}$$

b) Continuous Member

1) When $w_s \leq \text{Min}[l_{clear}/12; 600\text{mm}]$

$$l_e = \begin{cases} \text{Min}[l_{clear} + d; l_{cc}] & \text{Not integrally built with its supports} \\ l_{cc} & \text{Resting on rocker and roller bearings} \end{cases}$$

2) When $w_s > \text{Min}[l_{clear}/12; 600\text{mm}]$

$$l_e = \begin{cases} l_{clear} & \text{In the end span with one end fixed and other continuous, or in the intermediate span} \\ \text{Min}\left[l_{clear} + \frac{d}{2}; l_{clear} + \frac{w_s}{2}\right] & \text{In the end span with one end free and other continuous} \\ l_{clear} + \frac{w_s}{2} & \text{Resting on rocker and roller bearings} \end{cases}$$

c) Cantilever Member

$$l_e = \begin{cases} l_{cs} & \text{At the end of the continuous span} \\ l_{clear} + \frac{d}{2} & \text{All other cases} \end{cases}$$

where

l_{clear} = clear distance between supports,

l_{cc} = distance between center to center of supports,

l_{cs} = distance between free end to center of support,

d = effective depth of the member, and

w_s = width of the support.

9.2.1.1.2 Staircase

The effective span l_e of the slab in a staircase (see Fig. 24 and Table 36) that has no stringer beams shall be:

$$l_e = \begin{cases} l_{cc} & \text{Where supported at top \& bottom risers} \\ & \text{by beams spanning parallel to the risers} \\ l_g + \text{Min}[w_{\text{landing,top}}; w] + \text{Min}[w_{\text{landing,bot}}; w] & \text{Where spanning on to edge of a landing} \\ & \text{slab spanning parallel to the risers} \end{cases}$$

where

l_{cc} = Centre-to-centre distance between the beams,

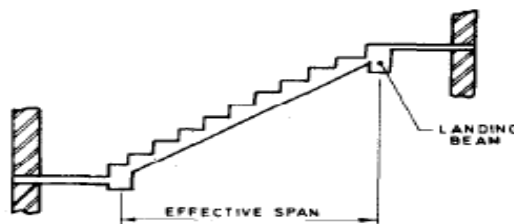
l_g = Going of the staircase measured horizontally;

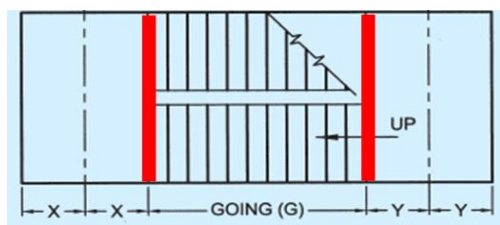
w = Width of the stair slab,

$w_{\text{landing,top}}$ = Width of the landing at the top end of the stair slab, and

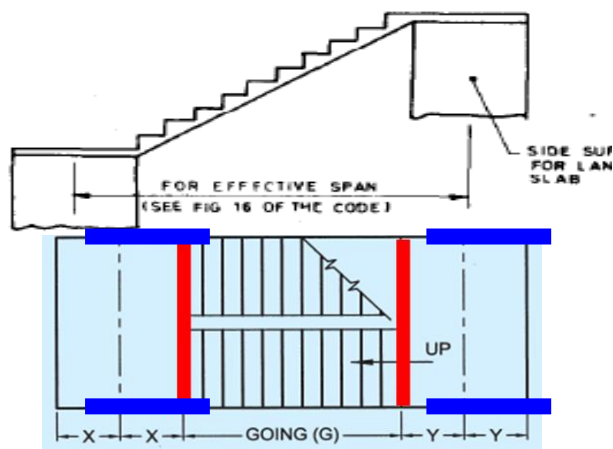
$w_{\text{landing,bot}}$ = Width of the landing at the bottom end of the stair slab.

Where spanning on to the edge of a landing slab, which spans in the same direction as the stairs, the stair slab shall be considered to act together with the landing slabs to form a single slab and the span determined as the centre-to-centre distance of the beams or walls supporting the slab.

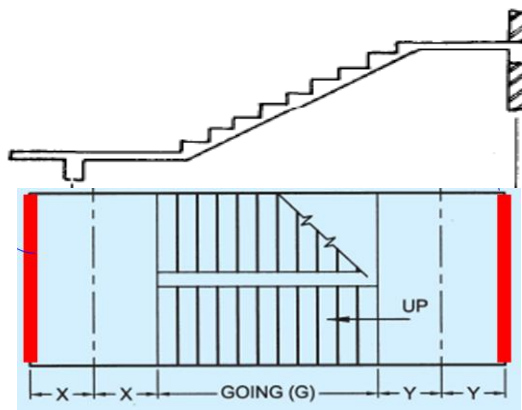




(a)



(b)



(c)

FIG. 24 EFFECTIVE SPAN OF STAIR SLABS SUPPORTED AT EACH END BY LANDING SLABS SPANNING PARALLEL TO THE RISERS

Table 36 Effective Span of Stair Slabs supported at each end by Landing Slabs spanning parallel to the Risers
(Clause 9.2.1.1.2)

X	Y	Effective Span (m)
$< 1 \text{ m}$	$< 1 \text{ m}$	$G + X + Y$
$< 1 \text{ m}$	$\geq 1 \text{ m}$	$G + X + 1 \text{ m}$

$\geq 1\text{m}$	$< 1\text{ m}$	$G + 1m + Y$
$\geq 1\text{m}$	$\geq 1\text{m}$	$G + 1m + 1m$

9.2.1.2 Effective depth

The effective depth d of a reinforced or prestressed concrete member (other than a deep beam) shall be taken as the distance between the centroid of the area of tension reinforcement and the outermost fiber on the compression face, excluding the thickness of any material not cast monolithically with the member and of any concrete provided to allow for wear. This shall be applicable to slabs, shallow beams, columns and structural walls, but not to deep beams and corbels. For deep beams, lever arm shall be used instead of effective depth, as per 9.3.2.2(c).

9.2.1.3 Effective width of flange

Unless determined more accurately, the effective width b_{fe} of the flange of a beam in a monolithic beam-slab system of a structure shall be taken as:

a) *Beam resisting compression in flanged members*

$$b_{fe} = \begin{cases} \frac{l_0}{6} + b_w + 6D_f & \text{T - Beams} \\ \frac{l_0}{12} + b_w + 3D_f & \text{L - Beams} \end{cases} ,$$

$$\leq b_w + 0.5(l_{c1} + l_{c2})$$

and of an isolated beam with flange:

$$b_{fe} = \begin{cases} b_w + \frac{l_0}{\frac{l_0}{b_f} + 4} & \text{T - Beams} \\ b_w + \frac{0.5l_0}{\frac{l_0}{b_f} + 4} & \text{L - Beams} \end{cases} ,$$

$$\leq b_f$$

where

b_f = actual width of flange,

b_w = breadth of the web,

D_f = depth of the flange,

b_f = actual width of the flange of the isolated beam,

l_0 = distance between points of zero moments in the beam,

= $0.7l_e$ in members of continuous beams and moment frames,

l_{c1} = clear distance from web of flanged beam to that of adjoining beam on side 1,

l_{c2} = clear distance from web of flanged beam to that of adjoining beam on side 2, and

l_e = effective span of the flanged beam.

- b) Beam resisting tension in flanged members, or beams in structures located in Earthquake Zones IV, V and VI as per IS 1893 (Part 1), $b_{fe} = b_w$

9.2.1.4 Effective height

The effective height of a column or wall shall be taken as per 8.8.2.

9.2.1.5 Cover to reinforcement

9.2.1.5.1 Nominal Cover

9.2.1.5.1.1 It shall be taken as the clear thickness of plain concrete from the exterior edge of the concrete member to the bar nearest to the exterior surface, namely the distance from the exterior face to the nearest:

- transverse bars when stirrups, links or ties are used,
- other bars when stirrups, links or ties are not used, and
- outer face of the coupler, when threaded couplers are used for splicing of bars.

9.2.1.5.1.2 The provisions specified herein are applicable to cast in-situ normal aggregate concretes. Hence:

- a) Lesser nominal cover than those specified herein may be permissible in precast components, if permitted from durability and fire considerations;
- b) Higher nominal cover than those specified herein shall be used when lightweight or porous aggregate concretes are used; and
- c) Higher nominal cover for bars with couplers threaded, measured from the outer face of the threaded coupler. In no case, the transverse reinforcement shall be placed at the location of the coupler.

9.2.1.5.1.3 The nominal cover shall be subjected to the tolerance specified in Table 37.

9.2.1.5.1.4 The nominal cover at a bar shall not be less than:

- $1.5d_b$, when $d_b \leq 25$ mm, and
- a) $2.0d_b$, when $d_b > 25$ mm,

where d_b is the diameter of the bar. When bars of different diameters are used, the nominal cover shall be based on the bar with the largest diameter. When couplers are used, the stirrups shall not be placed over the couplers.

9.2.1.5.2 *Effective cover*

The effective cover for each layer of bars shall be taken as the distance of the centroid of that layer of bars from the edge of the concrete member in focus, and shall be taken as nominal cover to that layer of bar plus half the diameter of the:

- a) largest of all bars provided in that layer, and
- b) largest equivalent circular bar in that layer of bars provided as bundled bars in that layer.

Table 37 Tolerance of a nominal cover to bars in concrete members

(Clause 9.2.1.5.1.3)

	<i>Tolerance (mm) based on exposure conditions</i>

<i>Effective Depth (mm)</i>	Mild	Moderate	Severe	Very Severe	Extreme
< 200	0 to +10	0 to +10	0 to +5	0 to +5	0 to +5
200 - 500	0 to +12	0 to +12	0 to +7	0 to +7	0 to +7
> 500	0 to +15	0 to +15	0 to +10	0 to +10	0 to +10

9.2.2 Absolute Dimensions

The preliminary sizing of concrete members and selection of diameter of steel bars shall comply with the sizes specified hereunder.

9.2.2.1 Nominal diameter of steel bars

Where limitations on spacing of bars are based on bar diameter, the diameter of a round bar shall be its nominal diameter. In case of bars which are ribbed, crimped, bundled or not round, it shall be taken as the diameter of an equivalent circular bar of the same axial cross-sectional area effective in resisting tension. A group of bars bundled in contact shall be treated as a single bar of nominal diameter derived from the total equivalent area using the nominal diameter of individual bars.

9.2.2.2 Sizing of concrete members

The minimum size of concrete members shall be determined based on considerations of strength, durability and fire resistance. The largest of the three values shall be adopted in design.

9.2.2.2.1 Durability Requirement

When the structure is designed to meet the requirements of durability criteria, especially to resist corrosion of steel and concrete, the concrete members shall have at least the minimum nominal cover specified by **9.2.1.5.1** (given in Table 37).

9.2.2.2.2 Fire resistance requirement

When the structure is designed to meet the requirements of robustness criteria especially to resist fire, the concrete members shall have at least the minimum sizes specified by **9.2.2.2.3**. These minimum sizes specified shall be applicable only when the minimum nominal cover requirements mentioned in **9.2.2.2.3.1** are met with.

9.2.2.2.3 Strength requirement

Based on strength criteria, concrete members shall have at least the following minimum sizes:

<i>Structural Element</i>	<i>Minimum size</i>
Slabs	125 mm thick
Beams	250 mm × 300 mm
Columns	300 mm × 300 mm
Structural walls	200 mm thick
Non-structural walls	125 mm thick
Footings	150 mm thickness at the tip of cantilever

These minimum sizes shall be adopted along with the minimum nominal cover specified in **9.2.1.5.1**.

9.2.2.2.3.1 In special residential buildings that comply with the following conditions:

- a) building has a maximum of 3 storeys;
- b) buildings has bay lengths less than 4.5m along each plan directions;
- c) storey height in any storey is less than 3.5m;
- d) building does not have a stilt storey;
- e) structural system is a moment frame or moment frame with structural wall; and
- f) at least 70 percent of the frame panels in each storey are infilled with unreinforced masonry walls;

the minimum sizes of members shall be taken as:

<i>Structural Element</i>	<i>Minimum size</i>
Slabs	115 mm thick
Beams	200 mm × 250 mm
Columns	250 mm × 250 mm
Structural walls	160 mm thick
Non-structural walls	125 mm thick

Footings	150 mm thickness at the tip of cantilever

9.2.2.3 *Maximum diameter of bars*

The reinforcing steel bars used in concrete structures shall conform to IS 1786; hereinafter the term bars shall refer to reinforcing steel bars.

9.2.2.3.1 *Longitudinal reinforcement*a) *Beams*

The maximum diameter of bar shall depend on size of the beam, but in general can be taken as 25 mm or less. When 28mm or larger bars are used, the crack width requirements specified in 8.10 and durability requirements specified in 9.5 shall be complied with.

b) *Slabs*

The maximum diameter of longitudinal bar shall be one-eighth of the total thickness of the slab.

c) *Columns*

The maximum diameter shall be $\text{Min}[b; D]/10$, where b and D are the dimensions of the cross-section of the column.

d) *Structural walls*

The maximum diameter shall depend on the size of the wall, but to be taken less than 20 mm.

9.2.2.3.2 *Transverse reinforcement*a) *Beams*

The maximum diameter shall depend on size of beam, but to be taken less than 16 mm.

b) *Columns*

The maximum diameter shall depend on size of column, but to be taken less than 16 mm.

c) *Structural walls*

The maximum diameter shall be 12 mm.

9.2.2.4 *Minimum size of bars*

9.2.2.4.1 *Longitudinal reinforcement*

a) *Beams*

The minimum diameter shall be 10 mm for hangar bars, 12 mm for main bars and 8 mm for bars placed as web reinforcement.

b) *Slabs*

The minimum diameter shall be 8 mm of both main and distribution steel bars. In case of welded mesh manufactured in factories, this provision may be relaxed as per IS 4948.

c) *Columns*

The minimum diameter shall be 12 mm.

d) *Walls*

The minimum diameter shall be 10 mm of vertical bar in the web, and 12 mm of vertical bar in the boundary element, except when bar diameter controls the crack width (such as water tanks).

9.2.2.4.2 *Transverse reinforcement*

a) *Beams*

The minimum diameter shall be 8 mm.

b) *Slabs*

When shear stress in slab exceeds the permissible value as given in **9.2.6.1(d)**, the minimum diameter taken shall be 8 mm.

c) *Columns*

The minimum diameter $d_{ct,min}$ shall be:

$$d_{ct,min} = \text{Max} \left[8 \text{ mm}; \frac{d_{cl,max}}{4} \right],$$

where $d_{cl,max}$ is the diameter of the largest longitudinal bar.

d) *Walls*

The minimum diameter shall be 8 mm of transverse stirrups, ties and horizontal longitudinal bars.

9.2.2.5 *Maximum spacing of bars*

Unless calculations of crack widths (as per **8.10.2** and **8.4.1.3**) show that a greater spacing is acceptable, the requirements specified hereunder shall apply to flexural members in normal internal or external conditions of exposure.

9.2.2.5.1 *Beams*

a) *Longitudinal reinforcement*

The maximum clear spacing between parallel steel bars or group/pair of bars (near the tension face) shall be that specified in Table 38. The maximum spacing of longitudinal bars (including side face reinforcement) specified in Table 38 is applicable to members exposed to mild and moderate internal environments and not to the members exposed to severe, very severe and extreme environments (specified in **8.5.3.1**), unless in the calculation of design bending moment capacity M_L , the design stress in longitudinal steel bars is limited to 300 MPa in the limit state method and 165 MPa in the serviceability stress method. The values in Table 38 shall be read in conjunction with **7.3.1.4**, when redistribution is considered.

b) *Transverse reinforcement*

The maximum spacing of transverse reinforcement measured along the axis of the member shall not exceed the smaller of $0.75d$ and 300 mm, where d is the effective

depth of beam. The first stirrup in a beam shall be at a maximum distance of 50mm from the face of the column or support.

9.2.2.5.2 Slabs

The horizontal distance between parallel bars (on the tension face) shall not be greater than the smaller of 300 mm or $3d$ in case of main reinforcement bars and 450 mm or $5d$ in case of distribution reinforcement, where d is the effective depth of solid slab.

Table 38 Maximum Clear Spacing between individual Bars or groups of Bars in Beams

[Clause 9.2.2.5.1(a)]

Grade of Steel f_y (MPa)	Redistribution of Moment				
	- 30 %	- 15 %	0 %	+ 15 %	+ 30 %
250	215	260	300	300	300
415	125	155	180	210	235
500	105	130	150	175	195
550	-	120	140	160	-

9.2.2.5.3 Columns

The maximum spacing between adjacent

- longitudinal bars shall be 300mm, and
- transverse hoops shall be $s_{\max} = \text{Min}[300 \text{ mm}; 16d_{lb, \min}; \text{Min}(b, D)]$

where

$d_{lb, \min}$ = diameter of the smallest longitudinal bar;

b = one dimension of the column; and

D = other dimension of the column.

The first hoop in a column shall be at a maximum distance of 50mm from the face of the top of the slab, soffit of the beam or top of the foundation.

9.2.2.5.4 Walls

The maximum spacing between adjacent

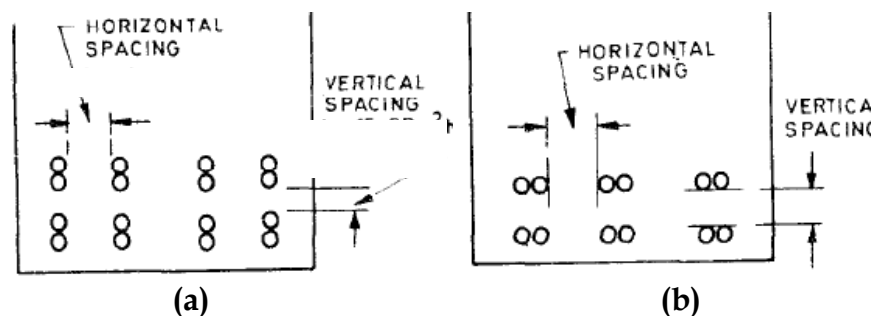
- vertical bars shall be $\text{Min}[300 \text{ mm}; 2t_w]$, and
- horizontal bars shall $\text{Min}[300 \text{ mm}; 2t_w]$,

where t_w is the thickness of the wall.

9.2.2.6 Minimum spacing of bars**9.2.2.6.1 Reinforced concrete members**

In all concrete members (such as slabs, beams, columns, walls, shafts and shells), the minimum spacing of bars shall be as specified hereunder:

- a) The horizontal distance between two parallel main bars in beams, columns and walls shall be not less than the largest of the following:
 - 1) Diameter of a bar, if the diameters are equal,
 - 2) Diameter of the bar of the largest size, if the diameters are unequal, and
 - 3) 5 mm more than the maximum nominal size of coarse aggregate.
- b) The vertical distance between any two layers (see Fig. 25) shall be not less than the largest of the following:
 - 1) Diameter of a bar, if the diameters are equal,
 - 2) Diameter of the bar of the largest size, if the diameters are unequal,
 - 3) 5 mm more than the maximum nominal size of coarse aggregate, and
 - 4) 16 mm.
- c) Where needle vibrators are required to pass through the horizontal bars to vibrate the concrete in the core of the member, then the said distance between the parallel main bars or groups of bars shall be sufficient to enable to pass the vibrator (at the angle needed).



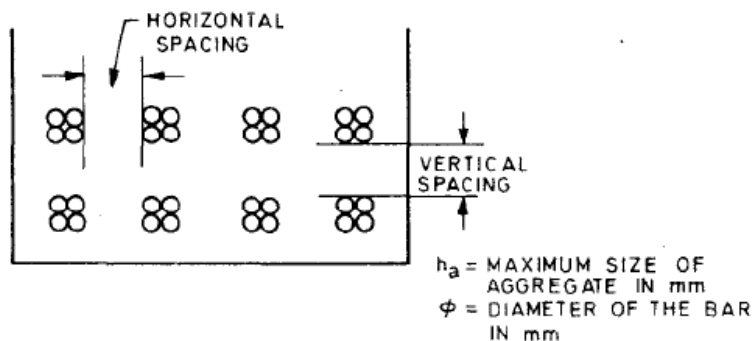


FIG. 25 MINIMUM CLEAR SPACING BETWEEN GROUPS OF BARS: (A) VERTICAL PAIRS, (B) HORIZONTAL PAIRS, AND (C) BUNDLES

9.2.2.6.2 Prestressed concrete members

- a) Grouping of cables or ducts in post-tensioned bonded cables
 - 1) Bundles shall not be permitted of more than two ducts each.
 - 2) If there is possibility of obstruction of flow of concrete or in severe exposure condition, horizontal grouping should be avoided.
 - 3) A pair of ducts placed horizontally or vertically touching each other shall be permitted, only in the straight portion of the cable subject to limitations given below (Fig. 26)
 - i) Two cables can be grouped horizontally provided diameter of each duct is not more than 50 mm.
 - ii) Two cables can be grouped vertically provided diameter of each duct is not more than 110 mm.
 - iii) Two cables shall not be bundled over the curved length of cable in the plane of curvature.
- b) The horizontal and vertical distances between ducts of groups of cables, ducts of cables, or tendons shall not be less than the greater of the following:
 - 1) *Post-Tensioning – Bonded or Unbonded Embedded Cables* (Fig. 26):
 - i) Outer diameter of a duct, between single ducts or between a pair of ducts and the next pair;
 - ii) 50 mm, and
 - iii) Maximum nominal size of coarse aggregate plus 10 mm.

The above shall be applicable for major lengths of the cables. These requirements can be relaxed locally near transition lengths of the cable, provided the minimum clear spacing between the adjacent cables along the line joining the centers of two cables is not less than the smaller of outer diameter of the duct and 50 mm.

- 2) *Post-Tensioning – Cables or Large Bars* (Fig. 26)
 - i) Outer diameter of a cable or bar;
 - ii) 50 mm, and
 - iii) Maximum nominal size of coarse aggregate plus 10 mm.
 - 3) *Pre-Tensioning – Single Wire/Tendon* (Fig. 27)
 - i) Three times the diameter of a wire/tendon;
 - ii) 20 mm, and
 - iii) mm more than the maximum *nominal size of coarse aggregate*.
- c) Cables or ducts may be grouped together vertically in groups of not more than two.

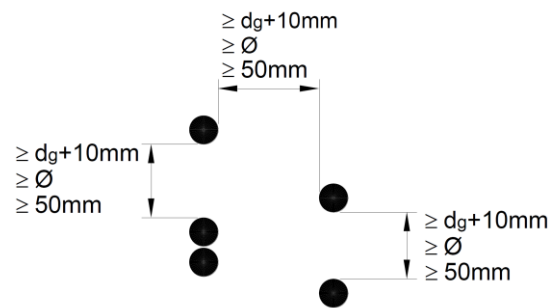
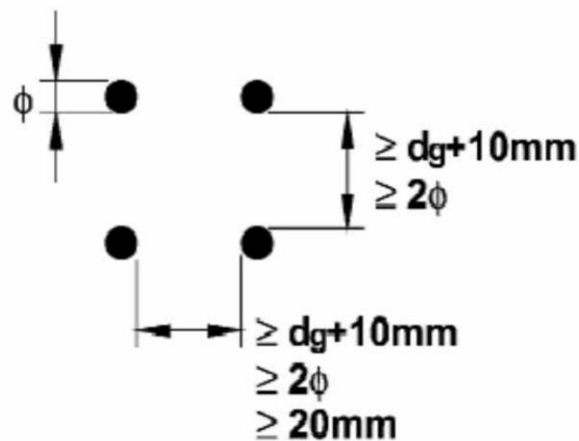


FIG. 26 MINIMUM CLEAR SPACING BETWEEN POST-TENSIONED DUCTS

FIG. 27 MINIMUM CLEAR SPACING BETWEEN PRE-TENSIONED TENDONS
(REDRAW)**9.2.2.7 Development length**

9.2.2.7.1 A bar at any section shall extend on each side of the section by providing the required development length, end anchorage or a combination thereof. The development length L_d shall be estimated based on the tension or compression stress required to be developed at that section, as:

$$L_d = \frac{\phi f_s}{4\tau_{bd}},$$

where

- ϕ = nominal diameter of the bar,
- f_s = design stress in bar at the section considered under the action of any of the design load combinations related to strength criteria specified in Table 3, and
- τ_{bd} = design bond stress as specified in **8.2.6.1(b)**.

9.2.2.7.2 When design load combinations related to serviceability criteria specified in Table 4 are used, the design bond stress specified in **8.4.2.1(b)** shall be used.

9.2.2.7.3 In the estimation of development length:

- a) The anchorage values arising from hooks and bends are included for bars in tension, and excluded for bars in compression; and
- b) The bent length of bars is included for bars in tension, and only the projected length of bar for bars in compression (Fig. 28).

9.2.2.7.4 The development length is the same for bars of members being designed by the limit state method and for bars of members being checked for meeting serviceability requirements by the allowable stress method.

9.2.2.7.5 The development length of each bar in a set of bundled bars shall be that for the individual bar increased by:

- a) 10 percent when in contact with one other bar,
- b) 20 percent when in contact with two other bars, and
- c) 33 percent when in contact with three other bars.

9.2.2.7.6 When more steel is provided than that required by design calculations, then the development length L_d estimated by the above expression can be reduced further. The reduced development length L_{dr} shall be estimated as:

$$L_{dr} = \left(\frac{A_{s,required}}{A_{s,provided}} \right) L_d.$$

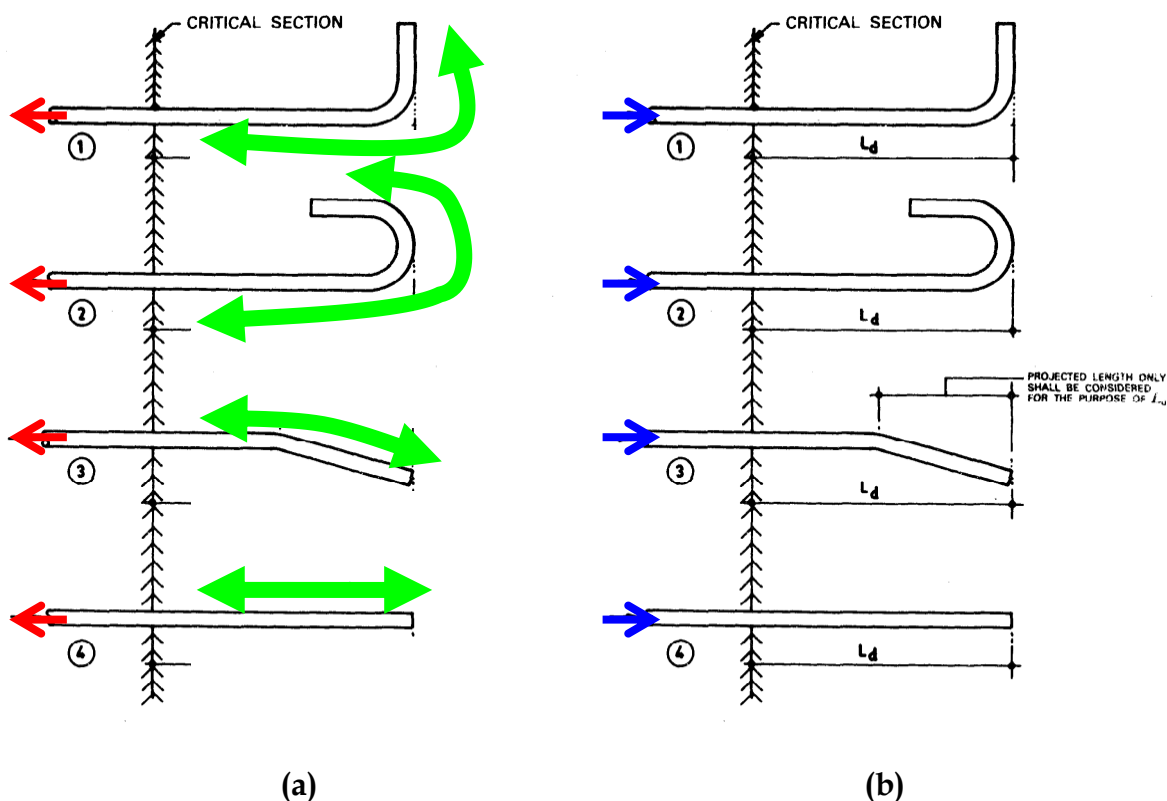


FIG. 28 DEVELOPMENT LENGTH OF BARS IN: (A) TENSION, AND (B) COMPRESSION

9.2.2.8 Anchorage of bars

The anchorage length of straight bar in tension or compression shall be the development length of the bar in tension or compression, respectively. When straight lengths are not available to accommodate the required development length within the member, then in bars subjected to:

- a) tension, the use of standard bends and standard hooks (conforming to IS 2502) and straight lengths shall be permissible. And, the anchorage values to be used to estimate the available development length shall be taken as (Fig. 29):
 - 1) $4\phi_b$ for each 45° bend, subject to a maximum of $16\phi_b$, and
 - 2) $16\phi_b$ for a standard U-type hook; and
- b) compression, the use of standard bends and standard hooks (conforming to IS 2502) and straight lengths shall be taken advantage of only as much as the projected length of the hooks, bends and straight lengths from the start of the bend, along the direction of the compression force.

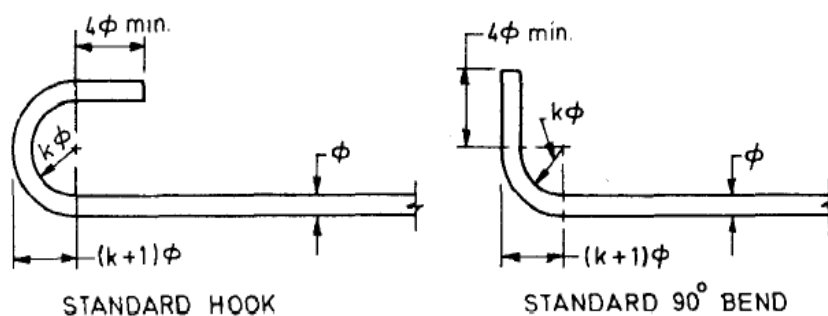


FIG. 29 DEFINITIONS OF STANDARD HOOK AND STANDARD BEND

9.2.2.8.1 Anchorage of mechanical devices

Mechanical or other devices conforming to IS 16712 capable of developing the strength of the bar without damage to concrete may be used as anchorage with the approval of the engineering-charge. At least the minimum straight length shall be provided before mechanical device as specified in IS 16712.

9.2.2.8.2 Anchorage in shear reinforcement

Notwithstanding any provisions of this standard, in transverse reinforcement (such as stirrups and transverse ties), complete development lengths and anchorage shall be deemed to have been provided when the bar is bent around a bar of at least its own diameter ϕ_b through an angle of at least 90° , 135° and 180° and is continued beyond the end of the curve for a length of at least $8\phi_b$, $6\phi_b$ and $4\phi_b$, respectively, as specified in IS 2502.

9.2.2.8.3 Bearing stresses at bends

The bearing stress f_b in concrete inside bends and hooks complying with IS 2502 need not be checked. Otherwise, the bearing stress inside a bend shall be estimated as:

$$f_b = \frac{F_{bt}}{r\phi_b},$$

where

F_{bt} = tensile force due to design loads in the bar or group of bars,

r = internal radius of the bend, and

ϕ_b = size of the bar, or in bundled bars, size of the equivalent area.

and, f_b so estimated shall be limited to:

$$f_b < \begin{cases} \frac{1.5f_{ck}}{1 + 2\left(\frac{\phi_b}{a}\right)} & \text{For Limit State Method} \\ \frac{f_{ck}}{1 + 2\left(\frac{\phi_b}{a}\right)} & \text{For Working Stress Method} \end{cases}$$

where

- f_{ck} = Characteristic Strength of concrete considered in design, and
 a = Center to center distance between bars or groups of bars perpendicular to the plane of the bending, for a particular bar or group of bars in contact
= Clear cover plus size of bar, for a bar or group of bars adjacent to the face of the member

9.2.2.8.4 Other requirements

Further,

- a) If a change in direction of tension or compression reinforcement induces a resultant force acting outward tending to split the concrete, such force should be taken up by additional links or stirrups. Additional requirements are specified in Liquid retaining structures in IS 3370.
- b) Bent tension bar at a re-entrant angle should be avoided.

9.2.2.9 Splicing of bars

Where splices are provided in the reinforcing bars, they shall be away (as far as possible) from the sections of maximum stress and be staggered. Splices in flexural members shall not be at sections, where the bending moment demand M_D is more than 50 percent of the limiting moment capacity M_L .

When bars are required to be spliced:

a) Lap Splicing

- 1) Shall not be used in concrete members carrying axial tension,
- 2) May be used in concrete members in flexural tension,
- 3) May be used in concrete members in axial or flexural compression, and
- 4) Shall be avoided to the extent possible of longitudinal bars of diameters of 16mm and higher in flexural members. Mechanical couplers complying with the requirements of IS 16712 shall be used instead;

- 5) When used, the member shall be provided with closed loop transverse hoops along the full length of lapping of bars in tension and compression, with:
 - i) spacing at not more than 150 mm, and
 - ii) length over which these transverse hoops are provided is at least $L_{dt}/3$ or $L_{dc}/3$ from the two ends of splice length (Fig. 30);
- b) Mechanical coupling shall be adopted preferably for bars of diameters 25 mm or more. Mechanical couplers shall comply with the requirements of IS 16712 shall be used instead; and
- c) Welding of bars shall not be permitted. In exceptional cases, where welding is inevitable, the design stress in reinforcing steel shall be limited to $0.3f_y$.

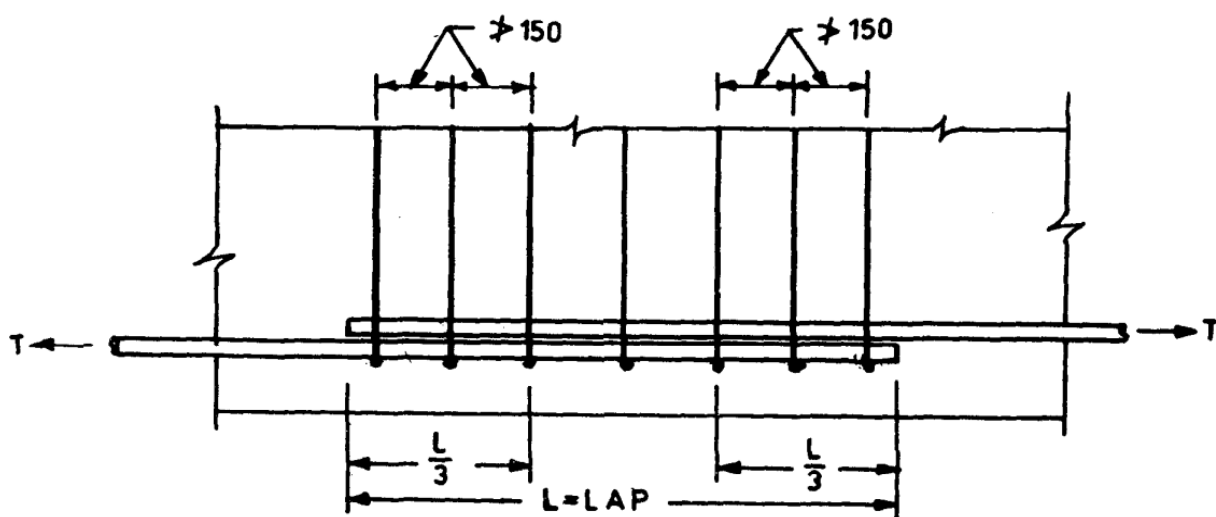


FIG. 30 CLOSED LOOP TRANSVERSE HOOPS ALONG THE FULL LENGTH OF LAPPING OF BARS IN TENSION AND COMPRESSION

9.2.2.9.1 Lap splicing

When it is inevitable to lap splice the bars of diameter less than 25mm, the provisions of the clause shall be applicable, but not to longitudinal bars in compression, when the compression bar is not enclosed within transverse ties that have closed loops with 135° hook ends; and transverse bars.

a) Longitudinal bars in tension

Where longitudinal bars in tension are spliced in flexural members, the lap splices shall be:

- 1) of length:
 - i) $L_{lap} = \text{Max}[L_{dt}; 30d_b]$, where L_{dt} is the development length in tension and includes the anchorage values of hooks and bends, and d_b the diameter of the bar being lapped, and have a straight length not less than $\text{Max}[200\text{mm}; 15d_b]$; and
 - ii) $1.4L_{lap}$, if the bar is located at the top of a section where the nominal cover is less than $2d_b$, or the corner of a section where the nominal cover to either face is less than $2d_b$ or where clear distance between adjacent laps is less than $L_{lap} = \text{Max}[75\text{mm}; 6d_b]$; and
 - iii) $2L_{lap}$, if both (i) and (ii) above apply.
- 2) As far away as possible from the sections of maximum bending moment, and enclosed within closely-spaced closed-hoop transverse reinforcement complying with the requirements of confinement steel in IS 13920 (Part 2/Sect 3).
- 3) Mandatorily in beams and preferably in columns such that not more than half the bars are spliced at a section; they shall be considered as staggered if the center to center distance of the splices is not less than $1.3L_{lap}$, where L_{lap} is calculated as in **82.2.8(a)(1)**. If more than half the bars are spliced at a section, the lap length shall be increased by 50 percent of L_{lap} .
- 4) Such that their lap lengths are calculated on the basis of diameter of the smaller bar, when bars of two different diameters are spliced; and
- 5) Made by splicing one bar at a time, when bundled bars are spliced; such individual splices of bars within a bundle shall be staggered; they shall be considered as staggered if the center to center distance of the splices is not less than $1.3L_{lap}$, where L_{lap} is calculated as in **82.2.8(a)(1)**.

b) Longitudinal bars in compression

Where longitudinal bars in compression in flexural members are spliced, the lap splices shall be spliced, the lap splices shall be of length $L_{lap} = \text{Max}[L_{dc}; 30d_b]$ where L_{dc} is the development length in compression (and it includes the anchorage values of hooks and bends) and d_b the diameter of the bar being lapped, and have a straight length not less than $\text{Max}[200\text{mm}; 20d_b]$.

c) Further,

- 1) Lap length including anchorage value of hooks for bars in flexural tension shall be L_d or $30d_b$, whichever is larger, and for direct tension shall be $2L_d$ or $45d_b$, whichever is larger. The straight length of the lap shall not be less than $20d_b$ or 200 mm.
- 2) Where lap occurs for a tension bar located at:
 - i) top of a section as cast and the minimum cover is less than twice the diameter of the lapped bar, the lap length shall be increased by a factor of 1.4.
 - ii) corner of a section and the minimum cover to either face is less than twice the diameter of the lapped bar or where the clear distance between adjacent laps is less than 75 mm or 6 times diameter of lapped bar, whichever is greater, the lap length should be increased by a factor of 1.4.
 - iii) where both conditions (i) and (ii) apply, the lap length should be increased by a factor of 2.0.
- 3) When splicing welded wire fabric (conforming to IS 4948), lap splices of fabric shall be made so that overlap measured between the extreme cross wires shall be not less than the spacing of cross wires plus 100 mm.

9.2.2.9.2 Mechanical coupling

When mechanical couplers are used to splice reinforcement bars, the provisions below shall be applicable:

- a) The couplers shall conform to IS 16712, and
- b) Not more than half the bars are spliced at a section.

9.2.2.10 Curtailment of bars

Curtailment of bars shall be permitted, provided:

- a) Steel reinforcing bars shall extend beyond the point at which it is no longer required to resist flexure (where the limiting moment capacity of the section considering only the continuing bars is equal to the design moment demand) at least for a distance at least equal to the larger of d and $12d_b$, except at simple support or end of cantilever. In addition, (b) also shall be satisfied.
- b) Flexural reinforcement shall not be terminated in a tension zone unless any one of the following conditions is satisfied:
 - 1) The shear force demand at the cut-off point does not exceed that the design shear strength specified in 8.2.6.1(d) corresponding to the balance continuing steel, including the shear strength of web reinforcement provided.
 - 2) Stirrup area in excess of that required for shear and torsion is provided along each terminated bar over a distance from the cut-off point equal to $0.75d$ of the

member. The excess stirrup area shall be not less than that specified in **8.2.4.1(b)(1)**. The resulting spacing shall not exceed $d/(8\beta_a)$, where β_a is the ratio of the area of bars cut-off to the total area of bars at the section, and d the effective depth.

- 3) The continuing bars provide double the area required for flexure at the cut-off point and the shear does not exceed 75 percent of that permitted.

9.2.2.10.1 Positive moment reinforcement

- a) At least 33 percent of positive moment reinforcement in simply supported members and 25 percent of positive moment reinforcement in continuous members shall extend along the same face of the member into the support, to a length equal to $L_d/3$.
- b) When a flexural member is part of a continuous beam or moment frame system, the positive reinforcement required to be extended into the support shall be anchored to develop its design stress in tension at the face of the support.
- c) At supports of simply supported beams and at points of inflection in continuous members, the diameter of the positive moment (tension) reinforcement shall be such that L_d computed by **8.2.2.6** considering $f_{d,steel}$ to be $0.87f_y$, does not exceed:

$$\frac{M_L}{V_L} + L_0, \text{ when end of reinforcement sustains tensile action, and}$$

$$\frac{1.5M_L}{V_L} + L_0, \text{ when end of reinforcement is confined by compressive action,}$$

where

- M_L = Limiting Moment of the section assuming all reinforcement at the section to be stressed to design stress in *Limit State Design* and the permissible stress in *Allowable Stress Design*;
- V_L = Design Shear Force at the section; and
- L_0 = Sum of anchorage beyond the centre of the support and equivalent anchorage of any hook or mechanical anchorage at simple support; and
- = $\text{Max}[d; 20d_b]$ at the point of inflection.

9.2.2.10.2 Negative moment reinforcement

At least one-third of the total reinforcement provided for negative moment at the support shall extend beyond the point of inflection for a distance not less than $\text{Max}[d; 20d_b; L_c/16]$.

9.2.2.10.3 Curtailment of bundled bars

Bars in a bundle shall terminate at different points spaced apart by not less than $40d_b$, except for bundles stopping at a support.

9.2.2.11 *Cover to reinforcing bars and prestressing tendons*

The cover is the clear thickness of concrete provided from the nearest surface of concrete of the member to either outermost surface of the reinforcing steel bar or the pre-stressing tendon/duct. Nominal cover shall be taken as the largest of the requirements based on:

- a) Strength considerations as per **8.2.2.10**,
- b) Durability considerations as per **8.5.5.1(a)**, and
- c) Fire resistance considerations as per **8.6.1.1**.

In any case, minimum values for member type shall be specified in Table 38.

The nominal cover requirements specified hereunder are applicable to cast in-situ reinforced and prestressed concrete structures, including precast constructions. Lesser thickness than those specified herein may be permissible for precast construction, but such relief shall be provided by the respective standard but with due considerations of strength, durability and fire resistance as specified in this standard.

a) Reinforcing steel

- 1) The minimum nominal cover to surface of all reinforcement bars (including stirrups) and group of bars in contact, for safe transmission of tension force through bond to concrete shall be taken as per Table 39 in both reinforced and prestressed concrete members. This nominal cover shall not be less than the diameter of the longitudinal bar with largest diameter, or the equivalent diameter of the group of the longitudinal bars in contact. In members made of precast concrete or high performance concrete, the nominal cover can be reduced up to a maximum of 5 mm.
- 2) Also, the minimum nominal cover shall in no case be less than those specified in **8.5.5.1(a)** and **8.6.1.1** from the requirements of durability and fire considerations, respectively. These values of cover are applicable for normal weight aggregate concrete.

Table 39 Minimum cover to reinforcing bars for safe transmission of bond forces in concrete members
[Clause 9.2.2.11(a)]

<i>Minimum Cover (mm)</i>					
<i>Slabs</i>	<i>Beams</i>	<i>Waffle slab ribs</i>	<i>Columns</i>	<i>Walls</i>	<i>Footings Piles</i>
Concrete Members with Concretes other than High Performance Concrete					
20	25	30	40 Longitudinal Bar	35	50
Concrete Members with Precast Concrete or High Performance Concrete					
20	20	25	35 Longitudinal Bar	30	45

b) *Prestressing Steel*

The minimum cover requirement for unbonded tendons for desired fire resistance shall be more than that for bonded tendons. The minimum clear cover to prestressing steel used in bonded tendons measured along any direction from surface of tendon or sheathing duct closest to the surface shall be the larger of the following:

- 1) From strength considerations in bonded tendons,
 - i) Diameter of largest tendon or sheathing duct,
 - ii) Nominal aggregate size plus 10 mm
- 2) From strength considerations in unbonded tendons,
 - i) 75 mm in beams, and
 - ii) 45 mm in slabs.
- 3) From durability considerations
 - i) Diameter of largest tendon or sheathing duct, and
 - ii) Values specified in **8.5.5.1(a)**.
- 4) From fire resistance considerations
 - i) Diameter of largest tendon or sheathing duct, and
 - ii) Values specified in **8.6.1.1**.

A maximum of two unbonded strands shall be grouped in beams. Unbonded strands shall not be grouped in slabs.

9.2.3 *Relative Dimensions*

Concrete structures shall be proportioned such that they comply with the requirements specified hereunder at the level of members and sections.

9.2.3.1 Maximum slenderness of members**a) Slabs**

The maximum span to depth ratios l_e/d of slabs shall be as specified in Table 40, where l_e is the effective span (in m) of the slab panel in the direction being considered of bending, and d is the effective depth of the slab in the direction being considered of bending. These values are applicable only for solid slabs. The requirements for flat slabs are provided in **8.3.2.1(d)**. The values given in Table 40 shall be divided with the following factors:

- 1) μ_T based on area of bottom tension steel at mid-span and grade of steel, given by:

$$\mu_T = \frac{2}{1 + \left(\frac{f_y}{415} \right) \left(\frac{A_{st,provided}}{A_{st,required}} \right)},$$

- 2) μ_C based on area of compression steel, when provided, given by:

$$\mu_C = 1 + 0.7p_C,$$

- 3) μ_F based on area of effective width of flange of the member, given by:

$$\mu_F = \begin{cases} 1.0 & 0.00 < \frac{b_w}{b_{f,eff}} \leq 0.25 \\ 1.0 + \frac{1}{3} \left(\frac{b_w}{b_{f,eff}} - 0.25 \right) & 0.25 < \frac{b_w}{b_{f,eff}} \leq 1.00 \end{cases}, \text{ and}$$

- 4) μ_G based on aspect ratio of the slab, given by:

$$\mu_G = \frac{2}{1 + \beta},$$

where

$A_{st,provided}$ = area of tension steel provided in the member,

$A_{st,required}$ = area of tension steel required from strength considerations in the member,

b_w = breadth of the web of the member,

- $b_{f,eff}$ = breadth of the effective flange width of the member,
- p_c = percentage of steel (expressed as a fraction) provided on compression face of the member, and
- β = ratio of the larger and smaller sizes of the slab panel in plan, more than 2.

Table 40 Maximum span to depth ratio l_e/d of Slabs with two-way slab action

[Clause 9.2.3.1(a)]

Sl No.	Boundary Condition of One-Way Slabs	Maximum Span to Depth Ratio l_e/d (l_e is smaller of the two plan dimensions of the slab)
(1)	(2)	(3)
1	Continuous at both ends	32
2	Continuous at one end	29
3	Simply supported slabs	26
4	Cantilever	7

b) Beams

The maximum span to depth ratios l_e/d of beams shall be considered as specified in **Table 41**, where l_e is the effective span of the slab panel in the direction being considered of bending, and d the effective depth of the slab in the direction being considered of bending.

In a simply supported or continuous beam, the maximum clear distance $l_{clear,max}$ between the lateral restraints (to compression faces) shall be:

$$l_{clear,max} = \text{Min} \left[60b; 250 \frac{b^2}{d} \right],$$

where

b = Breadth of the compression face midway between the lateral restraints of the beam; and

d = Effective depth of the beam.

Table 41 Maximum span to depth ratio l_e/d of beams

[Clause 9.2.3.1(a)]

S.No.	Boundary Condition of Beam	Maximum Span to Depth Ratio l_e/d	
		l_e (m) $\leq$ 10m	l_e (m) $>$ 10m
(1)	(2)	(3)	(4)
1	Continuous at both ends	20	$20 \left(\frac{10}{l_e} \right)$
2	Continuous at one end	18	$18 \left(\frac{10}{l_e} \right)$
3	Simply supported slabs	16	$16 \left(\frac{10}{l_e} \right)$
4	Cantilever	7	$7 \left(\frac{10}{l_e} \right)$

In a cantilever beam, the maximum clear distance $l_{clear,max}$ from the free end of the cantilever to the lateral restraint (to compression faces) shall be:

$$l_{clear,max} = \text{Min} \left[25b; 100 \frac{b^2}{d} \right],$$

where

b = breadth of the compression face midway between the lateral restraint and the free end of the beam; and

d = effective depth of the beam.

The values given in Table 41 shall be multiplied with the following factors:

- 1) μ_T based on area of bottom tension steel at mid-span and grade of steel, given by:

$$\mu_T = \frac{\alpha}{1 + \sqrt{2 + \left(\frac{f_y}{415} \right) \left(\frac{A_{st,provided}}{A_{st,required}} \right)}},$$

- 2) μ_C based on area of compression steel, given by:

$$\mu_C = 1 + 0.7p_C, \text{ subject to } p_C \leq 2, \text{ and}$$

3) μ_F based on area of effective width of flange of the member, given by:

$$\mu_F = \begin{cases} 1.0 & 0.00 < \frac{b_w}{b_{f,eff}} \leq 0.25 \\ 1.0 + \frac{1}{3} \left(\frac{b_w}{b_{f,eff}} - 0.25 \right) & 0.25 < \frac{b_w}{b_{f,eff}} \leq 1.00 \end{cases}, \text{ and}$$

4) μ_G based on aspect ratio of the beam, given by:

$$\mu_G = \frac{2}{3},$$

where

$$\alpha = \begin{cases} 2.6 & Fe415 \\ 2.0 & Fe500 \\ 1.6 & Fe550' \\ 1.2 & Fe600 \end{cases}$$

$A_{st,provided}$ = area of tension steel provided in the member,

$A_{st,required}$ = area of tension steel required from strength consideration in the member,

b_w = breadth of the web of the member,

$b_{f,eff}$ = breadth of the effective flange width of the member, and

p_C = percentage of steel (expressed as a fraction) provided on compression face of the member.

c) Columns

For the purposes of structural design, a compression member shall be treated based on its slenderness ratios about the two axes of bending as:

a) *Pedestal*, if $\frac{l_{ex}}{D} \leq 3$ and $\frac{l_{ey}}{b} \leq 3$,

b) *Short Column*, if $3 < \frac{l_{ex}}{D} \leq 12$ and $3 < \frac{l_{ey}}{b} \leq 12$, and

c) *Long Column*, if $\frac{l_{ex}}{D} > 12$ or $\frac{l_{ey}}{b} > 12$,

where

b = width of the cross-section of the column,

D = depth of the cross-section of the column,

l_{ex} = effective length for bending about an axis perpendicular to the dimension D , and

l_{ey} = effective length for bending about an axis perpendicular to the dimension b .

In a column laterally supported at both ends, the maximum clear distance $l_{clear,max}$ between the lateral restraints shall be:

$$l_{clear,max} = \text{Min} \left[60 \text{Min}(b; D); 250 \frac{b^2}{D} \right],$$

where

b = breadth of cross-section of the column perpendicular to the plane of bending, and

D = depth of cross-section of the column in the plane of bending.

In a cantilever column, the maximum clear distance $l_{clear,max}$ between the lateral restraint and the free end of the column shall be:

$$l_{clear,max} = \text{Min} \left[25 \text{Min}(b; D); 100 \frac{b^2}{D} \right],$$

d) *Structural Walls*

The H_{we}/t_w ratio of structural walls (in sectional elevation) shall not exceed 30, where H_{we} is the *effective height* and t_w the overall thickness of the wall web.

Structural walls shall be classified as:

Extreme Squat Structural Walls if $0 < H_w/L_w \leq 0.5$;

Squat Structural Walls	if $0.5 < H_w/L_w \leq 1.0$;
Intermediate Structural Walls	if $1.0 < H_w/L_w \leq 2.0$, and
Slender Structural Walls	if $2.0 < H_w/L_w \leq 7.0$.

Structural walls shall not have elevation aspect ratio H_w/L_w more than 7.0.

9.2.3.2 *Cross-sectional aspect ratio**a) Beams*

The maximum D/b ratio of the cross-section of a beam without a compression flange shall be 2.5.

b) Columns

The maximum D/b ratio of a column shall be 2.5. Columns with D/b ratio is in the range 2.5 to 4.0 are discouraged. When used under compulsion, they shall be designed and detailed as columns, and not as structural walls.

c) Structural Walls

The minimum L_w/t_w ratio of a structural wall shall be $4 < (L_w/t_w) < 20$.

9.2.3.3 *Distinction between columns and structural walls*

Notwithstanding, the provisions of **8.2.3.1** and **8.2.3.2**, a vertical member shall be designed as a structural wall, if it has the bending moment diagram about its major axis along its full height like that of a vertical cantilever in single curvature along its full height [Fig. 31(a)], or at best one point of contra-flexure owing to frame-wall interaction (especially at the roof level), even though with small jumps at the levels where the beams and slabs frame into it [Fig. 31(c)]. The bending moment at the roof level need not be zero, owing to the presence of beams at that level.

Else, the vertical member shall be designed as column, if it is in double curvature along the whole or part of its height [Fig. 31(b)].

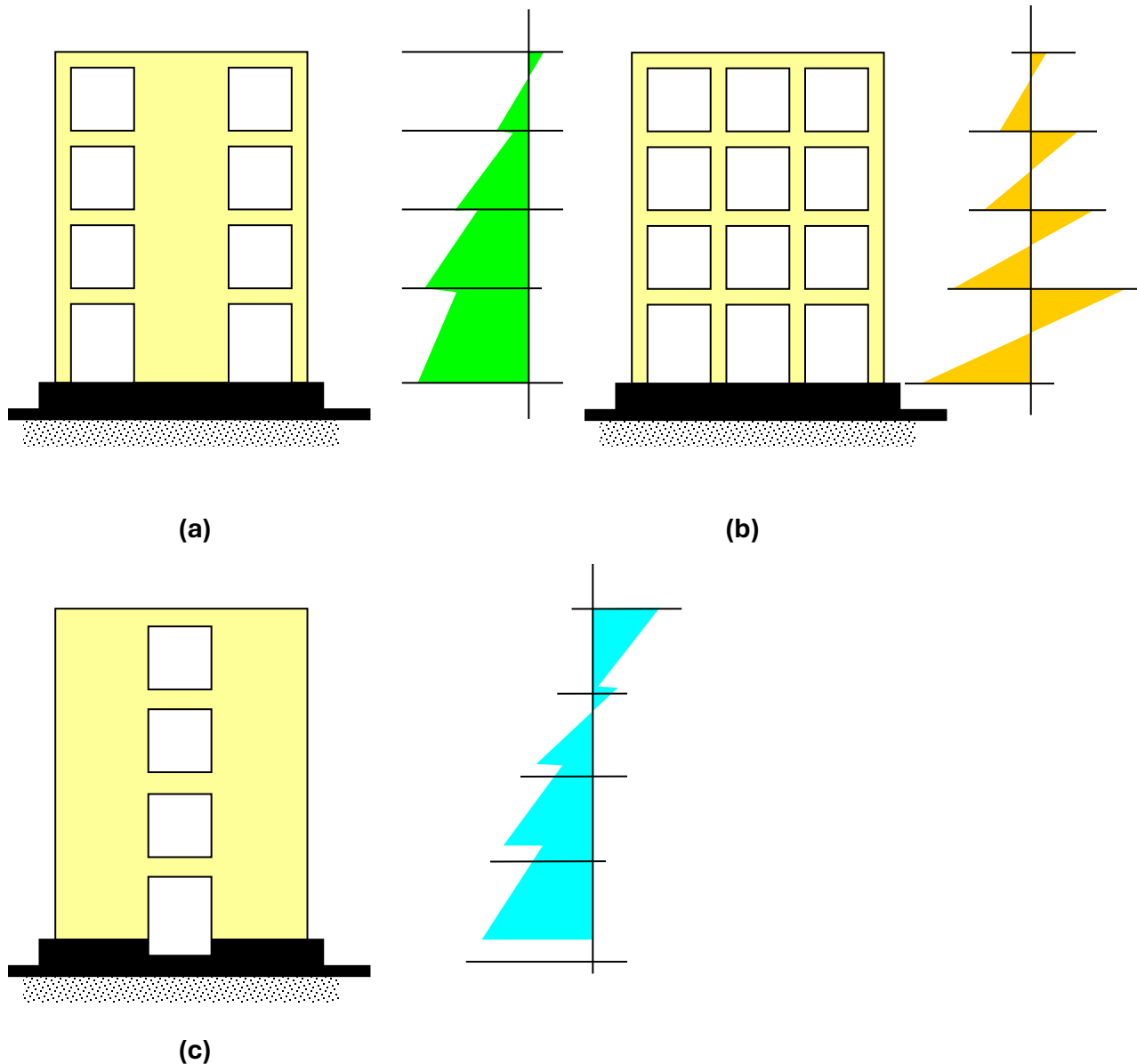


FIG. 31 BENDING MOMENT DIAGRAMS OF VERTICAL MEMBERS: (A) FRAME COLUMN, (B) STRUCTURAL WALL, AND (C) COUPLED STRUCTURAL WALL

9.2.4 Amount of Reinforcements

The minimum and maximum amounts of reinforcements in concrete structures shall be taken as **9.2.4.1** and **9.2.4.2**, respectively. Longitudinal reinforcement bars shall be provided to carry the axial force and bending moments imposed on the members. The transverse reinforcement bars shall be provided to carry shear forces imposed on the members, and to hold the longitudinal bars without buckling under its design levels

loads. These transverse reinforcement shall be considered to provide effective lateral support is given when they are in the form of either circular rings capable of taking circumferential tension or by polygonal links (lateral ties) with internal angles not exceeding 135°.

Bars may be arranged singly, or in pairs in contact, or in groups of three or four bars tied together in contact. The cross-sections where bars are bundled shall be enclosed within closed stirrups. Also, bundled bars shall be tied together to ensure the bars remaining together during concreting. Bars larger than 32 mm diameter shall not be bundled, except in columns.

The recommendations for earthquake-resistant design and detailing of concrete structures in earthquake zones III, IV, V and VI shall be as per IS 13920 (Parts 1 and 2: Sec 3).

9.2.4.1 Minimum Reinforcement

Concrete members shall be provided with at least the amount of longitudinal and transverse reinforcements specified hereunder.

a) Longitudinal reinforcement

Concrete members shall be provided with at least the amount of longitudinal reinforcement specified hereunder.

The minimum percentage of longitudinal steel in vertical members, namely columns and structural walls, shall be as per **9.2.4.1(a)(3)** as in a column, if $\beta < 0.05$, and as per **9.2.4.1(a)(4)** as in a wall, if $\beta \geq 0.05$, where β is the ratio of the pure flexural lateral translational stiffness and pure shear lateral translational stiffness, given by:

$$\beta = \frac{\left(\frac{12EI}{L^3} \right)}{\left(\frac{GA_s}{L} \right)}.$$

1) Slabs

The minimum area $A_{st,min}$ (in mm²) on the tension face in solid slabs shall be:

$$A_{st,min} = 1000D \begin{cases} 0.0015 & \text{Fe250 grade steel} \\ 0.0012 & \text{Fe415 and higher grade steel} \\ 0.0012 & \text{For welded steel mesh} \end{cases}$$

$\leq 250 \text{ mm}^2$ on one face

$> \left(\frac{2}{3}\right)250 \text{ mm}^2$ on tension face, and $\left(\frac{1}{3}\right)250 \text{ mm}^2$ on compression face,

where

D = total depth of the slab (in mm).

The minimum area $A_{st,min}$ (in mm^2) on the tension in flat slabs shall be:

$$\frac{A_{st,min}}{1000D} = 0.0018$$

In slabs on grade, the minimum reinforcement of 0.12 percent shall be provided on the tension face and 360 mm^2 on the other face.

2) Beams

The minimum area $A_{st,min}$ of longitudinal tensile bars in a beam or of the untensioned longitudinal tensile bars in a PSC beam shall be given by:

$$\frac{A_{st,min}}{bd} = \frac{0.25\sqrt{f_{ck}}}{f_y},$$

where

b = breadth of the beam parallel to the axis of bending,

d = effective depth of the beam perpendicular to the axis of bending,
and

f_{ck} = characteristic strength (MPa) of concrete, and

f_y = characteristic strength (MPa) of reinforcing steel.

In all slabs, beams and footings, where the depth of the web in a beam exceeds 750 mm, the side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not be less than 0.1 percent of the web area and shall be distributed equally on two faces at a spacing not exceeding 300 mm or web thickness whichever is less.

3) Columns

The minimum area $A_{s,min}$ of longitudinal bars in a column (on all faces together) shall be given by:

$$\frac{A_{s,min}}{bD} = \begin{cases} 0.008 & \text{Concretes of grades M20 – M35} \\ 0.010 & \text{Concretes of grades M40 – M55} \\ 0.012 & \text{Concretes of grades M60 – M75} \end{cases}$$

When closed loop transverse ties are provided in a column as transverse reinforcement, the minimum number of longitudinal bars shall be four in rectangular columns, and six in circular columns.

In pedestals of columns in which the longitudinal bars is not taken in account in strength calculations, the area of longitudinal bars shall not be less than 0.15 percent of the cross-sectional area provided.

When a column is provided with a cross-sectional area larger than that required to carry the loads, then the minimum area of longitudinal bars shall be calculated based on the area of concrete required to carry the loads and not on the actual area provided, subject to the value not being less than 75 percent of the $A_{s,min}$ values given above.

4) Walls

Concrete structural walls meant to carry axial force , shear force and bending moment shall be provided with a curtain of uniformly spaced longitudinal (vertical) bars on each face. Concrete Walls with only one curtain of vertical bars in the entire cross-section shall not be considered as structural walls.

In structural walls (such as those with thickness is 200 mm or more), the vertical bars shall be provided in two curtains, that is one on each face of the wall. The minimum area $A_{st,min}$ (mm²) of longitudinal (vertical) bars on both faces together shall be:

$$A_{st,min} = 1000t_W \left(\begin{cases} 0.005 & \text{Fe250grade steel} \\ 0.004 & \text{Fe415 and higher grade steel} \\ 0.004 & \text{For welded mesh (of bar diameter less than 16mm)} \end{cases} \right)$$

where t_W is the thickness of the wall in mm.

5) *Footings*

In footings of thickness:

- i) less than 750 mm, at least one curtain of reinforcement (along each plan direction) shall be provided on the tension face of the footing, and
- ii) 750 mm or more, at least in two curtains of reinforcement (along each plan direction) shall be provided one on the tension and the other compression faces of a minimum of 0.12 percent each, and at least 360 mm² of side face reinforcement.

The minimum area $A_{st,min}$ of longitudinal bars (in mm²) in each curtain of reinforcement in a footing shall be:

$$A_{st,min} = 1000t_F \left\{ \begin{array}{ll} 0.0015 & \text{Fe250 grade steel} \\ 0.0012 & \text{Fe415 and higher grade steel} \\ 0.0012 & \text{For welded mesh (of bar diameter less than 16mm)} \end{array} \right\} \geq 360 \text{ mm}^2$$

,

where t_F is the thickness of the footing (in mm).

6) *Raft foundations*

In rafts, at least in two curtains of reinforcement shall be provided one along each plan direction, on both the top and bottom face. The minimum area $A_{st,min}$ of longitudinal (horizontal) bars (in mm²) in each curtain of reinforcement in a raft shall be:

$$\frac{A_{st,min}}{1000D} = 0.0012,$$

where D is the thickness of the raft in mm.

b) *Transverse reinforcement*

The amount and spacing of transverse reinforcement in beams shall be governed by considerations of design for shear force and torsion moment. Notwithstanding the

above, concrete members shall be provided with at least the amount of transverse bars specified hereunder.

1) Beams

The minimum area $A_{sv,min}$ of transverse reinforcement of all legs together in a RC beam shall be:

$$A_{sv,min} = \frac{\tau_{Lc} b d}{0.87 f_y \left(\frac{d}{s_v} \right)_{INT}},$$

where

τ_{Lc} = Shear strength (MPa) of concrete corresponding to 0.8 percent of longitudinal steel

$$= \begin{cases} 0.40 + 0.005(f_{ck} - 25) & \text{M25 - M35} \\ 0.47 + 0.005(f_{ck} - 40) & \text{M40 - M55} \\ 0.54 + 0.003(f_{ck} - 60) & \text{M60 - M70} \end{cases}$$

b = Breadth (mm) of the beam along the axis of bending,

d = Effective depth (mm) of the beam in the plane of bending,

s_v = Spacing (mm) of the transverse stirrups,

f_y = Characteristic strength (MPa) of reinforcing steel, which shall not be more than 415 MPa for the purpose of calculation of $A_{sv,min}$; and

$$\left(\frac{d}{s_v} \right)_{INT} = \text{Integral part of the number.}$$

The minimum area $A_{sv,min}$ (mm²) of vertical transverse reinforcement per unit length (in plan) in a prestressed concrete beam shall be as below:

i) *Members subjected to dynamic loading:*

$$A_{sv,min} = 1000b_w \left(\left\{ \begin{cases} 0.0030 & D_w > 4t_w \\ 0.0020 & D_w \leq 4t_w \end{cases} \text{ Fe415} \right. \right. \left. \left\{ \begin{cases} 0.0020 & D_w > 4t_w \\ 0.0015 & D_w \leq 4t_w \end{cases} \text{ Others} \right. \right), \text{ and}$$

ii) *Members not subjected to dynamic loading:*

$$A_{sv,min} = 1000b_w (0.0010),$$

where

b_w = Breadth (mm) of the beam web along the axis of bending, and
 D_w = Depth (mm) of the beam web normal to the axis of bending.

2) Columns

The minimum area $A_{sv,min}$ (mm²) of transverse reinforcement in a column shall be the largest of the following three requirements:

i) Shear force condition

$$A_{sv,min} = \frac{\tau_{Lc} b d}{0.87 f_y \left(\frac{d}{s_v} \right)_{INT}},$$

where

τ_{Lc} = shear strength (MPa) of concrete corresponding to 0.8 percent of longitudinal steel

$$= \begin{cases} 0.40 + 0.005(f_{ck} - 25) & \text{M25 - M35} \\ 0.47 + 0.005(f_{ck} - 40) & \text{M40 - M55} \\ 0.54 + 0.003(f_{ck} - 60) & \text{M60 - M70} \end{cases}$$

b = breadth (mm) of the column in the plane perpendicular to the plane of bending,

d = effective depth (mm) of the column in the plane of bending,

s_v = spacing (mm) of the transverse ties,

f_y = characteristic strength (MPa) of reinforcing steel, which shall not be more than 415 MPa for the purpose of calculation of $A_{sv,min}$

; and

$$\left(\frac{d}{s_v} \right)_{INT} = \text{Integral part of the number.}$$

ii) Axial force condition

$$\frac{A_{s,min}}{bD} = \frac{0.45 f_{ck}}{\varepsilon_{cp} E_s} \left\{ 1 - \left(1 - \frac{2b'}{b} \right) \left(1 - \frac{2d'}{D} \right) \right\},$$

where

b = breadth (mm) of the column parallel to the axis of bending,

D = depth (mm) of the column perpendicular to the axis of bending,

b' = nominal cover (mm) to column bars parallel to the axis of bending,

d' = nominal cover (mm) to column bars perpendicular to the axis of bending,

f_{ck} = characteristic strength (MPa) of concrete,

f_y = characteristic strength (MPa) of reinforcing steel,

ε_{cp} = strain in concrete at ultimate strain (Table 45 for different grades of concrete), and

E_s = Modulus of elasticity of reinforcing steel (given in 7.2.2.1).

iii) *Lateral confinement condition*

$$\frac{A_{s,min}}{bs_v} = 0.004,$$

b = breadth (mm) of column normal to the direction of reinforcement considered, and

s_v = spacing of transverse reinforcement in the vertical direction.

3) Walls

The minimum area $A_{sv,min}$ (mm²) of transverse reinforcement in a structural wall shall be the larger of the following two requirements:

i) *Thermal condition*

The minimum area $A_{st,min}$ (mm²) of transverse (horizontal) tensile bars per meter length in the thickness direction of the wall shall be:

$$A_{st,min} = 1000t_w \left\{ \begin{array}{l} 0.0025 \text{ Fe250 grade steel} \\ 0.0020 \text{ Fe415 and higher grade steel} \\ 0.0020 \text{ For welded mesh (of bar diameter less than 16mm)} \end{array} \right\},$$

where t_w (in mm) is the thickness of the wall. This horizontal reinforcement shall be provided in two curtains, one on each face of the wall.

ii) *Lateral confinement condition*

The minimum area $A_{st,min}$ of transverse (horizontal) tensile bars along the length of the wall (in both curtains) together shall be:

$$\frac{A_{st,min}}{t_W s_v} = 0.004,$$

t_W = thickness of wall normal to the direction of reinforcement considered,
and

s_v = spacing of transverse reinforcement in the vertical direction.

The minimum area $A_{st,min}$ (mm²) of transverse (horizontal) tensile bars along the thickness of the *wall* per length of the wall shall be:

$$\frac{A_{st,min}}{1000 s_v} = 0.004,$$

s_v = spacing (mm) of transverse reinforcement in the vertical direction.

9.2.4.2 Maximum reinforcement

Concrete members shall not be provided with the amounts of longitudinal and transverse reinforcement more than those specified hereunder.

a) Longitudinal reinforcement

A maximum amount of longitudinal steel is specified hereunder.

1) Slab

The maximum area $A_{st,max}$ of flexural tensile steel in a slab shall be:

$$\frac{A_{st,max}}{bd} = \alpha \left(\frac{f_{ck}}{0.87 f_y} \right) \left(\frac{x_{bal}}{d} \right) = 0.02,$$

where

b = 1000 mm, breadth of the beam in the plane perpendicular to the plane of bending;

d = effective depth of the beam in the plane of bending,

D = depth of the beam in the plane of bending,

x_{bal} = balanced depth of the beam in the plane of bending,

α = average design stress coefficient used to estimate the design compressive force in concrete, when maximum stress block is

developed on the compression side, under balanced condition (Fig. 32);

f_{ck} = characteristic strength of concrete (MPa); and
 f_y = characteristic strength of reinforcing steel (MPa).

2) Beams

The maximum area $A_{st,max}$ of flexural tensile steel in a beam shall be:

$$\frac{A_{st,max}}{bd} = \alpha \left(\frac{f_{ck}}{0.87 f_y} \right) \left(\frac{x_{bal}}{d} \right) = 0.02,$$

and the maximum area $A_{sc,max}$ of flexural compressive steel in a beam shall be:

$$\frac{A_{sc,max}}{bD} = 0.02,$$

where

b = breadth of the beam in the plane perpendicular to the plane of bending;
 d = effective depth of the beam in the plane of bending,
 D = depth of the beam in the plane of bending,
 x_{bal} = balanced depth of the beam in the plane of bending,
 α = average design stress coefficient used to estimate the design compressive force in concrete, when maximum stress block is developed on the compression side, under balanced condition (Fig. 32);
 f_{ck} = characteristic strength (MPa) of concrete; and
 f_y = characteristic strength (MPa) of reinforcing steel.

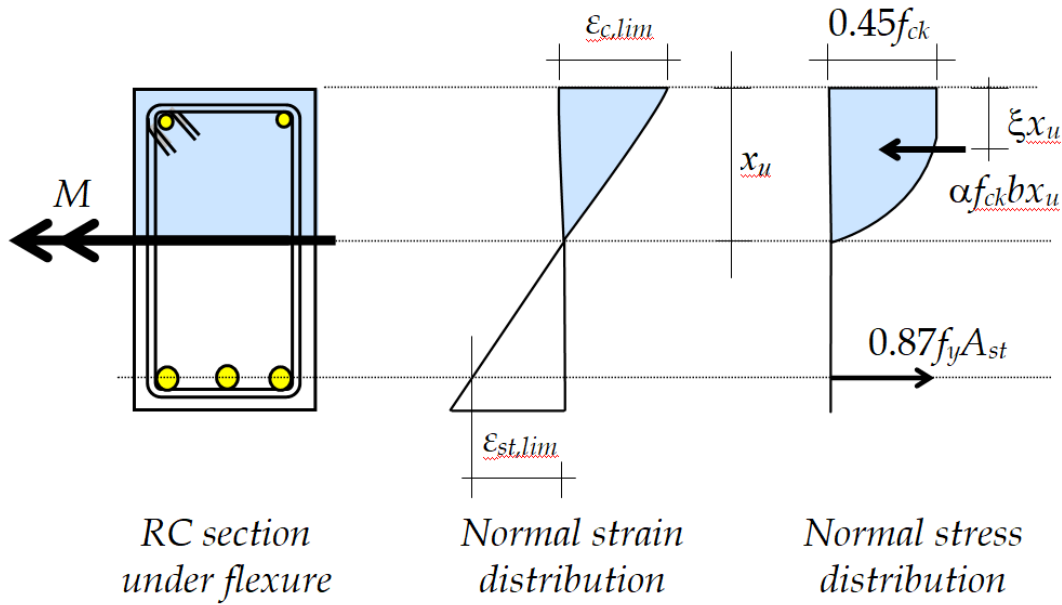


FIG. 32 BALANCED CONDITION OF A CONCRETE BEAM IN FLEXURE

3) Columns

The maximum area $A_{s,max}$ of total longitudinal steel in a column shall be given by:

- i) When longitudinal bars are spliced only by mechanical couplers

$$\frac{A_{s,max}}{bD} = 0.040$$

- ii) When longitudinal bars are spliced only by lapping, or by both mechanical couplers and lapping

$$\frac{A_{s,max}}{bD} = 0.025$$

4) Structural walls and boundary elements of structural walls

The maximum area $A_{sl,max}$ of total longitudinal (vertical) steel in the web of the structural wall in the two curtains together shall be given by:

$$\frac{A_{sl,max}}{t_w L_{web}} = 0.02.$$

The maximum area $A_{sl,max}$ of total longitudinal steel in the boundary element of a structural wall shall be given by:

- i) When longitudinal bars are spliced only by mechanical couplers

$$\frac{A_{s,max}}{t_w L_{web}} = 0.04, \text{ and}$$

- ii) When longitudinal bars are spliced only by lapping, or by both mechanical couplers and lapping

$$\frac{A_{s,max}}{bD} = 0.03.$$

Extra attention shall be exercised during preparation of reinforcement cage and concreting to address the likely congestion at locations of lapping.

b) Transverse reinforcement

A concrete member shall be deemed to have met the maximum transverse steel requirement, if the design shear stress τ_D in beams, columns and structural walls complies with:

$$\tau_D < \tau_{c,max},$$

in accordance with **9.2.6.1(d)**.

9.2.5 Critical Sections

The stress-resultants shall be obtained at least at critical sections as specified hereunder, and the members designed for the same. Reinforcing steel bars shall be designed for the critical sections; these bars can be curtailed at sections farther away from the critical section as per **9.2.5**.

In the design of beams and slabs continuous over supports, the provisions on positioning of imposed load specified in **8.7.3.5.2** shall be applicable.

9.2.5.1 Bending moment

The critical section for negative bending moment in a member shall be at the face of the support in monolithic concrete structures or continuous concrete beams, and at the location determined considering the continuity between the adjoining concrete members in non-monolithic concrete structures.

And, the critical section for positive bending moment in a member shall be:

- a) In simply-supported beam of concrete structures subjected to:
 - 1) Uniformly distributed loads – at mid-span; and
 - 2) Non-uniformly distributed loads – at a location to be determined by calculations; and
- b) at a location determined considering the continuity between the adjoining concrete members in continuous concrete beams, non-monolithic concrete structures and monolithic concrete structures subjected to non-uniformly distributed loads.

9.2.5.2 Shear force

The critical section for shear force in slabs and beams shall be considered as specified hereunder.

9.2.5.2.1 Slabs*a) Solid slabs*

The critical section for shear force in a solid slab shall be taken as per **9.2.5**.

b) Flat slabs

The critical section for shear shall be at a distance $d/2$ from the periphery of the column, capital or drop, measured perpendicular to the plane of the slab, where d is the effective depth of the section (Fig. 33). The shape in plan is geometrically similar to the support immediately below the slab (Fig. 34); for column sections with re-entrant angles, the critical section shall be taken as per Fig. 34(c) and (d).

In columns near the free edge of a flat slab, the critical section shall be taken as per Fig. 35.

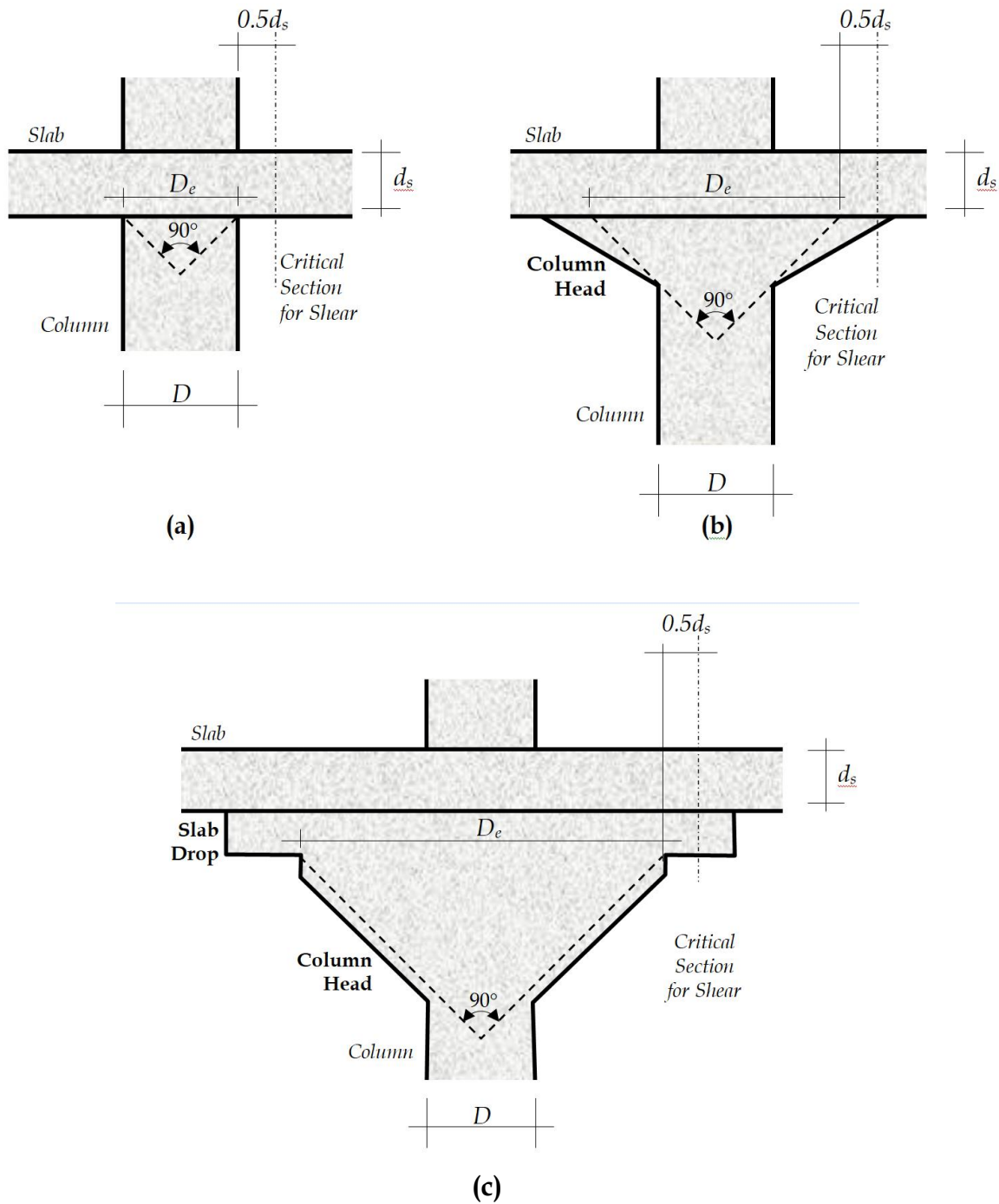


FIG. 33 CRITICAL SECTION FOR SHEAR FORCE IN FLAT SLABS:
(A) SLAB WITHOUT DROP AND COLUMN WITHOUT COLUMN HEAD,
(B) SLAB WITHOUT DROP AND COLUMN WITH COLUMN HEAD, AND
(C) SLAB WITH DROP AND COLUMN WITH COLUMN HEAD

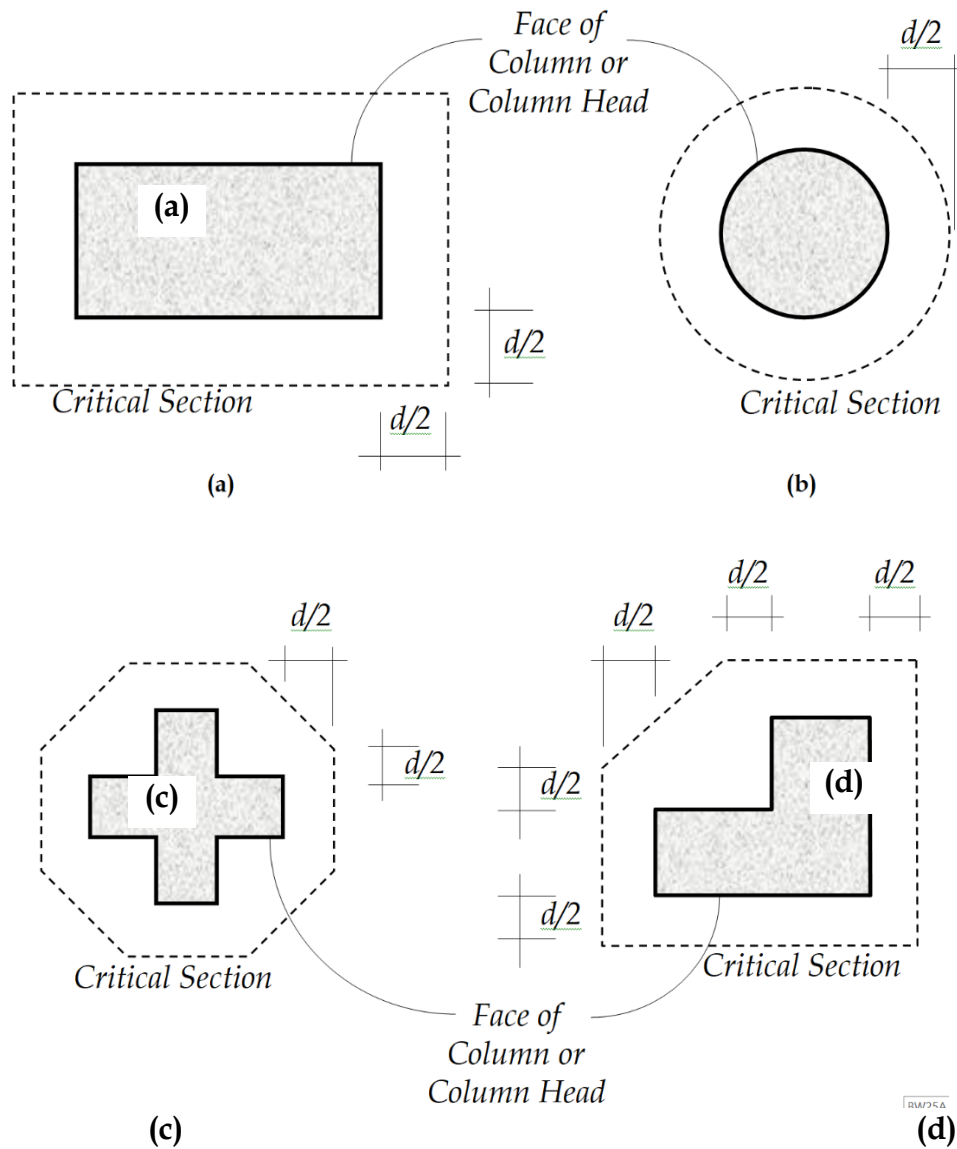


FIG. 34 CRITICAL SECTION IN PLAN FOR SHEAR FORCE IN FLAT SLABS

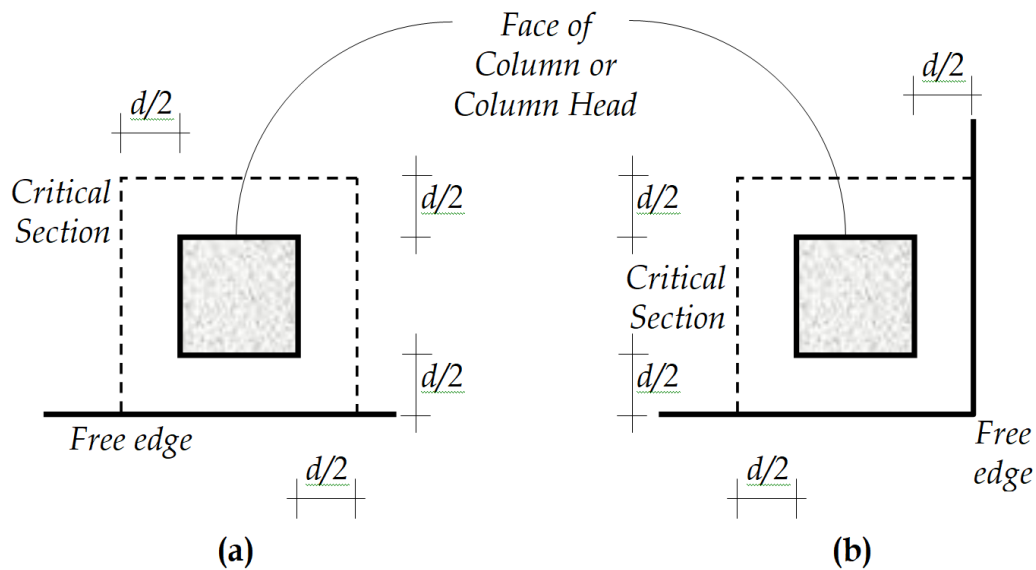


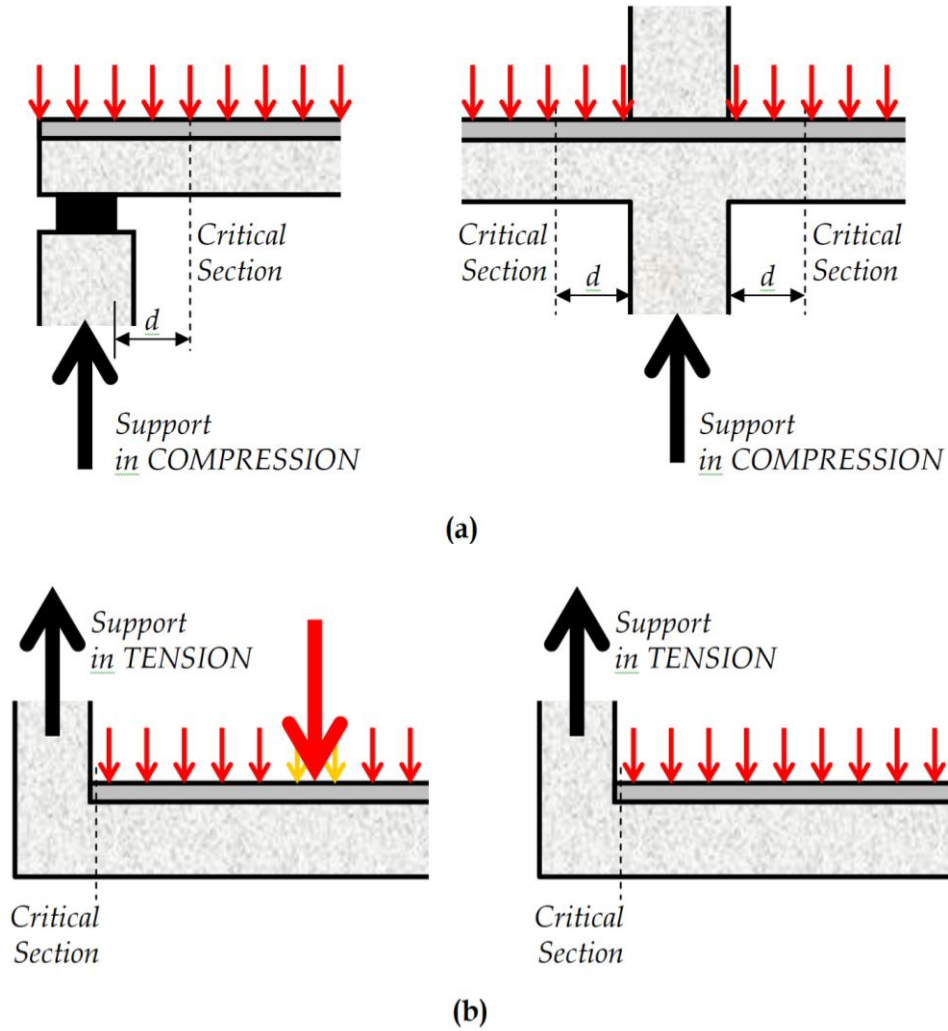
FIG. 35 EFFECT OF FREE EDGES ON CRITICAL SECTION (IN PLAN) FOR SHEAR FORCE IN FLAT SLABS

9.2.5.2.2 Reinforced concrete beams

When beams carry uniformly distributed load, or the principal load is a concentrated load and located at a distance more than $2d$ from the face of the support (even though the reaction due to the applied load introduces compression in the support), the critical section for shear force shall be at:

- The face of the support, when the member is supported from below [Fig. 36(a)] (such that the reaction due to applied loads introduces compression in the support); and
- A distance d from the face of the support, when the member is supported from above [Fig. 36(b)] (such that the reaction due to applied loads introduces tension in the support). Here, sections located at a distance less than d from the face of the support shall be designed for the same shear force as that computed at a distance d from the face of the support,

else the critical section for bending moment in a member shall be at the face of the support [Fig. 36(c)].



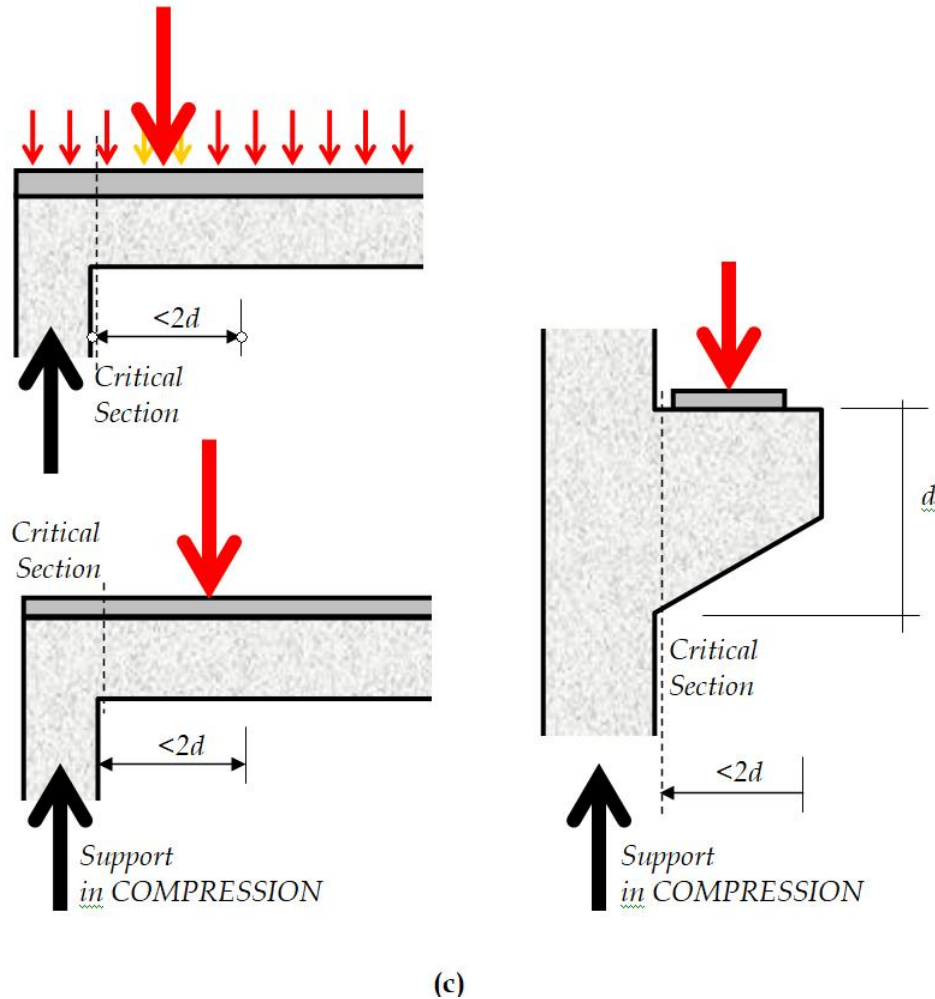


FIG. 36 CRITICAL SECTION FOR SHEAR FORCE WHEN SUPPORT IS IN:

- (A) COMPRESSION, AND LOAD IS UNIFORM – AT A DISTANCE d FROM THE FACE OF THE COLUMN,
- (B) TENSION, AND THE LOAD IS UNIFORM OR CONCENTRATED – AT THE FACE OF THE COLUMN, AND
- (C) COMPRESSION, AND LOAD IS UNIFORM OR CONCENTRATED – AT THE FACE OF THE COLUMN

9.2.5.2.3 PSC beams

The critical section for shear force in a PSC beam shall be taken at a distance d from the face of the support [as per Fig. 36(d)].

9.2.6 Design Strengths of Concrete and Steels

This standard refers to three types of strengths, namely characteristic strength, design strength, and overstrength. The design strength f_D of concrete, reinforcing steel and prestressing steel shall be obtained as:

$$f_D = \frac{f_{characteristic}}{\gamma_M},$$

where

$f_{characteristic}$ = characteristic strength of the material specified in 7.1, 7.2 and 7.3, and

γ_M = partial safety factor appropriate for material and the loading action being considered.

The design strengths of concrete and steels shall be used in the design of the structural concrete sections and members.

Higher grades of materials than that used in design shall not be used in beams.

9.2.6.1 Concrete

In the design of concrete sections and members, design lengths of lap splices and design development lengths (including design lengths of anchorage), the following seven design strengths of concrete shall be considered as specified hereunder:

- a) Limiting design bearing strength f_{Lbr} ,
- b) Limiting design bond strength τ_{Lbd} ,
- c) Limiting design axial strength f_{Lac} in axial compression, and limiting design axial strength f_{Lat} in axial tension,
- d) Limiting design shear stress capacity τ_{Lc} , and
- e) Limiting design flexural strength f_{Lbc} in bending compression and limiting design flexural strength f_{Lbt} in bending tension.

Designers may use values lower than those specified hereunder.

The partial safety factor γ_M used in arriving at the above design strengths of concrete are given in Table 42. Larger values of γ_M than those specified shall be considered, if the

engineer-in-charge deems it reasonable, for example, if RMC truck does not reach on time or if it is expected to reach late.

Table 42 Partial Safety Factors γ_M for Concrete Strengths

[Clause 9.2.6.1]

S.No.	Limiting Design Strength of Concrete	Partial Safety Factor γ_M
(1)	(2)	(3)
i)	Design Bearing Strength f_{Lbr}	1.5
ii)	Design Bond Strength τ_{Lbd}	1.8
iii)	Design Axial Strength	
	(a) f_{Lac} in compression	1.5
	(b) f_{Lat} in direct tension	–
iv)	Design Shear Strength	
	(a) τ_{Lc} in flexural shear	1.8
	(b) $\tau_{Lc,max}$ in maximum flexural shear	1.8
	(c) τ_{Lcp} in punching shear	1.8
	(d) τ_{Lcj} in distortional shear	1.8
v)	Design Flexural Strength	
	(a) f_{Lbc} in bending compression	1.5
	(b) f_{Lbt} in bending tension	–

a) Design bearing strength f_{Lbr}

The design bearing strength f_{Lbr} of concrete shall be taken as $0.45f_{ck}$, which is given by:

$$f_{Lbr} = \frac{0.68f_{ck}}{\gamma_M} = 0.45f_{ck},$$

where

f_{ck} = characteristic compressive strength of concrete, and

γ_M = partial safety factor for concrete = 1.5

b) Design bond strength τ_{Lbd}

The design bond strength τ_{Lbd} for reinforcing bars embedded in concrete shall be taken as:

$$\tau_{Lbd} = \begin{cases} \frac{0.300(f_{ck})^{0.6}}{\gamma_M} = 0.167(f_{ck})^{0.6} \leq 1.9 \text{ MPa} & \text{Fe250 bars in Tension} \\ \frac{0.480(f_{ck})^{0.6}}{\gamma_M} = 0.267(f_{ck})^{0.6} \leq 3.1 \text{ MPa} & \text{Fe415, Fe500 \& Fe550 bars in Tension} \\ \frac{0.375(f_{ck})^{0.6}}{\gamma_M} = 0.208(f_{ck})^{0.6} \leq 2.4 \text{ MPa} & \text{Fe250 bars in Compression} \\ \frac{0.600(f_{ck})^{0.6}}{\gamma_M} = 0.333(f_{ck})^{0.6} \leq 3.9 \text{ MPa} & \text{Fe415, Fe500 \& Fe550 bars in Compression} \end{cases}$$

where γ_M = partial safety factor for concrete = 1.8

In case of earthquake loading, the values given in Table 42 for bars in tension in beams and columns shall be used for bars on both faces.

In self compacting concretes, the design shear strength of concretes are much smaller than the values specified above. The above values shall be modified with suitable reduction factors determined based on experiments. In the absence of such experimental data, a reduction factor of **0.6** is recommended.

c) Design axial strengths f_{Lac} and f_{Lat}

The design axial strength f_{Lac} of concrete in axial compression shall be taken as:

$$f_{Lac} = \frac{0.68f_{ck}}{\gamma_M} = 0.45f_{ck},$$

where γ_M = partial safety factor for concrete in compression = 1.5

The design axial strength f_{Lat} of concrete in axial tension shall be taken as 0, when estimating the bending moment capacity or the axial force capacity in tension of a concrete section or member.

d) Design shear strengths

The design shear strength τ_{Lc} of concrete in concrete members provided with transverse reinforcement shall be taken as specified hereunder.

The design shear strength τ_{Lc} of concrete and the maximum design shear strength $\tau_{Lc,max}$ of concrete shall be taken as:

$$\tau_{Lc} = \left(\frac{0.36 \lambda_a \lambda_s \lambda_g \sqrt[3]{(p_t f_{ck})}}{\gamma_M} \right) \delta = (0.2 \lambda_a \lambda_s \lambda_g \sqrt[3]{(p_t f_{ck})}) \delta, \text{ and}$$

$$\tau_{Lc,max} = \begin{cases} \left(\frac{0.15 f_{ck}}{\gamma_M} \right) \delta = (0.083 f_{ck}) \delta & \text{Concretes of grades M60 and lower} \\ \left(\frac{9 \text{ MPa}}{\gamma_M} \right) \delta = (5 \text{ MPa}) \delta & \text{Concretes of grades higher than M60} \end{cases},$$

where

$$\lambda_a = \text{Aggregate type factor} = \begin{cases} 1.00 & \text{Normal weight aggregate} \\ 0.75 & \text{Light weight aggregate} \end{cases},$$

$$\lambda_s = \text{Cross-section Size factor} = \begin{cases} 1.00 & \text{Effectivedepth } d \leq 400 \text{ mm} \\ \left(\frac{400}{d} \right)^{0.25} & \text{Effectivedepth } d > 400 \text{ mm} \end{cases},$$

$$\lambda_g = \text{Cross-section geometry factor} = \begin{cases} 0.83 & \text{Rectangular or square cross - section} \\ 0.70 & \text{Circular cross - section} \end{cases},$$

$$\gamma_M = \text{Partial safety factor for concrete in Shear} = 1.8,$$

$$f_{ck} = \text{Characteristic Compressive Strength of concrete (MPa), and}$$

$$p_t = \text{Fraction } A_{st}/bd \text{ of tension steel provided in the beam,}$$

$$\delta = \begin{cases} \left(1 + \frac{3P_D}{A_g f_{ck}} \right) \leq 1.5 & \text{for compressive axial load} \\ \left(1 - \frac{12P_D}{A_g f_{ck}} \right) \geq 0 & \text{for tensile axial load} \end{cases},$$

in which

$$A_{st} = \text{area of longitudinal tensile steel in the beam,}$$

$$b = \text{breadth of the beam parallel to the axis of bending,}$$

$$d = \text{effective depth of the beam perpendicular to the axis of bending,}$$

$$P_D = \text{design Axial Load on member obtained from factored loads applied on the structure,}$$

A_g = gross area of the concrete column, and
 f_{ck} = characteristic compressive stress capacity of concrete.

The design punching shear strength τ_{Lcp} of concrete shall be taken as:

$$\tau_{Lcp} = k_s \tau_{cp},$$

where

$$\tau_{cp} = \frac{0.375\sqrt{f_{ck}}}{\gamma_M} = 0.208\sqrt{f_{ck}}, \text{ and}$$

$$k_s = \left(0.5 + \frac{C_1}{C_2}\right) \leq 1,$$

wherein γ_M = partial safety factor in concrete for punching shear, taken as 1.8.

The design shear strength $\tau_{Lc,slab}$ of concrete in solid slabs shall be taken as:

$$\tau_{Lc,slab} = k_{slab} \tau_{Lc},$$

where

k_{slab} = Factor to account for larger thickness of solid slab, given below, and

τ_{Lc} = design shear strength of concrete given above.

This provision shall not apply to flat slabs.

Overall Depth (mm) D	300 or more	275	250	225	200	175	150 or less
k_{slab}	1.00	1.05	1.10	1.15	1.20	1.25	1.30

The design shear strength $\tau_{Lc,wall}$ and the maximum design shear strength $(\tau_{Lc,wall})_{\max}$ of concrete in structural walls shall be taken as:

$$\tau_{Lc,wall} = \begin{cases} \frac{0.30\sqrt{f_{ck}}}{\gamma_M} \left(3 - \frac{H_w}{L_w}\right) = 0.167\sqrt{f_{ck}} \left(3 - \frac{H_w}{L_w}\right) & \left(\frac{H_w}{L_w}\right) \leq 1 \\ \left[0.135 + 0.265\left(\frac{H_w}{L_w} - 2\right)\right]\sqrt{f_{ck}} & 1 < \left(\frac{H_w}{L_w}\right) < 2, \text{ and} \\ \frac{0.0675\sqrt{f_{ck}}}{\gamma_M} \left[\frac{\left(\frac{H_w}{L_w} + 1\right)}{\left(\frac{H_w}{L_w} - 1\right)}\right] = 0.0375\sqrt{f_{ck}} \left[\frac{\left(\frac{H_w}{L_w} + 1\right)}{\left(\frac{H_w}{L_w} - 1\right)}\right] & 2 \leq \left(\frac{H_w}{L_w}\right) \leq 7 \end{cases}$$

$$(\tau_{Lc,wall})_{\max} = 0.14f_{ck},$$

and the permissible shear stress $\tau_{Pc,wall}$ and the maximum permissible shear stress $(\tau_{Pc,wall})_{\max}$ of concrete in structural walls as:

$$\begin{aligned} \tau_{Pc,wall} &= 0.108\sqrt{f_{ck}} \left(3 - \frac{H_w}{L_w}\right), \text{ and} \\ &\leq 0.083\sqrt{f_{ck}} \end{aligned}$$

$$(\tau_{Pc,wall})_{\max} = 0.1f_{ck}.$$

where

f_{ck} = characteristic strength of concrete,

H_w = Height of the wall from its base,

L_w = Length of the Wall at its base,

γ_M = partial safety factor in concrete for punching shear, taken as 1.8.

The design distortional shear strength τ_{Lcj} of concrete in beam-column joints shall be taken as:

$$\tau_{Lcj} = \begin{cases} \frac{1.125\sqrt{f_{ck}}}{\gamma_M} & \text{Joints confined by beams on all 4 faces} \\ \frac{0.90\sqrt{f_{ck}}}{\gamma_M} & \text{Joints confined by beams on 3 faces} \\ \frac{0.75\sqrt{f_{ck}}}{\gamma_M} & \text{Other joints} \end{cases},$$

where γ_M = partial safety factor for concrete in joint shear, taken as 1.8

e) Design flexural strengths f_{Lbc} and f_{Lbt}

The design flexural strength f_{Lbc} of concrete in compression shall be taken as:

$$f_{Lbc} = \frac{0.68f_{ck}}{\gamma_M} = 0.45f_{ck},$$

where γ_M = partial safety factor for concrete in compression = 1.5

The design flexural strength f_{Lbt} of concrete in tension shall be taken as 0, when estimating the design bending moment capacity M_L of a concrete section or member.

9.2.6.2 Reinforcing steel

In the design of concrete sections and members, the following three design strengths of reinforcing steel shall be considered as specified hereunder:

- a) Design axial strength f_{Lst} of reinforcing steel in tension,
- b) Design axial strength f_{Lsc} of reinforcing steel in compression, and
- c) Design shear strength τ_{Ls} .

Designers may use values lower than those specified hereunder.

a) Partial safety factors γ_M for reinforcing steel

The partial safety factor γ_M for reinforcing steel shall be taken as given in Table 43.

Table 43 Partial Safety Factors γ_M for Reinforcing Steel

[Clause 9.2.6.2]

Sl No.	Design Strength of Reinforcing Steel	Partial Safety Factor γ_M
(1)	(2)	(3)
i)	Design axial strength	
	(a) Design Axial Strength f_{Lst} in tension	1.15

	(b) Design Axial Strength f_{Lsc} in compression	1.15
ii)	Design shear strength τ_{Ls}	1.15

b) Design axial strength f_{Lst}

The design axial strength f_{Lst} of reinforcing steel in tension shall be taken as $0.87 f_y$, which is given by:

$$f_{Lst} = \frac{f_y}{\gamma_M} = 0.87 f_y,$$

where

f_y = Characteristic tensile strength of reinforcing steel, and

γ_M = Partial safety factor for reinforcing steel in tension = 1.15.

The design axial strength f_{Lsc} of reinforcing steel in compression shall be taken as the design axial strength f_{Lst} of reinforcing steel in tension.

c) Design shear strength τ_{Ls}

The design shear strength τ_{Ls} of reinforcing steel bars shall be taken as:

$$\tau_{Ls} = \left(\frac{f_y / \sqrt{3}}{\gamma_M} \right)$$

9.2.6.3 Prestressing steel

In the design of prestressed concrete sections and members, the design axial strength f_{Lpt} of prestressing steel in tension and design shear strength τ_{Lpt} shall be considered. Designers may use values lower than those specified hereunder.

a) Partial safety factors γ_M for prestressing steel

The partial safety factor γ_M for prestressing steel strengths shall be taken as given in Table 44.

Table 44 Partial Safety Factors γ_M for Prestressing Steel

[Clause 9.2.6.3(a)]

Sl No.	Design Strength of Prestressing Steel	Partial Safety Factor γ_M
(1)	(2)	(3)
i)	Design axial strength f_{Lpt} in tension	1.15
ii)	Design shear strength τ_{Lpt}	1.15

Table 45 Strains at Design Peak Strength ε_{cp} and Ultimate Strain ε_{cu} of unconfined concrete in compression

[Clause 9.2.4.1(b)(2) and 9.2.6.4]

Sl No.	Characteristic Compressive Strength f_{ck} (MPa) of Concrete	ε_{cp}	ε_{cu}
(1)	(2)	(3)	(4)
i)	20	0.002 00	0.003 50
ii)	25	0.002 03	0.003 43
iii)	30	0.002 05	0.003 36
iv)	35	0.002 08	0.003 29
v)	40	0.002 10	0.003 23
vi)	45	0.002 13	0.003 16
vii)	50	0.002 15	0.003 09
viii)	55	0.002 18	0.003 02
ix)	60	0.002 20	0.002 95
x)	65	0.002 23	0.002 88
xi)	70	0.002 25	0.002 81

9.2.6.4 Confined concrete

The monotonic stress-strain ($f_{cc} - \varepsilon_{cc}$) curves of confined concrete in compression (see Fig. 37), to be used in the assessment of concrete structures, shall be taken as:

$$f_{cc}(\varepsilon_{cc}) = f_{cp} \begin{cases} \left(2\rho_\varepsilon - 2\rho_s\right) \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right)^3 - (4\rho_\varepsilon - 3\rho_s) \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right)^2 + 2\rho_\varepsilon \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right) & 0 < \varepsilon_{cc} \leq \varepsilon_T \\ f_T - \left[\frac{df_c}{d\varepsilon_c} \right]_{\varepsilon=\varepsilon_T} (\varepsilon_T - \varepsilon_{cc}) & \varepsilon_T < \varepsilon_{cc} < \varepsilon_{f_{\alpha}=0.5f_{cp}} \\ 0.5f_{cp} & \varepsilon_{f_{\alpha}=0.5f_{cp}} \leq \varepsilon_{cc} \end{cases},$$

where

ε_{ccT} = Strain at point of tangent to post-peak confined $\sigma - \varepsilon$ curve

$$= \varepsilon_{ccp} \left[\frac{4\rho_\varepsilon - 3\rho_s}{6\rho_\varepsilon - 6\rho_s} \right],$$

f_{ccT} = Stress at point of tangent to post-peak confined $\sigma - \varepsilon$ curve

$$= f_{cp} \left[(4\rho_\varepsilon - 3\rho_s) \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right)^3 - (4\rho_\varepsilon - 3\rho_s) \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right)^2 + 2\rho_\varepsilon \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right) \right],$$

$\left. \frac{df_{cc}}{d\varepsilon_{cc}} \right|_{\varepsilon_{\alpha}=\varepsilon_{ccT}}$ = Slope at point of tangent to post-peak confined $\sigma - \varepsilon$ curve

$$= - \left(\frac{f_{cp}}{\varepsilon_{cp}} \right) \left[(4\rho_\varepsilon - 3\rho_s) \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right)^2 - (4\rho_\varepsilon - 3\rho_s) \left(\frac{\varepsilon_{cc}}{\varepsilon_{ccp}}\right) + 2\rho_\varepsilon \right], \text{ and}$$

$$\varepsilon_{f_{cc}=0.5f_{cp}} = \varepsilon_{ccT} - \frac{(f_{ccT} - 0.5f_{cp})}{\left. \frac{df_{cc}}{d\varepsilon_{cc}} \right|_{\varepsilon_{\alpha}=\varepsilon_{ccT}}},$$

wherein

$$\rho_e = \text{effective percentage area ratio} = \begin{cases} \frac{2\rho_{ax}\rho_{ay}}{\rho_{ax} + \rho_{ay}} & \text{Rectangular section} \\ \rho_a & \text{Circular section} \end{cases},$$

$$\rho_s = \text{strength enhancement factor} = \left(\frac{f_{ccp}}{f_{cp}} \right) = 1 + 5\rho_e \left(\frac{f_y}{f_{cp}} \right),$$

$$\rho_\varepsilon = \text{strain enhancement factor} = \left(\frac{\varepsilon_{ccp}}{\varepsilon_{cp}} \right) = 1 + 2(\rho_s - 1),$$

f_{cp} = characteristic strength of unconfined concrete = $0.68f_{ck}$, and

f_{ccp} = peak strength of confined concrete,

ε_{cp} = strain at peak strength of unconfined concrete, as per Table 45, and

ε_{cu} = ultimate strain capacity of unconfined concrete, and

ε_{ccu} = ultimate strain capacity of confined concrete = $\text{Min}(\rho_e \varepsilon_{cu}; \varepsilon_{0.85f_{ccp} \text{ Post Peak}})$.

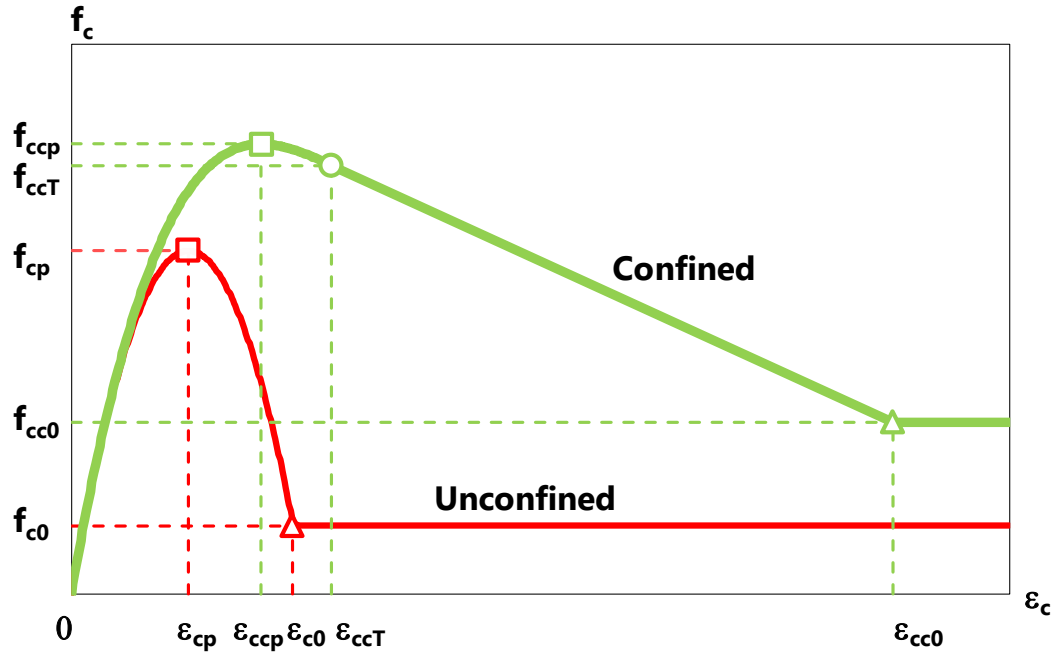


FIG. 37 STRESS-STRAIN CURVES OF CONFINED CONCRETE IN COMPRESSION FOR USE IN ASSESSMENT OF CONCRETE STRUCTURES

where

ρ_{ax} = Area ratio of transverse reinforcement along x-direction of a rectangular section

$$= \sum_{i=1}^{N_{TL}} A_{sxi} / B s_v ,$$

ρ_{ay} = Area ratio of transverse reinforcements along y-direction of a rectangular section

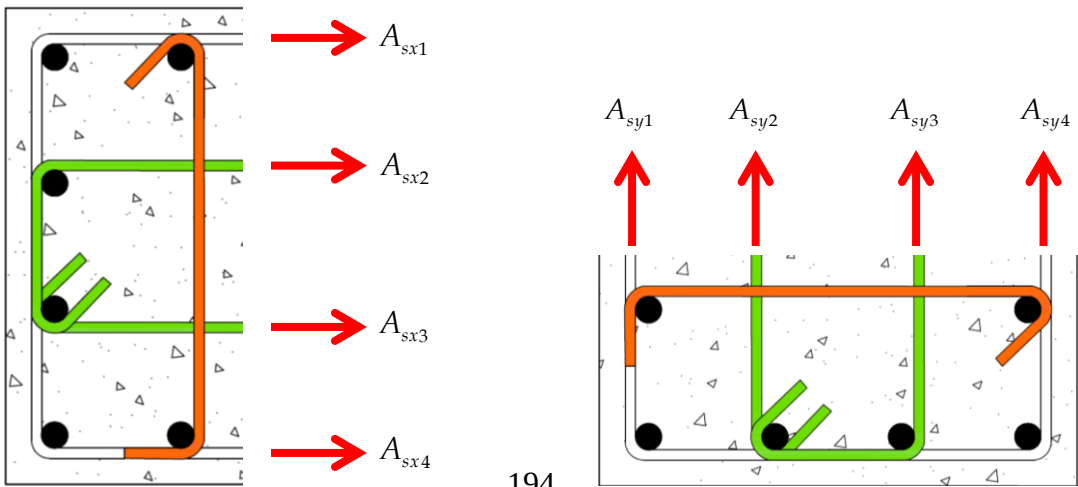
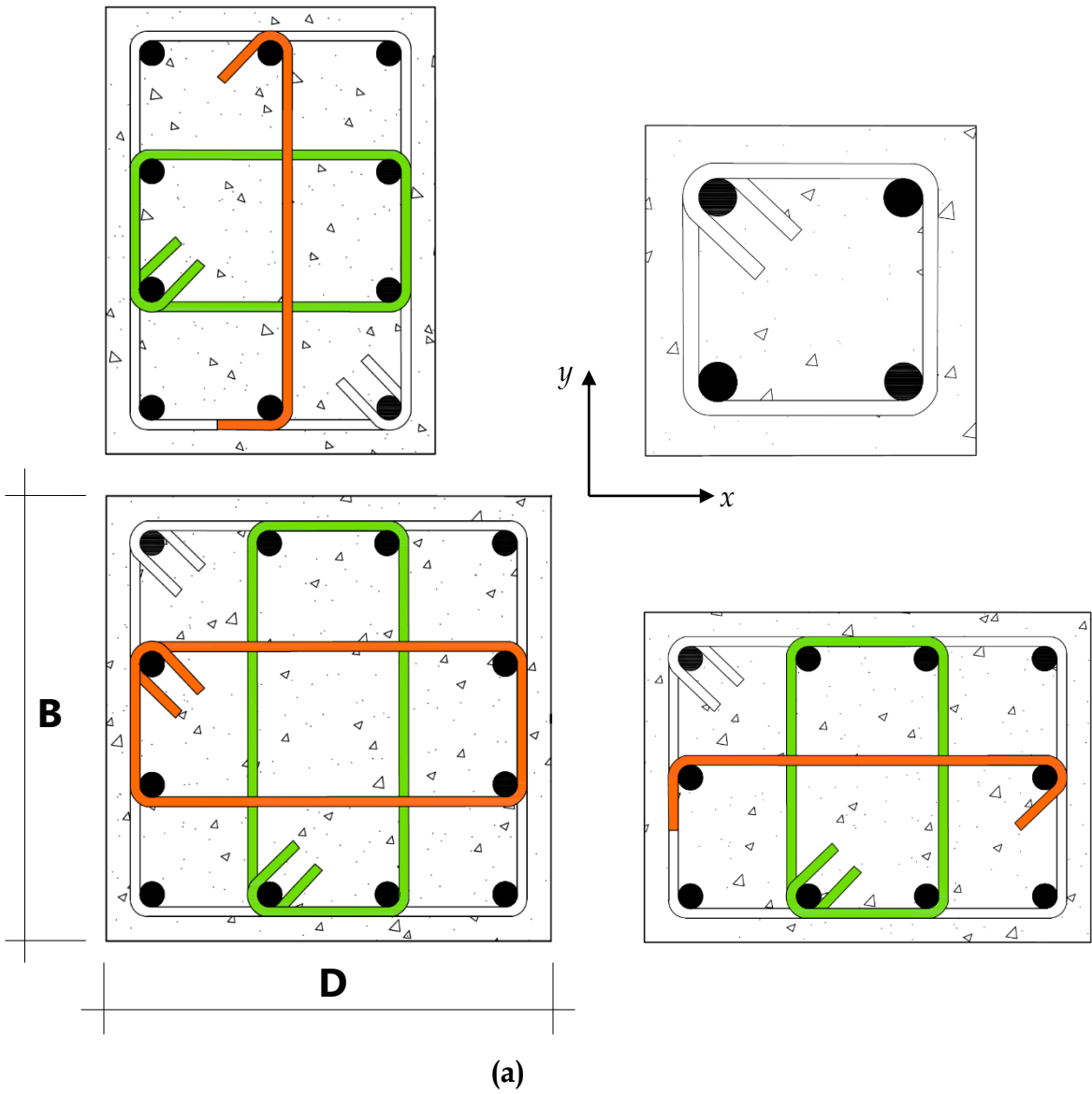
$$= \sum_{i=1}^{N_{TL}} A_{syi} / D s_v , \text{ and}$$

ρ_a = Area ratio of transverse reinforcements at any section of a circular section

$$= \left(2A_{sh} + \sum_i (A_{sL} \sin \theta)_i \right) / \phi s_v ,$$

wherein

- A_{sxi} = Area of cross-section of transverse bar i along x-direction perpendicular to dimension B of a rectangular section (Fig. 38b),
 A_{sy} = Area of cross-section of transverse bar i along y-direction perpendicular to dimension D of a rectangular section (Fig. 38c),
 A_{sh} = Area of cross-section of transverse hoop of a circular section (Fig. 39),
 A_{sL} = Area of cross-section transverse link i in a circular section (Fig. 39),
 N_L = Number of transverse links in the circular section (Fig. 39),
 θ_i = Angle subtended by link i from the diametrical cut section in a circular section (Fig. 39a),
 B = Dimension of cross-section along y-direction perpendicular to the transverse reinforcements along x-direction of a rectangular section (Fig. 38a), and
 D = Dimension of cross-section along x-direction perpendicular to the transverse reinforcements along y-direction of a rectangular section (Fig. 38a),
 ϕ = Diameter of the cross-section of a circular section (Fig. 39a), and
 s_v = Spacing of transverse reinforcement along the longitudinal axis of the member being confined.



(b) (c)
FIG. 38 DETAILS OF GEOMETRIC PARAMETERS USED IN ESTIMATION OF ρ_e IN RECTANGULAR SECTIONS

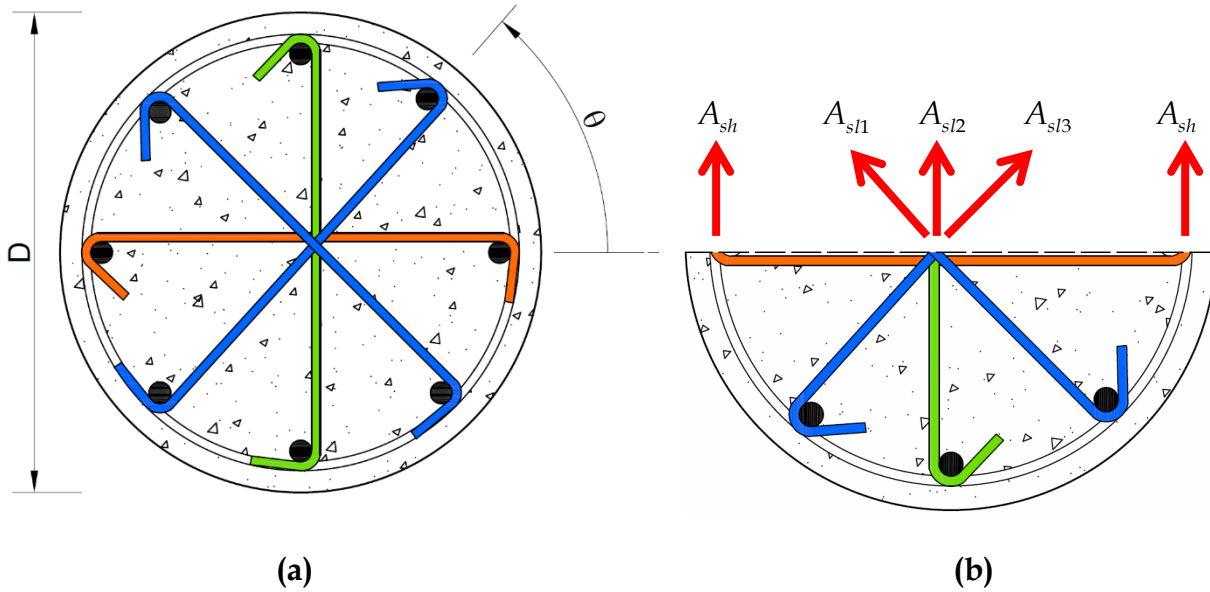


FIG. 39 DETAILS OF GEOMETRIC PARAMETERS USED IN ESTIMATION OF ρ_e IN CIRCULAR SECTIONS

9.2.6.5 Reinforcing steel

The monotonic stress-strain ($f_s - \varepsilon_s$) curve of reinforcing steel in both tension and compression shall be taken as (Fig. 40):

a) Design :
$$f_s(\varepsilon_s) = \begin{cases} \frac{f_y}{\gamma_M} \left(\frac{\varepsilon_s}{\varepsilon_y / \gamma_M} \right) & \text{for } 0 \leq \varepsilon_s \leq \left(\frac{\varepsilon_y}{\gamma_M} \right) \\ \frac{f_y}{\gamma_M} & \text{for } \left(\frac{\varepsilon_y}{\gamma_M} \right) < \varepsilon_s \leq \varepsilon_r \end{cases},$$

b) Characteristic : $f_s(\varepsilon_s) = \begin{cases} f_y \left(\frac{\varepsilon_s}{\varepsilon_y} \right) & \text{for } 0 \leq \varepsilon_s \leq \varepsilon_y \\ f_y & \text{for } \varepsilon_y < \varepsilon_s \leq \varepsilon_r \end{cases}, \text{ and}$

c) Overstrength : $f_s - \varepsilon_s$ curve determined from tests on actual *reinforcing bars*, where

f_y = Characteristic yield stress of reinforcing steel under uniaxial tension,

f_y^* = Actual yield stress of reinforcing steel under uniaxial tension,

f_u^* = Actual ultimate stress of reinforcing steel under uniaxial tension,

γ_M = partial safety factor for reinforcing steel, taken as 1.15,

E_s = Modulus of Elasticity of reinforcing steel, taken as per 7.2.2.1,

E_{sh} = Strain hardening modulus of reinforcing steel, taken as $(f_u^* - f_y^*)/(\varepsilon_r - \varepsilon_y)$

ε_y = f_y/E_s , yield strain of reinforcing steel under uniaxial tension,

$\varepsilon_{r\Omega}$ = Actual ultimate strain in reinforcing steel, taken from experimental data, and

Ω_u = ultimate overstrength factor for reinforcing steel, taken as f_u^*/f_y from experimental data.

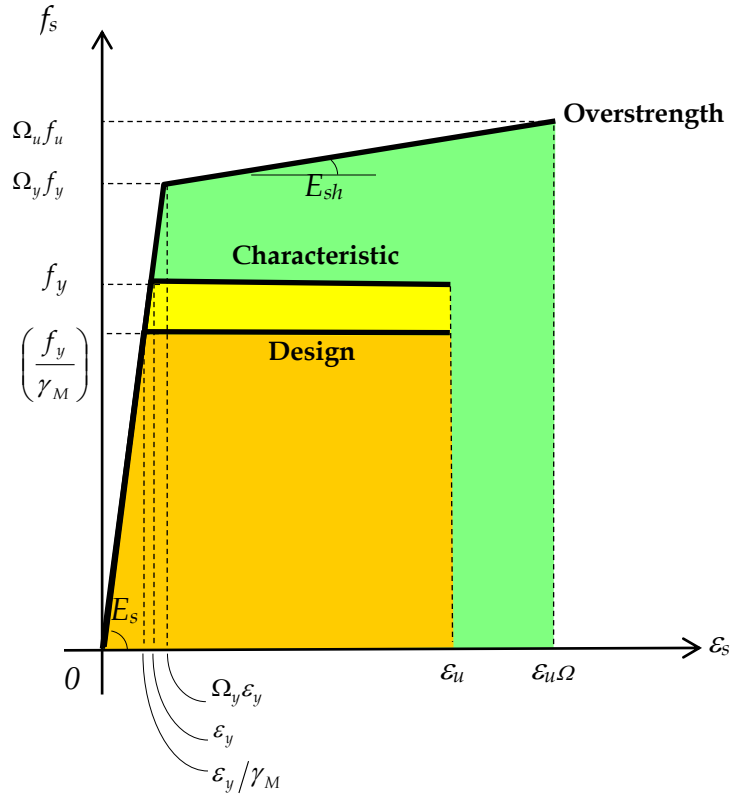


FIG. 40 DESIGN STRESS-STRAIN CURVES OF REINFORCING STEEL IN TENSION AND IN COMPRESSION FOR USE AS DESIGN, CHARACTERISTIC AND OVERSTRENGTH CURVES

9.2.6.6 Prestressing steel

The monotonic stress-strain ($f_{sp} - \varepsilon_{sp}$) curve of prestressing steel in both tension and compression shall be taken as (Fig. 41):

a) Design :
$$f_{sp}(\varepsilon_{sp}) = \begin{cases} \frac{f_{yp}}{\gamma_M} \left(\frac{\varepsilon_{sp}}{\varepsilon_{yp}} \right) & \text{for } 0 \leq \varepsilon_{sp} \leq \varepsilon_{yp} \\ \frac{f_{yp}}{\gamma_M} + \frac{(\varepsilon_{sp} - \varepsilon_{yp}) E_{shp}}{\gamma_M} & \text{for } \varepsilon_{yp} < \varepsilon_{sp} \leq \varepsilon_{up} \end{cases},$$

- b) Simplified Design :**
$$f_{sp}(\varepsilon_{sp}) = \begin{cases} \frac{f_{yp}}{\gamma_M} \left(\frac{\varepsilon_{sp}}{\varepsilon_{yp}} \right) & \text{for } 0 \leq \varepsilon_{sp} \leq \varepsilon_{yp} \\ \frac{f_{yp}}{\gamma_M} & \text{for } \varepsilon_{yp} < \varepsilon_{sp} \leq \varepsilon_{up} \end{cases},$$
- c) Characteristic :**
$$f_{sp}(\varepsilon_{sp}) = \begin{cases} f_{yp} \left(\frac{\varepsilon_{sp}}{\varepsilon_{yp}} \right) & \text{for } 0 \leq \varepsilon_{sp} \leq \varepsilon_{yp} \\ f_{yp} + (\varepsilon_{sp} - \varepsilon_{yp})E_{shp} & \text{for } \varepsilon_{yp} < \varepsilon_{sp} \leq \varepsilon_{up} \end{cases}, \text{ and}$$
- d) Overstrength :** $f_{sp}^* - \varepsilon_{sp}^*$ curve determined from tests on actual *prestressing steel* used,

where

- f_{yp} = characteristic yield strength of prestressing steel under uniaxial tension,
 f_{up} = characteristic ultimate strength of prestressing steel under uniaxial tension,
 f_{yp}^* = Actual yield strength of prestressing steel under uniaxial tension,
 f_{up}^* = actual ultimate strength of prestressing steel under uniaxial tension,
 γ_M = partial safety factor for prestressing steel, taken as 1.15,
 E_{sp} = modulus of elasticity of prestressing steel, taken as per 7.3.1.2.1,
 E_{shp} = strain hardening modulus of prestressing steel, taken as $(f_{up} - f_{yp})/(\varepsilon_{up} - \varepsilon_{yp})$,
 ε_{yp} = characteristic yield strain of prestressing steel under uniaxial tension (= f_{yp}/E_{sp}),
 ε_{up} = Characteristic ultimate strain capacity of prestressing steel, minimum assured value from manufacturer's data,
 Ω_{up} = ultimate overstrength factor of prestressing steel, taken as f_{up}^*/f_{up} from experimental data, and
 Ω_{yp} = yield overstrength factor of prestressing steel, taken as f_{yp}^*/f_{yp} from experimental data.

The simplified design monotonic stress-strain ($f_{sp} - \varepsilon_{sp}$) curve shall be used only in simply supported prestressed concrete members, whose failure does not jeopardize life.

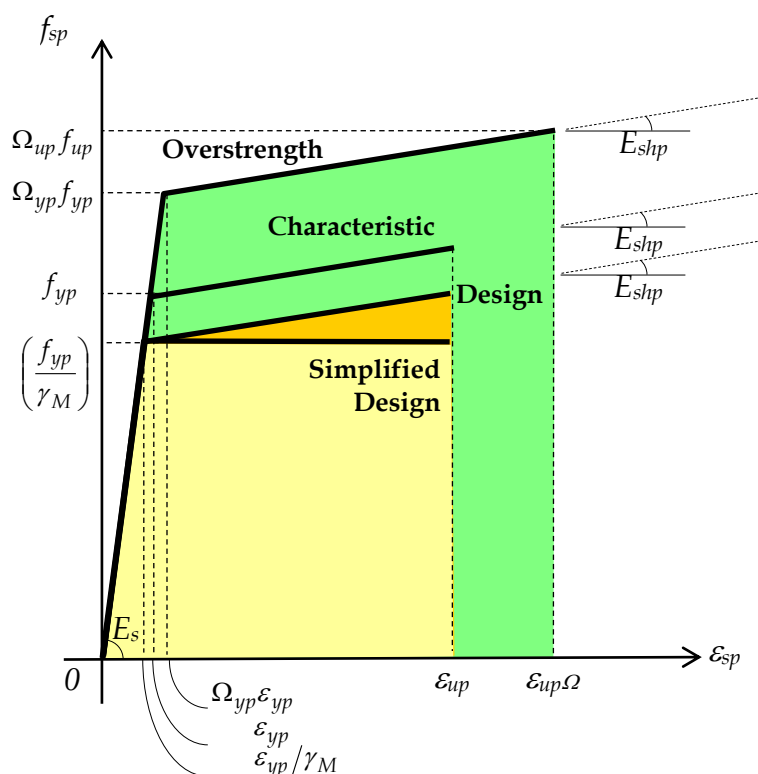


FIG. 41 DESIGN STRESS-STRAIN CURVES OF PRESTRESSING STEEL IN TENSION FOR USE AS DESIGN, CHARACTERISTIC AND OVERSTRENGTH CURVES

9.3 DESIGN FOR STRENGTH

The design for strength of a structure shall be performed by designing each of its members along with the joints and connections between them to resist the elongation/compression, bending, shearing or twisting (and the consequent stress-resultants) imposed on them by the probable most unfavourable combination of loads applied on the structure as specified in **Table 3** (with appropriate partial safety factors γ_L for loads in each combination).

The design strength of a member at a section (including the effect of partial safety factors γ_M for material strengths) shall be more than the corresponding demand associated with each stress-resultant or in combination, and shall be estimated as its resistance corresponding to when any of its constituent materials reaches a limit state (either of strain or stress) specified hereunder.

9.3.1 Stability

The structure shall meet the following requirements related to stability.

a) *Equilibrium stability*

A structure shall meet the provisions of overall stability specified in **9.1** before its members, joints and connections are designed for strength.

b) *Elastic stability*

Each member (namely beam, column and structural wall) shall meet the maximum slenderness ratios specified in **9.2.3.1** before it is designed for strength. Failure resulting from elastic instability in beams and columns can be avoided by complying with **9.2.3.1** and **9.2.3.2**, and accounting for effective lengths of members under axial, bending and combined axial-bending actions by suitably proportioning its *size* (as per **9.2.3**), and accounting for its bending moment capacity for the axial load appearing on it (as specified in **9.3.2.3** and **9.3.2.4**).

c) *Plastic stability*

The structure shall meet the provisions specified in **5.7** and **5.8** under the actions of loads combined as per **Table 6** and **6.4**.

9.3.2 Design strength capacities

The design strength capacities of sections of concrete members shall be those corresponding to the limit states of:

- a) strain specified in **9.3.2.3** and **9.3.2.4** when estimating axial force and bending moment capacities, respectively; and
- b) stress specified in **9.3.2.5** and **9.3.2.6** when estimating shear force and torsion moment capacities, respectively.

All concrete members provided with steel reinforcement bars, or steel reinforcement bars and prestressing steel, and subjected to:

- a) axial force and bending moment shall be designed to behave in a ductile manner; and

- b) shear force and torsion (if any) shall be designed to behave in a ductile manner (with yielding of reinforcing steel in tension and not with crushing of concrete in compression).

The provisions given hereunder shall be applicable to shallow concrete members. In deep concrete members, special calculations shall be performed for analysis and design, including the use of the strut and tie method or the prescriptive method in their analysis and design in accordance with 8.3.

9.3.2.1 Assumptions

The following assumptions shall be made when estimating the design axial load capacity P_L and design bending moment capacity M_L of shallow concrete members that are reinforced with reinforcing steel or prestressing steel:

- a) Plane sections normal to the axis of bending before bending, remain plane and normal to the axis of bending even after bending.
- b) The tensile strength of the concrete is ignored.
- c) The characteristic bending compressive strength of concrete is $0.68f_{ck}$, and the design bending compressive strength is $(0.68f_{ck})/\gamma_M = 0.45f_{ck}$, where the partial safety factor γ_M is 1.5 for concrete in compression.
- d) The monotonic design stress-strain ($f_c - \varepsilon_c$) curve of concrete in compression shall be taken as specified in 9.2.6.4.
- e) The characteristic tensile strength of:
 - 1) reinforcing steel is f_y , and the design tensile strength is $(f_y)/\gamma_M = 0.87f_y$, where the partial safety factor γ_M is 1.15 for steel reinforcement bars in tension and compression; and
 - 2) prestressing steel is f_{yp} , and the design tensile strength is $(f_{yp})/\gamma_M = 0.87f_{yp}$, where the partial safety factor γ_M is 1.15 for prestressing steel in tension.
- f) The monotonic design stress-strain curve:
 - 1) ($f_s - \varepsilon_s$) curve of reinforcing steel in tension and compression shall be taken as specified in 9.2.6.5, and
 - 2) ($f_{sp} - \varepsilon_{sp}$) curve of prestressing steel in tension shall be taken as specified in 9.2.6.6.
- g) The limiting strain in:
 - 1) Reinforcing steel in the outermost layer in compression or tension is:

$$\varepsilon_{s,\text{lim}} = 0.0020 + \frac{0.87 f_y}{E_s} ; \text{ and}$$

2) Prestressing steel in the outermost layer in tension is:

$$\varepsilon_{sp,\text{lim}} = 0.0020 + \frac{0.87 f_{yp}}{E_{sp}} .$$

h) The limiting strain $\varepsilon_{c,\text{lim}}$ in concrete shall be taken as below:

$$\varepsilon_{c,\text{lim}} = \varepsilon_{cu} .$$

9.3.2.2 Minimum eccentricity

9.3.2.2.1 All compression members (columns or structural walls) shall be designed for at least the bending moments arising out of the minimum eccentricities of the axial load for bending about each of the two axes of the cross-section of the member, given by:

a) For bending about the X axis perpendicular to the dimension D of the cross-section of the member,

$$\text{a. } e_{X,\text{min}} \geq \text{Max} \left(\frac{l_{u,X}}{500} + \frac{D}{30}; \frac{D}{20}; 20\text{mm} \right),$$

where $l_{u,X}$ is the unsupported lengths for bending about X-axis; and

b) For bending about the Y axis perpendicular to the dimension b of the cross-section of the member,

$$e_{Y,\text{min}} = \text{Max} \left(\frac{l_{u,Y}}{500} + \frac{b}{30}; \frac{b}{20}; 20\text{mm} \right),$$

where $l_{u,Y}$ is the unsupported lengths for bending about Y-axis.

9.3.2.2.2 All structural walls shall be designed for the additional eccentricity $e_{a,\text{wall}}$ due to slenderness effect in the out-of-plane direction given by:

$$e_{a,\text{wall}} = 2 \left(\frac{H_{we}}{2500} \right),$$

where H_{we} is the effective height of structural wall for bending about the minor axis.

9.3.2.2.3 The unsupported length $l_{u,X}$ and $l_{u,Y}$ (for bending about the two axes of the cross-section) of a compression member shall be taken as the clear distance between end restraints, except as below:

a) *Moment frame structures with beams restrained laterally by RC slabs*

Clear distance between the floor (below) and the soffit of the shallower beam (at the next higher floor level above) framing into the column in each direction;

b) *Moment frame structures with beams restrained laterally by RC beams with RC brackets at the beam-column joints –*

Clear distance between the floor (below) and the bottom edge of the bracket (at the next higher floor level above), provided the width of the bracket is at least that of the beam strut and the width is at least half that of the column;

c) *Braced frame structures*

Clear distance between consecutive braces (one below and one above) in each vertical plane, provided each brace is an adequate support ensured by:

- 1) Brace having adequate cross-section dimension,
- 2) Brace being anchored to restrain the column against lateral deflection, and
- 3) The angle between vertical planes through the braces is in the range 60° – 120° , when two such struts meet the column at about the same level; and

d) *Flat slab structures*

Smaller of the clear distances between the floor (below) and the lower extremity of the capital, the drop panel and the slab (above).

9.3.2.3 *Design limiting axial force capacity P_L*

9.3.2.3.1 *Limiting strains*

A concrete member shall be said to have reached its limit state at a section under the

action of axial force, when at a section:

a) *Reinforced Members:*

- 1) *Under compression* – concrete in compression and reinforcing steel in tension reach the *limiting strain* $\varepsilon_{c,lim}$; and
- 2) *Under tension* – reinforcing steel in tension reaches the limiting strain $\varepsilon_{s,lim}$;

b) *Prestressed Members:*

- 1) *Under compression* – concrete in compression, reaches the *limiting strain* $\varepsilon_{c,lim}$; and
- 2) *Under tension* – prestressing steel in tension reaches the limiting strain $\varepsilon_{s,lim}$; the contribution of reinforcing steel to P_L shall be neglected.

And, the axial force resisted by the concrete member when the member reaches the said limit state at a section shall be taken as the design axial force capacity P_L of that section.

9.3.2.3.2 Pedestals

The design limiting axial force capacity P_L of a RC pedestal [9.2.3.1(c)(i)] in compression shall be taken as:

$$P_L = 0.45f_{ck}A_g + (f_{sc} - 0.45f_{ck})A_s.$$

9.3.2.3.3 Short columns

The design limiting axial force capacity of a RC short column [9.2.3.1(c)(ii)]:

$$P_L = \begin{cases} \eta(0.45f_{ck}A_g + (f_{sc} - 0.45f_{ck})A_s) & \text{Compression} \\ \eta f_{st}A_s & \text{Tension} \end{cases},$$

and the design limiting axial force capacity of a prestressed concrete short column [9.2.3.1(c)(ii)]:

$$P_L = \begin{cases} \eta(0.45f_{ck}A_g + (f_{sc} - 0.45f_{ck})A_s - (f_{spi} - f_{spc})A_{sp}) & \text{Compression} \\ \eta(f_{spt} - f_{spi})A_{sp} & \text{Tension} \end{cases},$$

where

A_g = gross area of the cross-section in compression,

A_s = total area of longitudinal reinforcing steel,

A_{sp} = area of longitudinal prestressing steel in compression,

f_{ck} = characteristic strength of concrete,

f_{sc} = design stress in steel corresponding to an axial strain of $\varepsilon_{c,lim}$,

f_{spc} = stress in the prestressing steel (including the effect of losses) corresponding to an axial strain of $\varepsilon_{c,lim}$,

f_{spi} = initial stress in the prestressing steel (including the effect of losses),

f_{spt} = stress in the prestressing steel (including the effect of losses) corresponding to an axial strain of $\varepsilon_{c,lim}$ and

η = 0.90, the eccentricity factor to reduce the axial force capacity in view of the requirement to design for a minimum eccentricity of 5 percent of dimension of the member perpendicular to the axial of bending.

9.3.2.3.4 Long columns

A RC or PSC long column [9.2.3.1(c)(iii)] in compression or tension shall be treated as a beam-column for the purpose of this standard. Hence, its design axial force capacity P_L shall not be estimated independently. It shall be designed for the combined action of axial force – bending moment as specified in 9.3.2.7.

9.3.2.4 Design limiting bending moment capacity M_L

The design limiting bending moment capacity M_L of RC flexural members of rectangular cross-section shall be estimated as specified hereunder. When other cross-section shapes are used, the cross-section shall be discretised into a number of fibers (corresponding separately to reinforcing steel bars and concrete) parallel to the axis of bending, and M_L of those cross-sections shall be ascertained by same the procedure.

9.3.2.4.1 Limiting strains

A concrete member shall be said to have reached its limit state at a section under the action of bending moment, when at a section:

- a) *Reinforced Members*: either concrete or reinforcing steel reaches the corresponding limiting strain $\varepsilon_{c,lim}$ or $\varepsilon_{s,lim}$, respectively; and

- b) *Prestressed Members*: either concrete or prestressing steel reaches the corresponding limiting strain $\varepsilon_{c,\text{lim}}$ or $\varepsilon_{sp,\text{lim}}$, respectively.

And, the bending moment resisted by the concrete member when the member reaches the said limit state at a section shall be taken as the design bending moment capacity M_L of that section.

9.3.2.4.2 Slender beams

RC slender beams, slabs, footings and rafts shall be designed to be under-reinforced with steel reaching the limiting strain in tension before concrete reaches the limiting strain in compression. The design bending moment capacity M_L of RC beams shall be obtained by solving the following set of governing nonlinear equations:

- Equilibrium of forces and moment at a section,
- Compatibility of strains at a section, and
- Constitutive Law of constituent materials.

Because the constitutive law of concrete is nonlinear, the solutions shall be obtained iteratively.

9.3.2.4.2.1 The estimation of limiting moment capacity M_L of a doubly-reinforced rectangular RC section designed to be under-reinforced shall be as given hereunder (Fig. 42).

- Equilibrium of forces and moment at a section*

Force Equilibrium:

$$f_{c,\text{avg}}bx_L + (f_{sc} - f_{csc})A_{sc} = f_{yD}A_{st} \quad (1)$$

Moment Equilibrium:

$$M_L = f_{yD}A_{st}(d - x_L) + f_{c,\text{ave}}bx_L(x_L - x_{CG}) + (f_{sc} - f_{csc})A_{sc}(x_L - d'') \quad (2)$$

- Compatibility of strains at a section*

$$\varepsilon_c = \left(\frac{x_L}{d - x_L} \right) \varepsilon_{st,\text{lim}} \quad (3)$$

$$\varepsilon_{sc} = \left(\frac{x_L - d''}{d - x_L} \right) \varepsilon_{st,\text{lim}} \quad (4)$$

$$\varepsilon_{csc} = \left(\frac{x_L - d''}{d - x_L} \right) \varepsilon_{st,lim} \quad (5)$$

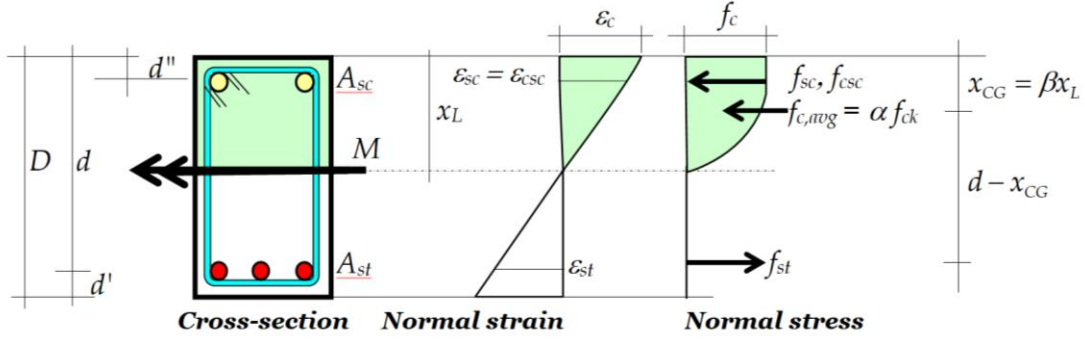


FIG. 42 STRAIN AND STRESS DISTRIBUTIONS ACROSS A DOUBLY-REINFORCED RECTANGULAR RC SECTION

c) *Constitutive law of constituent materials*

$$f_{c,avg}(\varepsilon_c) = \begin{cases} 0.45 f_{ck} \left[\left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ 0.45 f_{ck} \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases} \quad (6)$$

$$f_{sc}(\varepsilon_{sc}) = E_s \varepsilon_{sc} \quad (7)$$

$$f_{csc}(\varepsilon_{csc}) = \begin{cases} 0.45 f_{ck} \left[2 \left(\frac{\varepsilon_{csc}}{\varepsilon_{cp}} \right) - \left(\frac{\varepsilon_{csc}}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_{csc} \leq \varepsilon_{cp} \\ 0.45 f_{ck} & \text{for } \varepsilon_{cp} < \varepsilon_{csc} \leq \varepsilon_{cu} \end{cases} \quad (8)$$

For the section to be under-reinforced, strain ε_{st} in reinforcing steel on the tension side shall be the limiting strain $\varepsilon_{st,lim}$ specified, and that in the extreme fiber of concrete less than the limiting strain $\varepsilon_{c,lim}$, where

$$\varepsilon_{st,lim} = 0.0020 + \frac{f_y}{\gamma_L E_s}, \text{ and} \quad (9)$$

$$\varepsilon_{c,lim} < \varepsilon_{cu}. \quad (10)$$

Thus, the limiting depth x_L of neutral axis shall be less than the balanced depth x_{bal} of neutral axis, where x_{bal} is given by:

$$x_{bal} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{st,lim}} d, \quad (11)$$

Therefore, x_L shall be estimated as per the iteration scheme below:

Then, the limiting moment capacity M_L of the Member shall be given by:

$$\frac{M_L}{f_{ck} b d^2} = \alpha (1 - \beta) \left(\frac{x_L}{d} \right)^2 + \left(\frac{f_{yD}}{f_{ck}} \right) \left(\frac{A_{st}}{b d} \right) \left(1 - \frac{x_L}{d} \right) + \left(\frac{f_{sc} - f_{csc}}{f_{ck}} \right) \left(\frac{x_L}{d} - \frac{d''}{d} \right) \left(\frac{A_{sc}}{b d} \right) \quad (12)$$

where

$$\alpha = \begin{cases} 0.45 \left[\left(\frac{\varepsilon_{cL}}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_{cL}}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_{cL} \leq \varepsilon_{cp} \\ 0.45 \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_{cL}} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_{cL} \leq \varepsilon_{cu} \end{cases}, \quad (13)$$

$$\beta = \begin{cases} \left[\frac{1}{3} - \frac{1}{12} \left(\frac{\varepsilon_{cL}}{\varepsilon_{cp}} \right) \right] & \text{for } 0 \leq \varepsilon_{cL} \leq \varepsilon_{cp} \\ \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cL}}{\varepsilon_{cp}} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_{cL} \leq \varepsilon_{cu} \end{cases}. \quad (14)$$

9.3.2.4.3 Slabs, footings and rafts

The design bending moment capacity M_L of RC slabs, footings and rafts shall be estimated by the same method as that in RC beams, but considering unit width of the slab as width b of the beam for the purpose of calculations.

9.3.2.4.4 Prestressed concrete beams – bonded tendons

- a) Even PSC beams shall be designed to behave in a ductile manner with prestressing steel reaching the limiting strain in tension before concrete reaching the limiting strain in compression. The effect of prestressing steel alone shall be considered in the estimation of limiting moment capacity M_L of prestressed beams; the contribution to M_L shall be neglected of the untensioned longitudinal reinforcing steel.

- b) The limiting bending moment capacity M_L at a section of a PSC beam with bonded tendons also shall be obtained (as in slender beams) by solving iteratively the following nonlinear equations:
- 1) Equilibrium of forces and moment of forces at a section,
 - 2) Compatibility of strains at a section, and
 - 3) Constitutive law of constituent materials.
- c) Because the constitutive law of concrete is nonlinear, the solutions shall be obtained iteratively. Also, since contribution to M_L of reinforcing steel is neglected, the PSC section shall be treated as a singly-reinforced section.
- d) The limiting moment capacity M_L of a bonded prestressed rectangular section designed to be under-reinforced shall be estimated as given hereunder (Fig. 43).

1) *Equilibrium of forces and moment at a section*

$$\text{Force Equilibrium: } f_{c,avg} b x_L = f_{pD} A_{sp} \quad (17)$$

$$\text{Moment Equilibrium: } M_L = f_{pD} A_{sp} (d - x_{CG}) \quad (18)$$

2) *Compatibility of strains at a section*

$$\varepsilon_c = \left(\frac{x_L}{d - x_L} \right) \varepsilon_{sp,lim} \quad (19)$$

3) *Constitutive law of constituent materials*

$$f_{c,avg}(\varepsilon_c) = \begin{cases} 0.45 f_{ck} \left[\left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ 0.45 f_{ck} \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases} \quad (20)$$

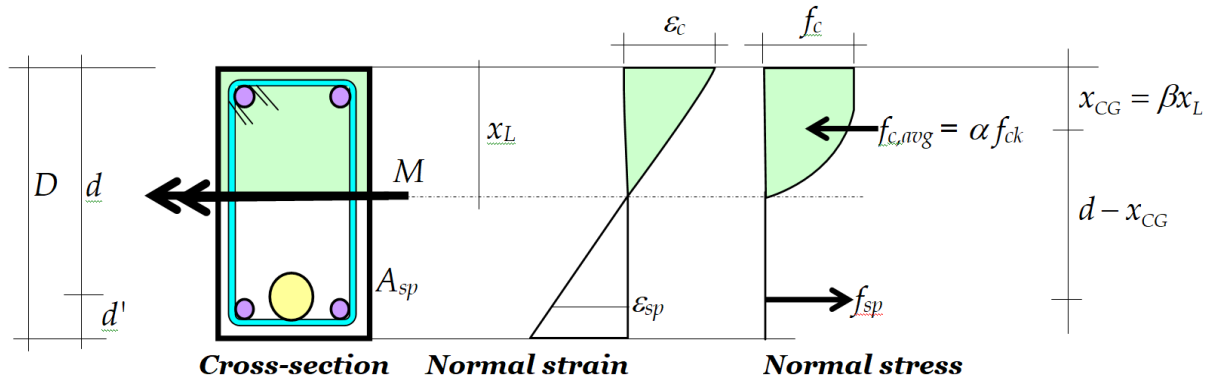


FIG. 43 STRAIN AND STRESS DISTRIBUTIONS ACROSS A BONDED PRESTRESSED RECTANGULAR SECTION

- e) For the bonded PSC section to be under-reinforced, strain ε_{sp} in reinforcing steel on the tension side shall be the limiting strain $\varepsilon_{sp,lim}$ specified, and that ε_c in the extreme fiber of concrete less than the limiting strain $\varepsilon_{c,lim}$, where

$$\varepsilon_{sp,lim} = 0.0020 + \frac{f_{yp}}{\gamma_L E_p}, \text{ and} \quad (21)$$

$$\varepsilon_{c,lim} < \varepsilon_{cu}. \quad (22)$$

But, to begin with, the limiting depth x_L of neutral axis shall be taken as the balanced depth x_{bal} of neutral axis given by:

$$x_{bal} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sp,lim}} d, \quad (23)$$

and x_L estimated by the iteration scheme given below:

$x_{assumed}$	ε_c	$f_{c,avg}(\varepsilon_c)$	$x_{estimated}$
mm	mm / mm	MPa	mm
	Eq.(19)	Eq.(20)	Eq.(17)
x_{bal}			
...			
...			
...			x_L

Then, the limiting moment capacity M_L of the member shall be given by:

$$\frac{M_L}{f_{ck} b d^2} = \alpha \left[1 - \beta \left(\frac{x_L}{d} \right) \right] \left(\frac{x_L}{d} \right) \quad (24)$$

where

$$\alpha = \begin{cases} 0.45 \left[\left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ 0.45 \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases}, \text{ and} \quad (25)$$

$$\beta = \begin{cases} \left[\frac{1}{3} - \frac{1}{12} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) \right] & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ \left[1 - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases} . \quad (26)$$

9.3.2.4.5 Prestressed concrete beams – unbonded tendons

9.3.2.4.5.1 The limiting bending moment capacity M_L at a section of a PSC beam with unbonded prestressing steel shall be obtained by:

- a) Considering stress f_{sp} in unbonded prestressing steel after losses at the section considered, and associated total strain ε_{sp} in unbonded prestressing steel as known at that section (Fig. 44) as given by:

$$\varepsilon_{sp} \approx \varepsilon_0 + 4 \left(\frac{e_M}{L} \right) \left(\frac{\delta}{L} \right),$$

where

ε_0 = initial strain in prestressing steel after losses,

e_M = eccentricity of unbonded tendon with respect to the anchorage at the end,

δ = expected Maximum deflection under the loads applied as per load combinations specified in Table 3 for strength criteria, and

L = center to center span between supports of the prestressed member;
and

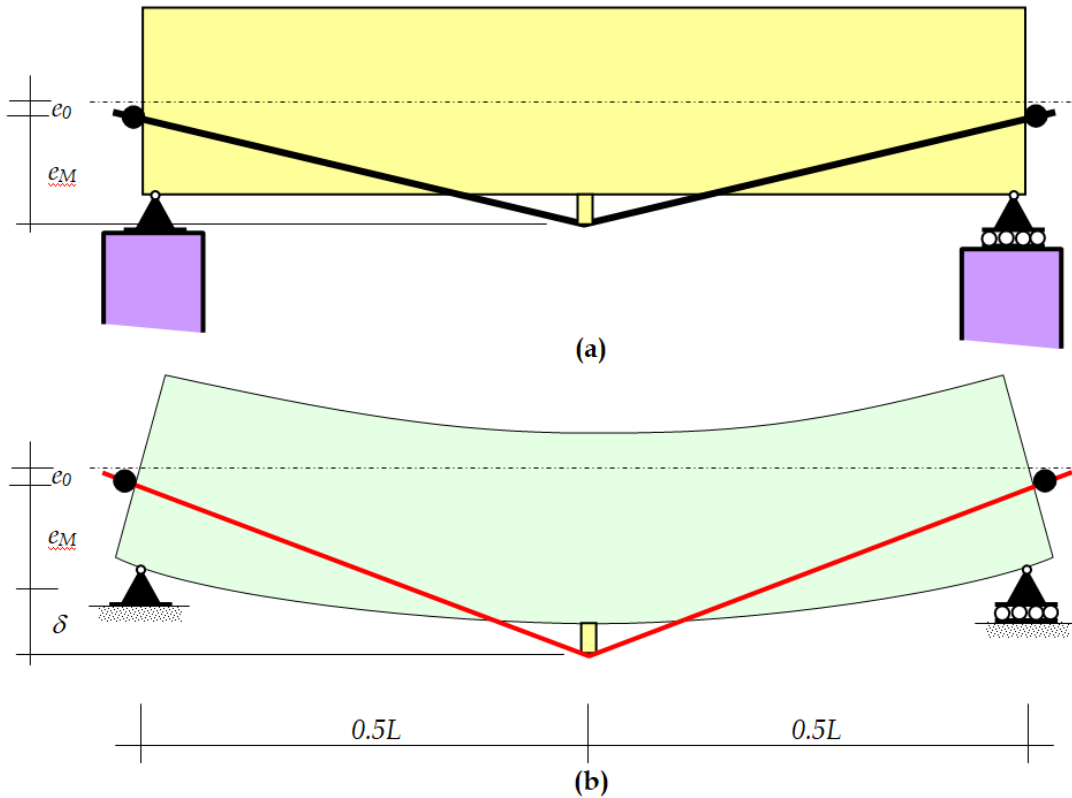


FIG. 44 PARAMETERS RELATED TO PROFILE OF PRESTRESSING STEEL IN AN UNBONDED PRESTRESSED MEMBER: (A) BEFORE DEFORMATION, AND (B) AFTER DEFORMATION

- b) Using the nonlinear system of governing equations namely equilibrium of forces at a section, and constitutive law of constituent materials.

9.3.2.4.5.2 The limiting moment capacity M_L of an unbonded prestressed rectangular section designed to be under-reinforced shall be estimated as given hereunder (Fig. 45).

- a) *Equilibrium of forces and moment at a section*

$$\text{Force Equilibrium} : f_{c,avg} b x_L = f_{sp} A_{sp} \quad (27)$$

$$\text{Moment Equilibrium} : M_L = f_{sp} A_{sp} (D_p - x_{CG}) \quad (28)$$

- b) *Compatibility of strains at a section*

$$\epsilon_c = \left(\frac{x_L}{D_p - x_L} \right) \epsilon_{sp} \quad (29)$$

c) *Constitutive Law of constituent materials*

$$f_{c,avg}(\varepsilon_c) = \begin{cases} 0.45 f_{ck} \left[\left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ 0.45 f_{ck} \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases} \quad (20)$$

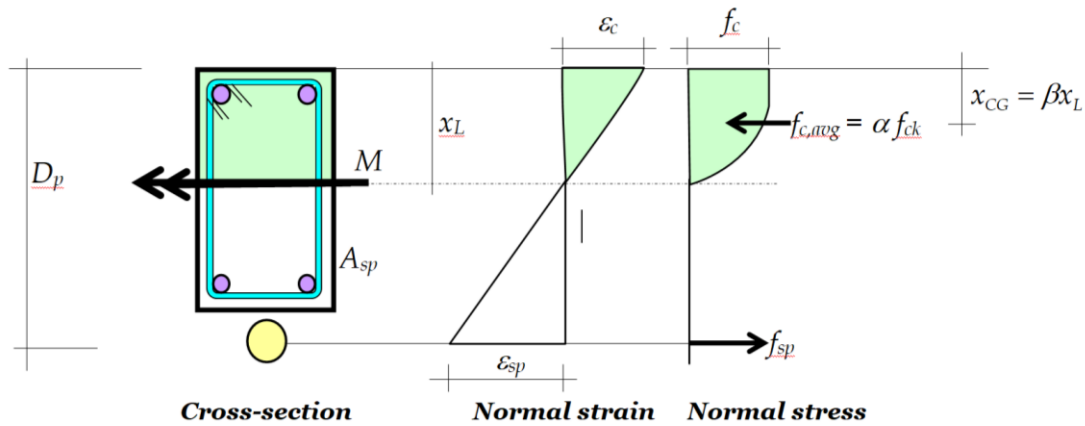


FIG. 45 STRAIN AND STRESS DISTRIBUTIONS ACROSS AN UNBONDED PRESTRESSED RECTANGULAR SECTION

9.3.2.4.5.3 For the PSC section to be under-reinforced, ε_{sp} and hence f_{sp} are known. Also, strain ε_c in the extreme fiber of concrete on the compression side shall be less than limiting strain $\varepsilon_{c,lim}$. But, to begin with, ε_c shall be taken as the limiting strain ε_{cu} , and x_L estimated by the iteration scheme given below:

$\varepsilon_{c,estimated}$	$f_{c,avg}(\varepsilon_c)$	x_L	$\varepsilon_{c,estimated}$
mm / mm	MPa	mm	mm / mm
	Eq.(20)	Eq.(27)	Eq.(29)
ε_{cu}			
...			
...			
...			ε_c

Once ε_c is solved, the design is acceptable only if $\varepsilon_c < \varepsilon_{cu}$:

- a) If $\varepsilon_c \leq \varepsilon_{cu}$, β shall be estimated using Eq.(26), and then x_{CG} using $x_{CG} = \beta x_L$. And, finally M_L shall be estimated using Eq.(28); and
- b) If $\varepsilon_c > \varepsilon_{cu}$, the cross-section geometry, grade of concrete, and amount of prestressing steel shall be revised.

9.3.2.4.6 Columns and structural walls

RC columns and structural walls shall be designed for combined effects of design axial force P_D and design bending moment M_D considering $P_L - M_L$ interaction of the member (as specified in 9.3.2.7), and not just for the effect of design bending moment M_D . When structural walls are provided with boundary elements, the said calculations shall be performed considering the full cross-section as a single entity; the cross-section shall not be considered separately as web and boundary elements and principle of superposition applied.

9.3.2.5 Design shear force capacity V_L

A concrete member shall resist the design shear force demand V_F at the critical section by both concrete and transverse reinforcement steel.

9.3.2.5.1 Limiting stress

Under the action of shear force, at the critical section of a reinforced concrete or prestressed concrete member:

a) Contribution of concrete

Concrete shall be assumed to have reached its limiting stress of τ_c , the design shear strength of concrete; and

b) Contribution of transverse reinforcing steel

Transverse reinforcing steel shall be assumed to have reached its limiting stress of $0.87 f_y$, the design tensile strength of reinforcing steel.

And, the shear force resisted by the concrete member when the member reaches these said limiting stresses simultaneously at a section shall be taken as the design shear force capacity V_L of that section. In flanged sections, only the width of the web shall be

considered in the estimation of the design shear force capacity V_L of a RC beam, column and structural wall.

Shear reinforcement shall be provided only in the form of vertical stirrups.

9.3.2.5.2 RC beams and columns

9.3.2.5.2.1 Shear reinforcement shall be provided as per the following:

- a) When $0 < \tau_D \leq \tau_C$: Minimum shear reinforcement shall be provided as specified in **9.2.4.1(b)**,
- b) When $\tau_C < \tau_D \leq \tau_{C,\max}$: Shear reinforcement shall be provided as specified hereunder, and
- c) When $\tau_{C,\max} < \tau_D$: The member shall be re-sized to increase its shear area,

where

τ_D = nominal design shear stress demand at a RC section

$$= \frac{V_D \pm \frac{M_D \tan \beta}{d}}{bd}.$$

in which

M_D = design bending moment at the critical section for shear, and

β = Angle of inclination of the compression surface of the beam in tapered beams, and taken as 0° in prismatic beams.

In the above expression, the negative sign applies when the bending moment M_D increases numerically in the same direction as the effective depth d increases, and the positive sign when it decreases.

9.3.2.5.2.2 The design shear force capacity V_L of a RC beam or column shall be estimated as:

$$V_L = V_{LC} + V_{LS},$$

where

$$\begin{aligned} V_{LC} &= \text{Contribution of concrete to } V_L \\ &= \tau_C b d, \end{aligned}$$

$$\begin{aligned} V_{LS} &= \text{Contribution of reinforcing steel to } V_L \\ &= 0.87 f_{yV} A_{SV} \left(\frac{d}{s_V} \right)_{INT}, \end{aligned}$$

in which

$$\begin{aligned} A_{SV} &= \text{total cross-sectional area of transverse steel reinforcement} \\ &= N_{SV} \left[\pi \left(\frac{\phi_{SV}}{2} \right)^2 \right], \end{aligned}$$

d = effective depth of the member,

s_V = spacing of the stirrups along the length of the member,

f_{yV} = characteristic strength of the bars used as stirrups,

N_{SV} = number of legs in transverse reinforcement at each section of the member,

ϕ_{SV} = diameter of the bar used as transverse reinforcement,

$(...)_{INT}$ = integral value of the number,

τ_C = design shear strength of concrete as specified in **9.2.6.1(d)**, and

$\tau_{C,max}$ = maximum design shear stress of concrete as specified in **9.2.6.1(d)**.

9.3.2.5.3 RC structural walls

The design shear force capacity V_L of a RC structural wall shall be estimated as:

$$V_L = V_{LC} + V_{LS},$$

where

$$\begin{aligned} V_{LC} &= \text{Contribution of concrete to } V_L \\ &= (0.8 L_w) t_w \tau_{LC,wall}, \end{aligned}$$

$$V_{LS} = \text{Contribution of reinforcing steel to } V_L$$

$$= 0.87 f_{yV} A_{SV} \left(\frac{0.8 L_w}{s_V} \right)_{INT},$$

in which

A_{SV} = Total cross-sectional area of transverse steel reinforcement

$$= N_{SV} \left[\pi \left(\frac{\phi_{SV}}{2} \right)^2 \right],$$

L_w = Length of the wall,

s_V = Spacing of the horizontal stirrups measured along the height of the wall,

f_{yV} = Characteristic strength of the bars used as stirrups,

N_{SV} = Number of legs in transverse reinforcement at each section of the member,

ϕ_{SV} = Diameter of the bar used as transverse reinforcement,

$(\dots)_{INT}$ = Integral value of the number,

$\tau_{Lc,wall}$ = *Design Shear Strength* of concrete in structural wall as specified in **9.2.6.1(d)**, and

$(\tau_{Lc,wall})_{max}$ = *Maximum Design Shear Stress* of concrete in structural wall as specified in **9.2.6.1(d)**.

9.3.2.5.4 Prestressed members

The design shear force capacity V_L of a PSC member shall be taken as smaller of shear force capacity obtained at two levels, namely design shear force capacity V_{LUC} of uncracked section and design shear force capacity V_{LCR} of cracked section, and shear reinforcement provided accordingly. Also, the prestressing steel shall not be considered when estimating V_L ; only the contribution of the transverse steel untensioned reinforcing steel provided in such members shall be considered.

a) Uncracked Section Design Shear Force Capacity

The design shear force capacity V_{LUC} of an uncracked PSC member shall be:

$$V_{LUC} = 0.67bD\sqrt{f_t^2 + (0.8f_{cp})f_t} + 0.87f_{yp}A_{sp} \cos \alpha_p ,$$

where

- b = breadth of the member, which shall be taken as breadth b_w of web in a flanged section,
- D = overall depth of the member,
- f_t = maximum principal tensile (taken as positive) stress given by $0.24\sqrt{f_{ck}}$, where f_{ck} is the characteristic compressive strength of concrete, and
- f_{cp} = compressive stress (taken as positive) at centroidal axis due to prestress,
- f_{yp} = characteristic strength of prestressing steel,
- A_{sp} = area of prestressing steel,
- α_p = angle of prestressing steel tendon with the longitudinal horizontal axis of the member.

Here, in the flanged members whose centroidal axis lies within the flange, the principal tensile stress shall be limited to $0.24\sqrt{f_{ck}}$ at the intersection of the flange and the web.

b) *Cracked section design shear force capacity*

The design shear force capacity V_{LCR} of the PSC member shall be estimated as:

$$V_{LCR} = V_{LCR,C} + V_{LCR,S} ,$$

where

- $V_{LCR,C}$ = Contribution of compression concrete to V_L

$$= \left(1 - 0.55 \frac{f_{pe}}{f_{yp}} \right) \tau_C b d + \left(\frac{V_D}{M_D} \right) M_0 ,$$

$$\geq (0.1\sqrt{f_{ck}}) b d ,$$
- $V_{LCR,S}$ = Contribution of reinforcing steel to V_L

$$= 0.87 f_{yV} A_{SV} \left(\frac{d}{s_V} \right)_{INT},$$

in which

f_{pe} = effective prestress after all losses have occurred, which shall not be taken as greater than $0.6 f_{yp}$,

f_{yp} = characteristic strength of the prestressing steel used,

V_D = design shear force demand at the section considered as per Table 4,

M_D = design bending moment demand at the section considered as per Table 4,

M_0 = bending moment that produces zero stress in the concrete at the depth d

$$= 0.8 f_{pt} \frac{I}{y},$$

f_{pt} = stress in concrete due to prestressing at centroid of prestressing steel,

d = effective depth of prestressing steel (*that is*, distance from extreme compression fiber to centroid of prestressing tendons) at the section considered,

y = distance of centroid of prestressing steel from centroid of concrete section,

I = second moment of inertia of the concrete section, and

the other quantities are as defined in 8.3.1.5(b).

The value of $V_{LCR,C}$ calculated at a particular section may be assumed to be constant over the distance $d/2$ measured in the direction of increasing moment from that particular section.

Also, when the prestressing tendon is inclined at the section at which V_L is being estimated, the component of prestressing forces along the longitudinal axis of the member alone shall be considered in estimation of V_L .

c) *Transverse shear reinforcement*

Transverse shear reinforcement shall be provided in a PSC section as per the following:

- 1) When $0 < V_D \leq 0.5V_L$: Minimum shear reinforcement shall be provided as specified in **9.2.4.1(b)**,
- 2) When $0.5V_L < V_D \leq \tau_{C,\max}bd$: Area A_{sv} of closed loop shear reinforcement of two legs shall be estimated as:

$$A_{sv} = \frac{V_D - V_L}{0.87f_y \left(\frac{d}{s_v} \right)_{INT}}$$

where

f_{yV} = characteristic strength of the bars used as stirrups,

d = effective depth of member with respect to untensioned steel,

s_V = spacing of the stirrups along the length of the member,

$\left(\frac{d}{s_V} \right)_{INT}$ = integral value of the number, and

$\tau_{C,\max}$ = maximum design shear stress of concrete as specified in **9.2.6.1(d)**.

- 3) When $\tau_{C,\max}bd < V_D$: The member shall be re-sized to increase its shear area.

d) *Punching shear*

Slabs and footings shall be designed to resist effects of both symmetrical and unsymmetrical punching.

The design shear stress demand τ_D at the critical section of a footing or a slab shall be estimated as:

$$\tau_D = \tau_{DS} + \tau_{DU},$$

where

τ_{DS} = design Shear Stress transferred from slab to column or footing to column by symmetric bending of slab or footing [Fig. 46(a)]

$$= \frac{V_{DS}}{(2a_1 + 2a_2)d}, \text{ and}$$

τ_{DU} = design Shear Stress transferred from slab to column or footing to column by unsymmetric bending of slab or footing [Fig. 46(b)] (assumed to vary linearly about the centroid of the critical section)

$$= \frac{(1-\alpha)M_{DU}}{\left(\frac{1}{3}a_1^2 + a_1a_2\right)d},$$

in which

V_{DS} = design shear force transferred from slab to column or footing to column by symmetric bending of slab or footing,

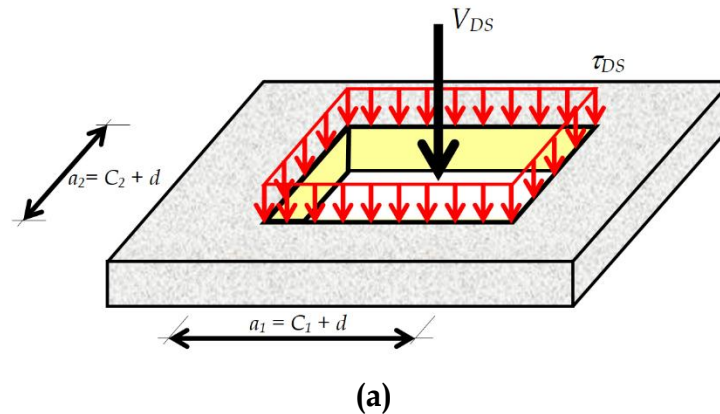
M_{DU} = design bending moment transferred from slab to column or footing to column by unsymmetric bending of slab or footing, arising from unbalanced gravity, wind, earthquake, blast, lateral impact, or other horizontal forces applied on the structure,

a_1 = overall dimension of the critical section for shear in the direction perpendicular to the axis about which moment acts (Fig. 46),
= $c_1 + d$,

a_2 = overall dimension of the critical section for shear in the direction of the axis about which moment acts (Fig. 46),
= $c_2 + d$,

d = effective depth of the slab or footing, and

$$\alpha = \frac{1}{1 + \frac{2}{3} \sqrt{\frac{a_1}{a_2}}}.$$



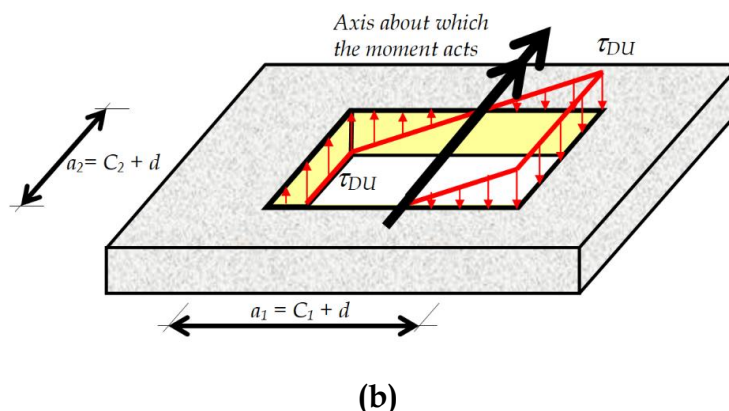


FIG. 46 GEOMETRY AT TRANSFER OF UNBALANCED BENDING MOMENT FROM FLAT SLAB TO COLUMN

9.3.2.6 Design torsional moment capacity T_L

9.3.2.6.1 General

- a) A member in torsion is said to have reached its limit state, when concrete at a section reaches the corresponding design shear strength τ_c of concrete and reinforcing steel stirrup at that section the corresponding characteristic tensile strength f_{yp} .
- b) All the eight assumptions listed in 9.3.2.1 shall be applicable when estimating the design torsional moment capacity also.
- c) In general, where the torsional stiffness (and strength) of members has not been considered in the analysis of the structure, no specific calculations for torsion shall be necessary. Adequate control in torsional cracking shall be assumed to have been provided by the required nominal shear reinforcement. But, where the torsional stiffness (and strength) of members has been considered in the analysis, the members shall be designed for torsion.
- d) The provisions of this shall be applicable only when:
 - 1) Solid rectangular beams with $D > b$,
 - 2) Hollow rectangular beams with $D > b$ and wall thickness $t > b/4$,
 - 3) Solid T-beams and I-beams, whose webs comply with the requirement of $D > b$, and
 - 4) PSC concrete beams have an average intensity of prestress less than $0.3f_{ck}$.

- e) In concrete structures, where torsional moment is required to maintain equilibrium, members shall be designed for torsion in accordance with the provisions given in 9.3.2.6. But, for such indeterminate concrete structures, where torsional moment can be eliminated by releasing redundant restraints, without compromising the safety of the member and thereby of the structure, no specific design for torsional moment is necessary, provided torsional stiffness is neglected in structural analysis when calculating the internal stress-resultants.

9.3.2.6.2 *Limiting stress*

A concrete member shall resist the design torsional moment demand T_F at the critical section by concrete, transverse reinforcement steel and longitudinal steel. Under the action of shear force, at the critical section of a reinforced concrete or prestressed concrete member:

- a) *Contribution of concrete*

Concrete shall be assumed to have reached its limiting stress of τ_c , the design shear strength of concrete; and

- b) *Contribution of transverse reinforcing steel*

Transverse reinforcing steel shall be assumed to have reached its limiting stress of $0.87 f_y$, the design tensile strength of reinforcing steel.

And, the shear force resisted by the concrete member when the member reaches the said limiting stresses simultaneously at a section shall be taken as the design shear force capacity V_L of that section.

Shear reinforcement shall be provided only in the form of vertical stirrups. Also, adequate control of torsional cracking shall be provided by the shear reinforcement as per 9.3.2.5.2.1.

9.3.2.6.3 *Critical section*

Sections located less than a distance d from the face of the support may be designed for the same torsional moment as computed at a distance d , where d is the effective depth of the member.

9.3.2.6.4 RC beams**a) Design for bending moment**

The total design bending moment M_{DT} acting at the critical section shall be:

$$M_{DT} = M_D + M_{DET},$$

where

M_D = design bending moment at the critical section of the member obtained from structural analysis, and

M_{DET} = equivalent design bending moment at the critical section of the member due to torsional moment T_D obtained from structural analysis,

$$= T_D \frac{\left(1 + \frac{D}{b}\right)}{1.7},$$

in which

T_D = design torsional moment at the critical section of the member obtained from structural analysis,

D = larger dimension of the cross-section of the concrete member, and

b = smaller dimension of the cross-section of the concrete member.

Further,

- 1) If $M_{DET} \leq M_D$, then design the member for the total moment M_{DT} acting in the direction of M_D ; and
- 2) If $M_{DET} > M_D$, then design the member for the total moment M_{DT} acting in the direction of M_D and for a moment of $M_{DET2} = (M_D - M_{DET})$ acting in the direction opposite to that of M_D .

b) Design for shear force

- 1) The total design shear force V_{DT} acting at the critical section shall be obtained as:

$$V_{DT} = V_D + V_{DET},$$

where

V_D = design shear force at the critical section of the member obtained from structural analysis, and

V_{DET} = equivalent design shear force at the critical section of the member due to Torsional Moment T_D obtained from structural analysis

$$= 1.6 \frac{T_D}{b},$$

in which

T_D = design torsional moment at the critical section of the member obtained from structural analysis, and

b = smaller dimension of the cross-section of the concrete member.

2) The total shear stress demand τ_{DT} at the critical section shall be obtained as:

$$\tau_{DT} = \frac{V_{DT}}{bd}.$$

3) Shear reinforcement shall be provided as per the following:

i) When $0 < \tau_{DT} \leq \tau_c$: Minimum shear reinforcement shall be provided as per **9.2.4.1(b)**;

ii) When $\tau_c < \tau_{DT} \leq \tau_{c,max}$:

When two legged closed hoops enclosing the corner longitudinal bars are used, the area A_{SV} of cross section of the 2 legs of transverse reinforcement shall be given by:

$$0.87 f_y \left(\frac{d_1}{s_v} \right)_{INT} A_{sv} \geq \left[\frac{T_D}{b_1} + \frac{V_D}{2.5} \right] \geq (\tau_{DT} - \tau_c) bd$$

where

f_{yV} = Characteristic strength of the bars used as stirrups,

d = Effective depth of member with respect to untensioned steel,

s_V = Spacing of transverse reinforcement along the length of the member, and

b_1 = Centre-to-centre distance between corner longitudinal bars in the direction of the width,

d_1 = Centre-to-centre distance between corner longitudinal bars in the direction of the depth,

$\left(\frac{d_1}{s_V} \right)_{INT}$ = Integral value of the number,

iii) When $\tau_{c,\max} < \tau_{DT}$: The member shall be re-sized to increase its shear area.

where

τ_c = design shear strength of concrete as specified in **9.2.6.1(d)**, and

$\tau_{c,\max}$ = maximum design shear stress of concrete as specified in **9.2.6.1(d)**.

9.3.2.6.5 *PSC Beams*

a) *Design for bending moment*

The total design bending moment M_{DT} acting at the critical section shall be:

$$M_{DT} = M_D + M_{DET},$$

where

M_D = design bending moment at the critical section of the member obtained from Structural Analysis, and

M_{DET} = equivalent design bending moment at the critical section of the member due to Torsional Moment T_D obtained from Structural Analysis,

$$= T_D \sqrt{1 + \frac{D}{b}},$$

in which

T_D = design torsional moment at the critical section of the member obtained from structural analysis,

D = larger dimension of the cross-section of the concrete member, and

b = smaller dimension of the cross-section of the concrete member.

Further,

1) If $M_{DET} \leq M_D$, then design the member for the total moment M_{DT} acting in the direction of M_D ; and

2) If $M_{DET} > M_D$, then design the member for the total moment M_{DT} acting in the direction of M_D and for a moment of:

i) $M_{DET2} = (M_D - M_{DET})$ acting in the direction opposite to that of M_D .

ii) $M_{DET3} = M_D \left(1 + \frac{x_1}{2e} \right)^2 \left(\frac{1 + \frac{2b}{D}}{1 + \frac{2D}{b}} \right)$ acting on the same cross-section about

an axis perpendicular to the axis of M_D ,

where

$$\begin{aligned}
 x_i &= \text{Smaller dimension of a closed hoop used as torsional shear reinforcement, and} \\
 e &= \frac{M_D}{V_D}.
 \end{aligned}$$

b) *Design for shear force*

The torsional moment T_{C1} carried by concrete shall be estimated as:

$$T_{C1} = T_C \left(\frac{e}{e + e_C} \right),$$

where

T_C = torsional moment carried by concrete

$$= \sum_{i=1}^{N_R} \left[0.15 b_i^2 D_i \left(1 - \frac{b_i}{3D_i} \right) \lambda_V \sqrt{f_{ck}} \right]$$

$$e = \frac{T_D}{V_D}, \text{ and}$$

$$e_C = \frac{T_C}{V_L},$$

in which

$$\lambda_V = \sqrt{1 + \frac{12 f_{cp}}{f_{ck}}},$$

b_i = smaller dimension of the rectangular part in the cross-section,

D_i = larger dimension of the rectangular part in the cross-section

T_D = design torsional moment demand obtained from structural analysis,

V_D = design shear force demand obtained from structural analysis,

V_L = design Shear Force Capacity obtained from 9.3.2.5, and

N_R = number of rectangles that the cross-section is comprised of.

The shear force V_{C1} carried by concrete shall be estimated as:

$$V_{C1} = V_L \left(\frac{e}{e + e_C} \right),$$

where V_L = design shear force capacity obtained from 9.3.2.5.

When two-legged closed hoops enclosing the corner longitudinal bars are used, the area A_{SV} of cross section of the 2 legs of transverse reinforcement shall be given by:

$$0.87 f_y A_{sv} \left(\frac{d_1}{s_v} \right)_{INT} \geq \text{Max} \left[\left\{ \frac{M_{DT}}{1.7 b_1} \right\}; \{(V_D - V_{C1}) + 2(T_D - T_{C1})\} \right],$$

$$\geq (\tau_{DT} - \tau_c) b d$$

where the various quantities shall be taken as defined before. Also, the above area shall not be less than the minimum transverse reinforcement required as per **9.2.4.1**.

9.3.2.7 Design combined axial force P_L – bending moment M_L capacity

9.3.2.7.1 The sufficiency of resistance of a RC member subjected to both axial force P_L and bending moments M_{Lx} and M_{Ly} about both the axes of the cross-section shall be examined considering the assumptions listed in **9.3.2.1** and complying with the three governing equations of structural mechanics, namely:

- Equilibrium of axial forces and bending moments about the two axes of the cross-section;
- Compatibility of strains across the cross-section; and
- Constitutive law of constituent materials, namely concrete and reinforcing steel.

9.3.2.7.2 Vertical members carrying lateral loads (columns and structural walls) shall be classified as a braced or unbraced member as below using the stability index Q_s defined in **8.2.3**:

- A column shall be considered braced in a given plane, if the structure as a whole is provided by walls or bracing or buttressing designed to resist all lateral forces in that plane, and if its stability index at each storey level satisfies the following inequality:
 $Q_s \leq 0.04$, and
- A column shall be considered unbraced in a given plane, if the structure as a whole is not provided by walls or bracing or buttressing designed to resist all lateral forces in that plane, OR if its stability index satisfies the following inequality at any storey level:
 $Q_s > 0.04$.

9.3.2.7.3 Columns and structural walls shall be designed for results of P_D and M_{DX} & M_{DY} from any of the following:

- a) Detailed geometric nonlinear analysis, or
- b) Linearized geometric nonlinear analysis (also called $P - \Delta$ analysis).

When results from any of the above are unavailable, results of linear analysis shall be used along with the additional bending moments given hereunder. The total bending moments M_{DX}^{TOP} , M_{DX}^{BOT} , M_{DY}^{TOP} and M_{DY}^{BOT} acting about the x-and y- axes of its cross-section at the top and bottom of slender columns shall be approximated by adding additional moments M_{DXA} and M_{DYA} to the bending moment demands obtained by linear structural analysis of the structure subjected to combinations of loads specified in **Table 3** for strength criteria.

9.3.2.7.4 M_{DXA} and M_{DYA} shall be estimated as:

M_{DXA} = Additional bending moment demand owing to lateral sway of the structure acting about the *x-axis* of the cross-section of the column

$$= \text{Max} \left[\frac{P_D D}{2000} \left(\frac{l_{eX}}{D} \right)^2 \kappa; P_D e_{X,\min} \right], \text{ and}$$

M_{DYA} = Additional bending moment demand owing to lateral sway of the structure acting about the *y-axis* of the cross-section of the column

$$= \text{Max} \left[\frac{P_D b}{2000} \left(\frac{l_{eY}}{b} \right)^2 \kappa; P_D e_{Y,\min} \right],$$

in which

P_D = factored design axial load demand on the member,

l_{eX} = average of the effective lengths of all vertical members in the storey when bending about the Y-axis of their cross-sections, when the structure is unbraced, and is effective length of the vertical member when bending about the X-axis of its cross-section, when the structure is braced,

l_{eY} = average of the effective lengths of all vertical members in the storey when bending about the Y-axis of their cross-sections, when the structure is unbraced, and is effective length of the vertical member when bending about the X-axis of its cross-section, when the structure is braced,

D = depth of the cross-section of the member perpendicular to the X-axis of bending,

b = depth of the cross-section of the member perpendicular to the Y-axis of bending,
and

$$\kappa = \frac{P_{L0} - P_D}{P_{L0} - P_{bal}} \leq 1, \text{ and}$$

$e_{X,,\min}$ and $e_{Y,\min}$ are minimum design eccentricities for bending about X- and Y-axes, as specified in 9.3.2.2;

wherein

P_{bal} = design axial load corresponding to the balanced condition of strain across the cross-section, namely concrete reaching a limiting strain of $\varepsilon_{c,\lim}$ corresponding to the grade of concrete, and steel reaching a limiting strain of $\varepsilon_{s,\lim}$ in the outermost layer of tension steel

$$= P_{bal} = \alpha_{\max} f_{ck} bD + \sum_{i=1}^{n_1} (f_{sc,i} - f_{csc,i}) A_{sc,i} + \sum_{j=1}^{n_2} f_{st,j} A_{st,j} \text{ for columns}$$

$$= P_{bal} = \alpha_{\max} f_{ck} t_w L_w + \sum_{i=1}^{n_1} (f_{sc,i} - f_{csc,i}) A_{sc,i} + \sum_{j=1}^{n_2} f_{st,j} A_{st,j} \text{ for structural walls;}$$

$f_{sc,i}$, $f_{csc,i}$ and $f_{st,j}$ in these expressions correspond to the strains associated with the balanced section, which are described hereunder.

9.3.2.7.5 General requirements for columns

In the design of the slender columns, the additional moments given hereunder shall be considered:

a) Unbraced columns

$$M_{DX}^{TOP} \approx M_{DX}^{TOP,L} + M_{DXA},$$

$$M_{DX}^{BOT} \approx M_{DX}^{BOT,L} + M_{DXA}$$

$$M_{DY}^{TOP} \approx M_{DY}^{TOP,L} + M_{DYA}, \text{ and}$$

$$M_{DY}^{BOT} \approx M_{DY}^{BOT,L} + M_{DYA},$$

where

$M_{DX}^{TOP,L}$ and $M_{DX}^{BOT,L}$ = factored bending moment demands at the top and bottom of the vertical member acting about the X-axis of its cross-section obtained

by linear structural analysis of the structure subjected to design loads applied on it; and

$M_{DY}^{TOP,L}$ and $M_{DY}^{BOT,L}$ = factored bending moment demands at the top and bottom of the vertical member acting about the Y-axis of its cross-section obtained by linear structural analysis of the structure subjected to design loads applied on it.

b) *Braced columns (with no concentrated loads acting along its height)*

For the purposes of design of braced columns, no distinction shall be made between moments at the top and bottom of the member; the members shall be designed for the same moment considered to act throughout the height of the member. The member shall be designed for the total moments given below:

The total bending moment demands about two axes of bending of the cross-section shall be estimated as:

$$M_{DX} \approx [Max\{(0.4M_{DX1} + 0.6M_{DX2}); 0.4M_{DX2}\} + M_{DXA}] \geq M_{DX2}, \text{ and}$$

$$M_{DY} \approx [Max\{(0.4M_{DY1} + 0.6M_{DY2}); 0.4M_{DY2}\} + M_{DYA}] \geq M_{DY2},$$

where

$$M_{DX1} = Min\{M_{DX}^{L,TOP}; M_{DX}^{L,BOT}\}$$

$$M_{DX2} = Max\{M_{DX}^{L,TOP}; M_{DX}^{L,BOT}\}$$

$$M_{DY1} = Min\{M_{DY}^{L,TOP}; M_{DY}^{L,BOT}\}$$

$$M_{DY2} = Max\{M_{DY}^{L,TOP}; M_{DY}^{L,BOT}\}$$

The axial load demand P_D and the total bending moment demands M_{DX} and M_{DY} on the columns obtained as above shall be considered as given hereunder.

9.3.2.7.6 Columns under uniaxial bending

Columns under uniaxial bending shall be designed such that the design demand point ($P_D; M_{DX}$) and ($P_D; M_{DY}$) lie inside the strength envelopes provided by the $P_L - M_{LX}$

and $P_L - M_{LY}$ interaction diagrams, respectively. These two interaction diagrams shall be drawn as per the principles laid down in 9.3.2.7.

Alternately, an approximate $P_L - M_L$ interaction diagram can be drawn for 8 strain states at 8 critical points (Table 46 and Fig. 47). For a RC section with steel only on top and bottom faces, the expressions for limiting P_L and M_L are given in Table 47; $\alpha_{\max}$ and $\beta_{\max}$ shall be taken as per Table 48. For a RC section with generalized distribution of longitudinal steel (Fig. 48), the following procedure may be employed:

a) *Equilibrium of Axial Force and Bending Moment at the section*

$$P_L = f_{c,avg} b x_L + \sum_{i=1}^n (f_{sc,i} - f_{csc,i}) A_{sc,i} - \sum_{j=1}^n f_{st,j} A_{st,j} \quad (41)$$

$$M_L = f_{c,avg} b x_L \left[\frac{D}{2} - \beta x_L \right] + \sum_{i=1}^n (f_{sc,i} - f_{csc,i}) A_{sc,i} \left[\frac{D}{2} - d_i^{top} \right] + \sum_{j=1}^n f_{st,j} A_{st,j} \left[\frac{D}{2} - d_j^{bot} \right] \quad (42)$$

b) *Compatibility of Strains at the cross section*

$$\varepsilon_{csc,i} = \left(\frac{x_L - d_i^{top}}{x_L} \right) \varepsilon_c \quad (43)$$

$$\varepsilon_{sc,i} = \left(\frac{x_L - d_i^{top}}{x_L} \right) \varepsilon_c \quad (44)$$

$$\varepsilon_{sti} = \left(\frac{d_i^{bot} - x_L}{x_L} \right) \varepsilon_c \quad (45)$$

c) *Constitutive Law of constituent materials at the section*

$$f_{c,avg} = \begin{cases} 0.45 f_{ck} \left[\left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right)^2 \right] & 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ 0.45 f_{ck} \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right] & \varepsilon_{cp} \leq \varepsilon_c \leq \varepsilon_{cu} \end{cases} \quad (46)$$

$$f_{csc,i} = \begin{cases} 0.45 f_{ck} \left[2 \left(\frac{\varepsilon_{csc,i}}{\varepsilon_{cp}} \right) - \left(\frac{\varepsilon_{csc,i}}{\varepsilon_{cp}} \right)^2 \right] & 0 \leq \varepsilon_{csc,i} \leq \varepsilon_{cp} \\ 0.45 f_{ck} & \varepsilon_{cp} \leq \varepsilon_{csc,i} \leq \varepsilon_{cu} \end{cases} \quad (47)$$

$$f_{sc,i} = \begin{cases} E_s \varepsilon_{sc,i} & 0 \leq \varepsilon_{sc,i} \leq \varepsilon_{syD} \\ 0.87 f_y & \varepsilon_{syD} \leq \varepsilon_{sc,i} \leq \varepsilon_{sr} \end{cases} \quad (48)$$

$$f_{st,i} = \begin{cases} E_s \varepsilon_{sc,i} & 0 \leq \varepsilon_{st,i} \leq \varepsilon_{syD} \\ 0.87 f_y & \varepsilon_{syD} \leq \varepsilon_{st,i} \leq \varepsilon_{sr} \end{cases} \quad (49)$$

$$\alpha = \begin{cases} 0.45 \left[\left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ 0.45 \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases}, \quad (50)$$

$$\beta = \begin{cases} \frac{\left[\frac{1}{3} - \frac{1}{12} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) \right]}{\left[1 - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) \right]} & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ \frac{\left[\frac{1}{2} - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) + \frac{1}{12} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right)^2 \right]}{\left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right]} & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases}. \quad (51)$$

In the above, $f_{c,avg}$ is the average compressive stress in concrete, $f_{csc,i}$ the compressive stress in concrete at the level of layer i of reinforcing steel in compression, $f_{sc,i}$ the compressive stress in reinforcing steel at the layer i , and βx_L the distance of resultant compressive force in concrete from the top fiber. Also, n_1 and n_2 are the numbers of longitudinal bars on the compression and tension sides of the neutral axis of the cross-section.

Table 46 Eight critical states of strain of a RC section only used to draw an approximate Limiting $P_L - M_L$ Interaction Diagram for bending about X-axis
[Clause 9.3.2.7.6]

Sl No.	Point	State of Strain	Top	Bottom
(1)	(2)	(3)	(4)	(5)
i)	A	Uniform compression	ε_{cu}	ε_{cu}
ii)	B	Compressive strain gradient	ε_{cu} at top concrete fiber	ε_{cp} at bottom concrete fiber

iii)	C	Strain gradient	ε_{cu} at top concrete fiber	0 at bottom concrete fiber
iv)	D	Strain gradient	ε_{cu} at top concrete fiber	0 at bottom steel level
v)	E	Balanced strain gradient	ε_{cu} at top concrete fiber	$\varepsilon_{st,lim}$ at bottom steel level
vi)	F	Strain gradient	0 at top steel level	$\varepsilon_{st,lim}$ at bottom steel level
vii)	G	Tensile strain gradient	0 at top concrete fiber	$\varepsilon_{st,lim}$ at bottom steel level
viii)	H	Uniform tension	$\varepsilon_{st,lim}$	$\varepsilon_{st,lim}$

Table 47 Expressions for Limiting Axial Load Capacity P_L and Limiting Bending Moment Capacity M_{LX} in a RC section with top and bottom steel only corresponding to the 8 salient points on its Limiting P-M Interaction Diagram for bending about X-axis
[Clause 9.3.2.7.6]

Poin t	Expressions for P_L and M_{LX}
A	$P_{LA} = 0.45f_{ck}BD + (f_{sc} - 0.45f_{ck})A_{sc} + (f_{st} - 0.45f_{ck})A_{st}$ $M_{LA} = 0$
B	$P_{LB} = 0.45f_{ck}BD + (f_{sc} - 0.45f_{ck})A_{sc} + (f_{st} - 0.45f_{ck})A_{st}$ $M_{LB} = (f_{sc} - 0.45f_{ck})A_{sc}\left[\frac{D}{2} - d''\right] - (f_{st} - 0.45f_{ck})A_{st}\left[\frac{D}{2} - d'\right]$
C	$P_{LC} = \alpha_{\max}f_{ck}BD + (f_{sc} - f_{csc})A_{sc} + (f_{st} - f_{cst})A_{st}$ $M_{LC} = \alpha_{\max}f_{ck}BD\left[\frac{D}{2} - \beta_{\max}D\right] + (f_{sc} - f_{csc})A_{sc}\left[\frac{D}{2} - d''\right] - (f_{st} - f_{cst})A_{st}\left[\frac{D}{2} - d'\right]$
D	$P_{LD} = \alpha_{\max}f_{ck}B(D - d') + (f_{sc} - f_{csc})A_{sc}$ $M_{LD} = \alpha_{\max}f_{ck}B(D - d')\left[\frac{D}{2} - \beta_{\max}(D - d')\right] + (f_{sc} - f_{csc})A_{sc}\left[\frac{D}{2} - d''\right]$
E	$P_{LE} = \alpha_{\max}f_{ck}Bx_L + (f_{sc} - f_{csc})A_{sc} - f_{st}A_{st}$ $M_{LE} = \alpha_{\max}f_{ck}Bx_L\left[\frac{D}{2} - \beta_{\max}x_L\right] + (f_{sc} - f_{csc})A_{sc}\left[\frac{D}{2} - d''\right] + f_{st}A_{st}\left[\frac{D}{2} - d'\right]$
F	$P_{LF} = f_{c,avg}Bd'' - f_{st}A_{st}$ $M_{LF} = f_{c,avg}Bd''\left[\frac{D}{2} - (d'' - \beta x_L)\right] + f_{st}A_{st}\left[\frac{D}{2} - d'\right]$

G	$P_{LG} = -f_{sc}A_{sc} - f_{st}A_{st}$ $M_{LG} = -f_{sc}A_{sc}\left[\frac{D}{2} - d''\right] + f_{st}A_{st}\left[\frac{D}{2} - d''\right]$
H	$P_{LH} = -f_{sc}A_{sc} - f_{st}A_{st}$ $M_{LH} = 0$

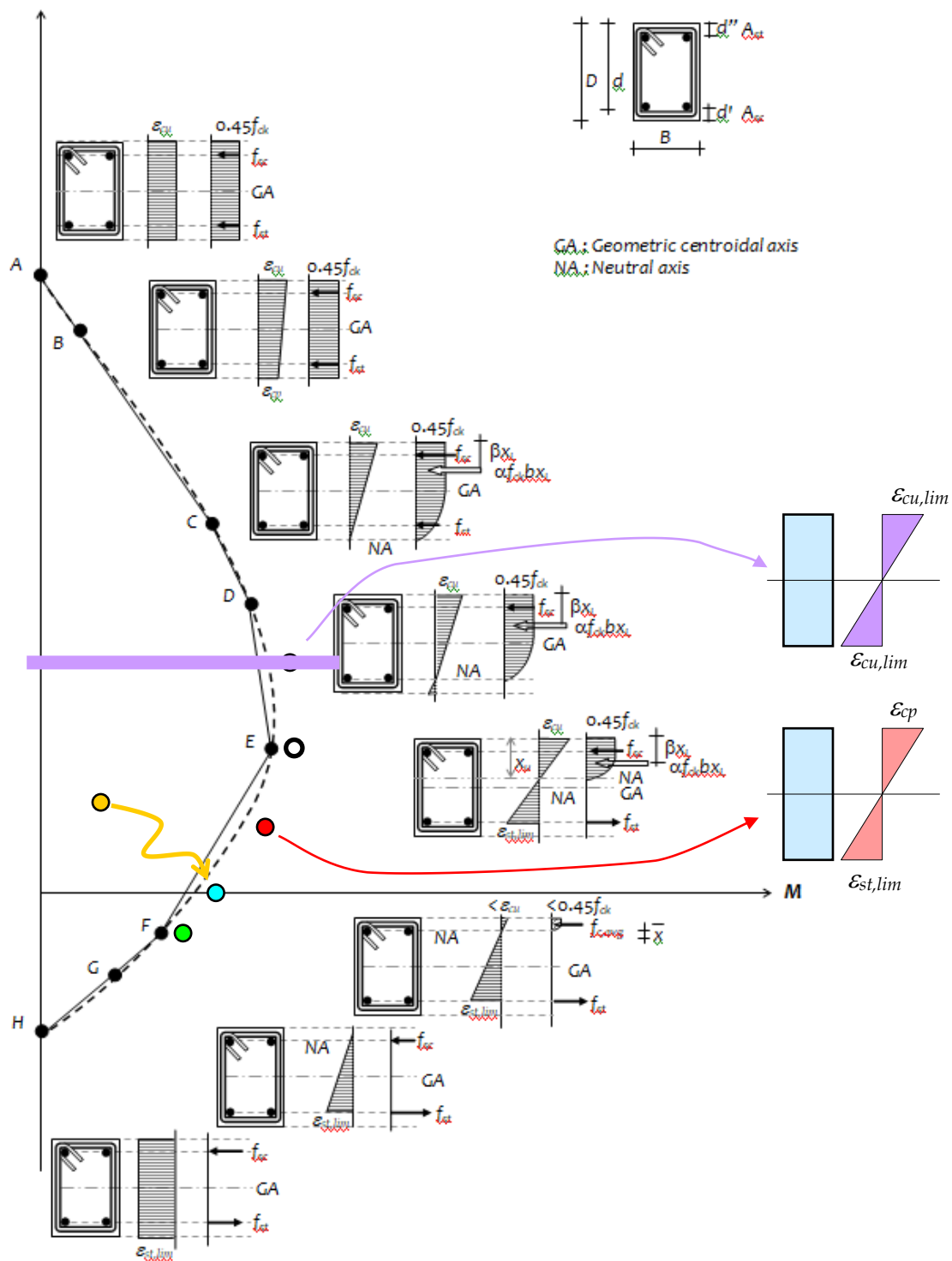


FIG. 47 NORMAL STRAIN AND STRESS DISTRIBUTIONS IN A RC SECTION
CORRESPONDING TO THE EIGHT SALIENT POINTS ON ITS DESIGN P-M
INTERACTION DIAGRAM

Table 48 Full Stress Block properties $\alpha_{\max}$ and $\beta_{\max}$ for different grades of concrete
[Clause 9.3.2.7.6 and Clause 9.3.2.7.8.3]

Sl No.	Grade of Concrete	Full Stress Block Properties	
		$\alpha_{\max}$	$\beta_{\max}$
(1)	(2)	(3)	(4)
i)	M 20	0.364	0.416
ii)	M 25	0.361	0.413
iii)	M 30	0.358	0.411
iv)	M 35	0.355	0.409
v)	M 40	0.352	0.407
vi)	M 45	0.349	0.404
vii)	M 50	0.346	0.402
viii)	M 55	0.342	0.399
ix)	M 60	0.338	0.396
x)	M 65	0.334	0.393
xi)	M 70	0.330	0.391
xii)	M 75	0.325	0.388
xiii)	M 80	0.321	0.386
xiv)	M 85	0.316	0.383
xv)	M 90	0.311	0.380
xvi)	M 95	0.305	0.377
xvii)	M 100 or higher	0.300	0.375

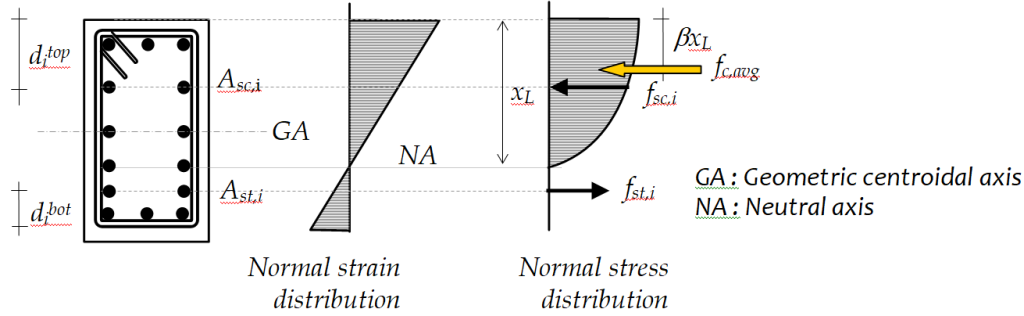


FIG. 48 GENERAL RECTANGULAR RC SECTION WITH REINFORCING STEEL ON ALL FOUR FACES

9.3.2.7.7 Columns under biaxial bending

Columns under biaxial bending shall be designed to satisfy the following inequality:

$$\frac{\sqrt{M_{DX}^2 + M_{DY}^2}}{\left[\frac{1}{\left(\frac{1}{M_{LX0}} \right)^\alpha + \left(\frac{\tan \beta}{M_{LY0}} \right)^\alpha} \right]^{\left(\frac{1}{\alpha} \right)}} < 1.0 ,$$

where

- M_{DX} = design bending moment demand acting about the X-axis of the cross-section of the reinforced concrete member obtained by detailed geometric nonlinear structural analysis of the structure subjected to design loads applied on it;
- M_{DY} = design bending moment demand acting about the Y-axis of the cross-section of the reinforced concrete member obtained by detailed geometric nonlinear structural analysis of the structure subjected to design loads applied on it;
- M_{LX0} = limiting bending moment capacity about the X-axis of the cross-section of the RC member corresponding under the action of pure bending moment (with no axial load on it), estimated as specified in 9.3.2.4,

M_{LY0} = limiting bending moment capacity about the Y-axis of the cross-section of the RC member corresponding under the action of pure bending moment (with no axial load on it), estimated as specified in 9.3.2.4,

$$\tan \beta = \left(\frac{M_{DY}}{M_{DX}} \right), \text{ and}$$

$$\alpha = \begin{cases} 2.5 & \text{if } P_D \leq 0.2P_{L0} \\ 2.0 + 0.5 \left[\frac{(P_D - 0.2P_{L0})}{0.6P_{L0}} \right] & \text{if } 0.2P_{L0} < P_D < 0.8P_{L0} , \\ 2.0 & \text{if } 0.8P_{L0} \leq P_D \end{cases}$$

in which

P_{L0} = limiting axial load capacity of the RC section under the action of pure axial load (with no bending moment on it), estimate as specified in 9.3.2.3.

Existing Methodology for columns under biaxial loading

Columns under biaxial bending shall be designed to satisfy the following inequality:

$$\left(\frac{M_{DX}}{M_{LX0}} \right)^\alpha + \left(\frac{M_{DY}}{M_{LY0}} \right)^\alpha \leq 1.0 ,$$

where

M_{DX} = design bending moment demand acting about the X-axis of the cross-section of the reinforced concrete member obtained by detailed geometric nonlinear structural analysis of the structure subjected to design loads applied on it;

M_{DY} = design bending moment demand acting about the Y-axis of the cross-section of the reinforced concrete member obtained by detailed geometric nonlinear structural analysis of the structure subjected to design loads applied on it;

M_{LX0} = limiting bending moment capacity about the X-axis of the cross-section of the RC member corresponding under the action of pure bending moment (with no axial load on it), estimated as specified in 9.3.2.4,

M_{LY0} = limiting bending moment capacity about the Y-axis of the cross-section of the RC member corresponding under the action of pure bending moment (with no axial load on it), estimated as specified in 9.3.2.4, and

$$\alpha = \begin{cases} 1.0 & \text{if } P_D \leq 0.2P_{L0} \\ 1 + \frac{(P_D - 0.2P_{L0})}{0.6P_{L0}} & \text{if } 0.2P_{L0} < P_D < 0.8P_{L0} \\ 2.0 & \text{if } 0.8P_{L0} \leq P_D \end{cases}$$

in which

P_{L0} = limiting axial load capacity of the RC section under the action of pure axial load (with no bending moment on it), estimate as specified in 9.3.2.3.

9.3.2.7.8 Structural Walls

9.3.2.7.8.1 The design axial load P_D and design bending moments M_{DX} and M_{DY} on RC structural walls shall be estimated as in biaxial bending of columns specified in 9.3.2.7. In particular:

- a) In the out-of-plane direction, structural walls shall be treated as slender columns and the same provisions applicable for columns (specified in 9.3.2.7 shall be complied with, including the effect of additional moments; and
- b) In the in-plane direction, additional moments shall not be used for estimating the total moment in the in-plane direction.

9.3.2.7.8.2 RC structural walls under uniaxial bending shall be designed such that the design demand point $(P_D; M_{DX})$ and $(P_D; M_{DY})$ lie inside the $P_L - M_{LX}$ and $P_L - M_{LY}$ interaction diagrams, respectively. These two interaction diagrams shall be drawn as per the principles laid down in 9.3.2.7.

9.3.2.7.8.3 Alternately, an approximate $P_L - M_L$ interaction diagram can be drawn using 5 strain distribution specified at 5 critical points (Table 49 and Fig. 49). For a RC wall section with uniformly distributed steel, the expressions for limiting P_L and M_L are given in Table 50; $\alpha_{\max}$ and $\beta_{\max}$ in the table shall be taken as per Table 48.

9.3.2.7.8.4 For a RC section with generalised distribution of longitudinal steel (Fig. 50), the following equations may be used:

- a) *Equilibrium of Axial Force and Bending Moment at the section*

$$P_L = \alpha f_{ck} t_w x_L + \sum_{i=1}^n (f_{sc,i} - f_{csc,i}) A_{sc,i} - \sum_{j=1}^n f_{st,j} A_{st,j} \quad (41)$$

$$M_L = \alpha f_{ck} t_w x_L \left[\frac{D}{2} - \beta x_L \right] + \sum_{i=1}^n (f_{sc,i} - f_{csc,i}) A_{sc,i} y_i + \sum_{j=1}^n f_{st,j} A_{st,j} y_j \quad (42)$$

- b) *Compatibility of Strains at the cross section*

$$\varepsilon_{csc,i} = \left(\frac{y_i}{x_L} \right) \varepsilon_c \quad (43)$$

$$\varepsilon_{sc,i} = \left(\frac{y_i}{x_L} \right) \varepsilon_c \quad (44)$$

$$\varepsilon_{st} = \left(\frac{y_j - x_L}{x_L} \right) \varepsilon_c \quad (45)$$

c) *Constitutive Law of constituent materials at the section*

$$f_{csc,i} = \begin{cases} 0.45f_{ck} \left[2 \left(\frac{\varepsilon_{csc,i}}{\varepsilon_{cp}} \right) - \left(\frac{\varepsilon_{csc,i}}{\varepsilon_{cp}} \right)^2 \right] & 0 \leq \varepsilon_{csc,i} \leq \varepsilon_{cp} \\ 0.45f_{ck} & \varepsilon_{cp} \leq \varepsilon_{csc,i} \leq \varepsilon_{cu} \end{cases} \quad (46)$$

$$f_{sc,i} = \begin{cases} E_s \varepsilon_{sc,i} & 0 \leq \varepsilon_{sc,i} \leq \varepsilon_{syD} \\ 0.87f_y & \varepsilon_{syD} \leq \varepsilon_{sc,i} \leq \varepsilon_{sr} \end{cases} \quad (47)$$

$$f_{st,i} = \begin{cases} E_s \varepsilon_{st,i} & 0 \leq \varepsilon_{st,i} \leq \varepsilon_{syD} \\ 0.87f_y & \varepsilon_{syD} \leq \varepsilon_{st,i} \leq \varepsilon_{sr} \end{cases} \quad (48)$$

$$\alpha = \begin{cases} 0.45 \left[\left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right)^2 \right] & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ 0.45 \left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right] & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases}, \quad (49)$$

$$\beta = \begin{cases} \frac{\left[\frac{1}{3} - \frac{1}{12} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) \right]}{\left[1 - \frac{1}{3} \left(\frac{\varepsilon_c}{\varepsilon_{cp}} \right) \right]} & \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{cp} \\ \frac{\left[\frac{1}{2} - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) + \frac{1}{12} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right)^2 \right]}{\left[1 - \frac{1}{3} \left(\frac{\varepsilon_{cp}}{\varepsilon_c} \right) \right]} & \text{for } \varepsilon_{cp} < \varepsilon_c \leq \varepsilon_{cu} \end{cases}. \quad (50)$$

In the above, αf_{ck} is the average compressive stress in concrete, $f_{csc,i}$ the compressive stress in concrete at the level of layer i of reinforcing steel in compression, $f_{sc,i}$ the compressive stress in reinforcing steel at the layer i , and βx_L the distance of resultant compressive force in concrete from the top fiber.

Table 49 Five critical states of strain of a RC rectangular wall section with longitudinal steel distributed uniformly used to draw an approximate limiting $P_L - M_L$ interaction diagram for bending about X-axis

[Clause 9.3.2.7.8.3]

Point	State of Strain	Top	Bottom
A	Uniform compression	ε_{cu}	ε_{cu}
B	Strain gradient	ε_{cu} at top concrete fiber	0 at bottom concrete fiber
C	Balanced strain gradient	ε_{cu} at top concrete fiber	$\varepsilon_{st,lim}$ at bottom steel level
D	Tensile strain gradient	0 at top concrete fiber	$\varepsilon_{st,lim}$ at bottom steel level
E	Uniform tension	$\varepsilon_{st,lim}$	$\varepsilon_{st,lim}$

Table 50 Expressions for limiting axial load capacity P_L and limiting bending moment capacity M_{LX} in a RC rectangular wall section with longitudinal steel distributed uniformly corresponding to the 5 salient points on its limiting $P-M$ interaction diagram for bending about major axis

[Clause 9.3.2.7.8.3 and 9.3.2.7.8.4]

Point	Expressions for P_L and M_{LX}
A	$P_{LA} = 0.45f_{ck}t_wL_w + \sum_{i=1}^n (f_{sc,i} - 0.45f_{ck})A_{sc,i}$ $M_{LA} = 0$
B	$P_{LB} = \alpha_{\max}f_{ck}t_wL_w + \sum_{i=1}^n (f_{sc,i} - 0.45f_{ck})A_{sc,i}$ $M_{LB} = \alpha_{\max}f_{ck}t_wL_w \left(\frac{L_w}{2} - \beta_{\max}x_L \right) + \sum_{i=1}^n (f_{sc,i} - 0.45f_{ck})A_{sc,i}y_i$
C	$P_{LC} = \alpha_{\max}f_{ck}t_wL_w + \sum_{i=1}^{n_1} (f_{sc,i} - f_{csc,i})A_{sc,i} + \sum_{j=1}^{n_2} f_{st,j}A_{st,j}$ $M_{LC} = \alpha_{\max}f_{ck}t_wL_w \left[\frac{L_w}{2} - \beta_{\max}x_L \right] + \sum_{i=1}^{n_1} (f_{sc,i} - f_{csc,i})A_{sc,i}y_i + \sum_{j=1}^{n_2} f_{st,j}A_{st,j}y_j$
D	$P_{LD} = -\sum_{j=1}^n f_{st,j}A_{st,j}$ $M_{LD} = \sum_{j=1}^n f_{st,j}A_{st,j}y_j$

E	$P_{LC} = -\sum_{j=1}^n 0.87 f_y A_{st,j}$ $M_{LC} = 0$

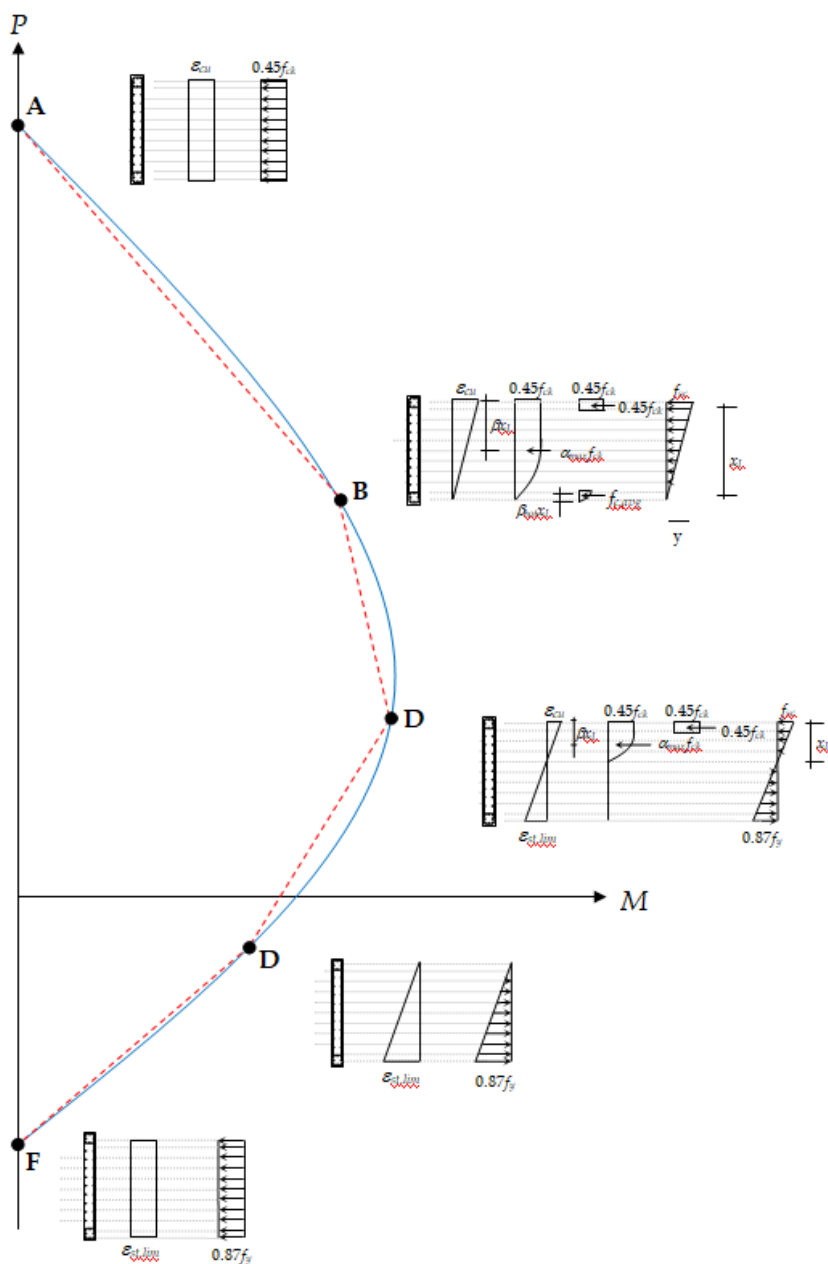


FIG. 49 NORMAL STRAIN AND STRESS DISTRIBUTIONS IN A RC SECTION CORRESPONDING TO THE EIGHT SALIENT POINTS ON ITS DESIGN P-M INTERACTION DIAGRAM

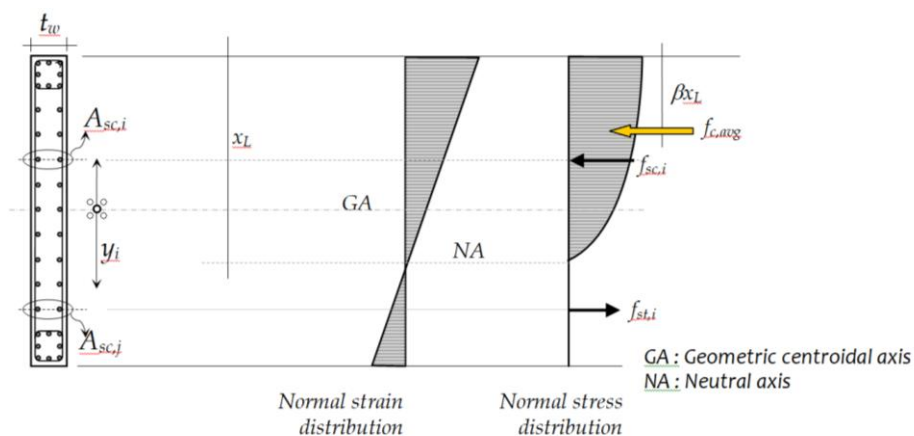


FIG. 50 GENERAL RECTANGULAR RC STRUCTURAL WALL SECTION
REINFORCING STEEL ON ALL FOUR FACES

9.3.2.8 Design combined axial force P_L – shear force V_L capacity

The relation between *design axial force* P_L and *design shear force* V_L capacities shall be estimated using the following axial stress σ – shear stress τ relation (Fig. 51):

$$\tau = 0.1 f_{ck} \sqrt{0.62 + 10.1 \left(\frac{\sigma_c}{f_{ck}} \right) + 5.8 \left(\frac{\sigma_c}{f_{ck}} \right)^2 - 18.6 \left(\frac{\sigma_c}{f_{ck}} \right)^3 + 2.09 \left(\frac{\sigma_c}{f_{ck}} \right)^4},$$

where

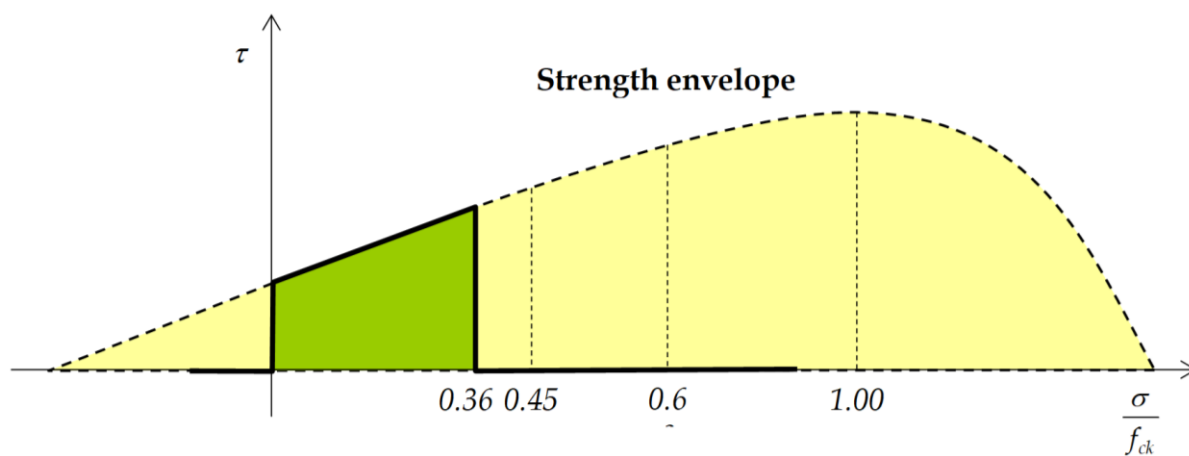


FIG. 51 LIMITING (AXIAL - SHEAR) STRESS ENVELOPE AT A POINT IN CONCRETE

9.3.2.9 Design combined shear force V_L – bending moment M_L capacity

The factored shear force demand V_F shall satisfy the following condition (Fig. 52):

$$V_D \leq \left[V_0 - \left(\frac{M_D}{M_0} \right) (V_0 - V^*) \right],$$

where

V_0 = design shear force capacity as specified in 9.3.2.5,

V^* = contribution of concrete to design shear force capacity V_D ,

M_0 = design bending moment capacity as specified in 9.3.2.4, and

M_D = design bending moment demand at the critical section of the member obtained from Structural Analysis.

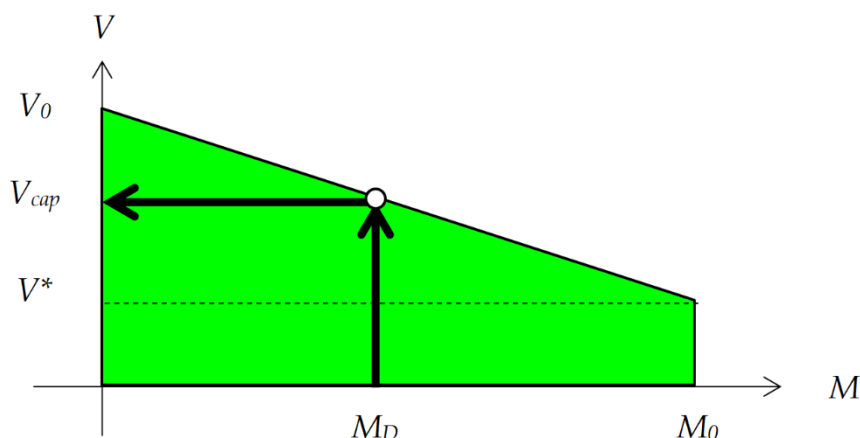


FIG. 52 LIMITING (SHEAR FORCE - BENDING MOMENT) STRENGTH ENVELOPE AT A CROSS-SECTION OF A REINFORCED CONCRETE MEMBER

9.3.2.10 Design combined axial force P_L – shear force V_L – bending moment M_L capacity

Members of select concrete structures that do not have redundancy shall be ensured to have sufficient combined $P_L - V_L - M_L$ capacity.

9.3.3 Design and Detail of RC Members

The design and detailing of reinforced concrete components and sub-systems shall be performed as provided hereunder.

a) Structural Design

The grades of materials (namely concrete and reinforcing steel) admissible for use in concrete structures shall be as specified in **1.2**. Further, in the calculations related to design of transverse reinforcement, the characteristic yield strength f_y shall not be taken to be more than 415 MPa, irrespective of the grade of transverse steel used.

The admissible combinations of the grades of concrete and reinforcing steel in reinforced concrete members shall comply with those specified by Table 51. The said combinations shall be applicable for both longitudinal and transverse steels.

b) Reinforcement Detailing

The detail requirements provided in this standard are mandatory for basic structural components and sub-systems. Members of concrete structures shall be provided with reinforcing steel at locations and faces where concrete member is likely to undergo axial tension, bending tension, axial compression, and/or bending compression. This reinforcement shall be arranged along longitudinal and transverse directions of the member.

Bars may be arranged singly, or in pairs in contact, or in groups of three or four bars bundled in contact. Bundled bars shall be enclosed within stirrups or ties. Bundled bars shall be tied together to ensure the bars remaining together. Bars larger than 32 mm diameter shall not be bundled, except in columns.

The recommendations for detailing for earthquake-resistant construction given in IS 13920 should be taken into consideration, where applicable.

Table 51 Combinations of Grades of Concrete and Reinforcing Steel admissible in reinforced concrete structures

[Clause 9.3.3(a)]

Grade of Concrete	Grade of Reinforcing Steel			
	Fe250	Fe415	Fe500	Fe550
M20	Yes	Yes	Yes	Yes

M25				
M30				
M35				
M40				
M45				
M50	Not Admissible			
M55				
M60				
M65	Not admissible			
M70				

9.3.3.1 Slabs

The general requirements for geometry shall be provided as specified in **8.2.1**.

a) Solid Slabs**1) Design**

Slabs shall be designed for Shear Force V and Bending Moment M to resist all forces transferred thorough them under the combination of loads specified in Table 3 for Strength Criteria. The capacity of the Slabs shall be estimated as specified in **9.3.2.4** and **9.3.2.5** at least at the critical sections specified in **9.2.5**.

2) Detailing

The general requirements for reinforcement shall be provided as specified in **9.2**. In addition, the requirements given hereunder shall be complied with.

a) One-way Slabs**i) Bottom reinforcement**

- 1) Main tension reinforcement shall be provided in the lower part of the slab along short span direction throughout the width of the long span, such that:
 - a) at least 50 percent of it extends over the full width of the support, and
 - b) remaining to within $0.25l$ of continuous support and $0.15l$ of discontinuous support.
- 2) Secondary tension reinforcement shall be provided in the lower part of the slab along long span direction throughout the width of the short span on top of the main tension reinforcement, such that:
 - a) at least 50 percent of it extends over the full width of the support, and
 - b) remaining to within $0.25l$ of continuous support or $0.15l$ of discontinuous support.

(ii) *Top reinforcement*

- 1) *Main tension reinforcement* shall be provided in the upper part of the slab along short span direction throughout the width of the long span, such that it extends at least:
 - a) $0.25l$ from the face of the *continuous support*, and
 - b) $0.15l$ of *discontinuous support*.
- 2) *Secondary tension reinforcement* shall be provided in the upper part of the slab along long span direction throughout the width of the short span *below the main tension reinforcement*, such that it extends at least:
 - (a) $0.25l$ from the face of the *continuous support*, and
 - (b) $0.15l$ of *discontinuous support*.

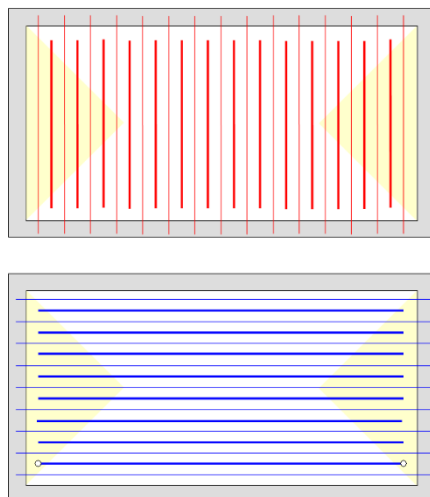


FIG. 53 REINFORCEMENT DETAILING OF ONE-WAY SOLID SLAB

b) Two-way Slabs restrained against torsion at the ends

The following detailing requirements shall be satisfied when not involving effects of *earthquake* and *temperature* loading:

- (i) Tension reinforcement provided at mid-span in the middle strip shall extend in the lower part of the slab to within $0.25l$ of a continuous edge, or $0.15l$ of a discontinuous edge.
- (ii) Over the continuous edges of a middle strip, the tension reinforcement shall extend (in the upper part of the slab) a distance of $0.15l$ from the support, and at least 50 percent shall extend a distance of $0.30l$.
- (iii) At a discontinuous edge, negative moments may arise, which depend on the degree of fixity at the edge of the slab. But, in general, tension reinforcement shall be provided equal to at least 50 percent of that provided at mid-span, extending $0.10l$ into the span.
- (iv) Reinforcement in edge strip, parallel to that edge, shall comply with the minimum given in 9.2.4.1 and the requirements for torsion given in (v)-(vi) below.
- (v) Torsion reinforcement shall be provided at any corner where the slab is simply supported on both edges meeting at that corner. It shall consist of top and bottom reinforcement, each with layers of bars placed parallel to the sides of the slab and extending from the edges a minimum distance of one-fifth of the shorter span. The area of reinforcement in each of these four layers shall be three-quarters of the area required for the maximum mid-span moment in the slab.

- (vi) Torsion reinforcement equal to half that described in (v) above shall be provided at a corner contained by edges over only one of which the slab is continuous.
- (vii) Torsion reinforcements need not be provided at any corner contained by edges over both of which the slab is continuous.
- (viii) Extend half the support tension steel at each support to at least an effective depth or 12 bar diameters beyond the nearest point of contra-flexure.
- (ix) Extend the full area of the support tension steel at each support to at least half the distance obtained in (iii) above.
- (x) At least 50 percent of tension reinforcement provided at mid-span shall extend to the supports, and the remaining 50 percent to within $0.10l_x$ or $0.10l_y$ of the support, as appropriate.

c) Two-way Slabs not restrained against torsion at the ends

i) Size and Position of Rib

In-situ ribs shall be not less than 65 mm wide. They shall be spaced at centers not greater than 1.5m apart and their depth, excluding any topping, shall be not more than four times their width. The ribs shall be formed along each edge parallel to the span of one way slabs. When the edge is built into a wall or rests on a beam, a rib at least as wide as the bearing shall be formed along the edge.

ii) Arrangement of Reinforcement

The maximum distance between bars given in 9.2.2.5 apply to areas of solid concrete in these slabs also. The curtailment, anchorage and cover to reinforcement shall be as described below:

- a) At least 50 percent of the total main reinforcement shall be carried through at the bottom on to the bearing and anchored as per 9.2.2.10;
- b) In slabs with permanent blocks, the side cover to the reinforcement shall not be less than 10 mm. In all other cases, cover shall be provided as per 9.2.1.5.

3) Restrained Slab with Unequal Conditions at Adjacent Panels

In some cases, the support moments calculated from 8.7.1.6 for adjacent panels may differ significantly. The following procedure shall be adopted to adjust them:

- i) Calculate the sum of moments at mid-span and supports (neglecting signs).
- ii) Treat the values from Table 26 as fixed end moments.
- iii) Distribute the fixed end moments across the supports, according to the relative stiffness of adjacent spans.
- iv) Adjust mid-span moment such that, when added to the support moments from (iii) above, the total moment should be equal to that from (i) above.

If the resulting support moments are significantly greater than the value from Table 30, the tension steel over the supports shall be extended further. The procedure should be as follows:

- a) Take the span moment as parabolic between supports; its maximum value is as found from (iv) above.
- b) Determine the points of contraflexure of the new support moments from (iii) above with the span moment from (a).
- c) Extend half the support tension steel at each end to at least an effective depth or 12 bar diameters beyond the nearest point of contraflexure.
- d) Extend the full area of the support tension steel at each end to half the distance from (c).

d) Flat Slabs

The design and detail of flat slabs shall be performed as specified hereunder.

1) Geometry

The minimum thickness of flat slabs shall be 125mm.

When drop panels are used, the minimum thickness of drop panel shall be the larger of:

- (a) 1.5 times the thickness of slab, and
- (b) 300 mm.

The drop panel shall extend in each direction from the centerline of support to a distance not less than *one-sixth the span length* measured from center-to-center of supports along that direction.

2) Design for Bending Moment

The middle strips, column strips and effective slab width of flat slabs shall be designed for the bending moments obtained as per 8.7.3.4.

3) Design for Shear Force

Flat slabs shall be designed for effects of both one-way and two way shear.

When openings in flat slabs are located at a distance less than *10 times* the thickness of the slab from a concentrated reaction, or when the openings are located within the column strips, the critical sections specified above shall be modified so that the part of the periphery of the critical section, which is enclosed by radial projections of the openings to the centroid of the reaction area, shall be considered ineffective (Fig. 54), and openings shall not encroach upon column head.

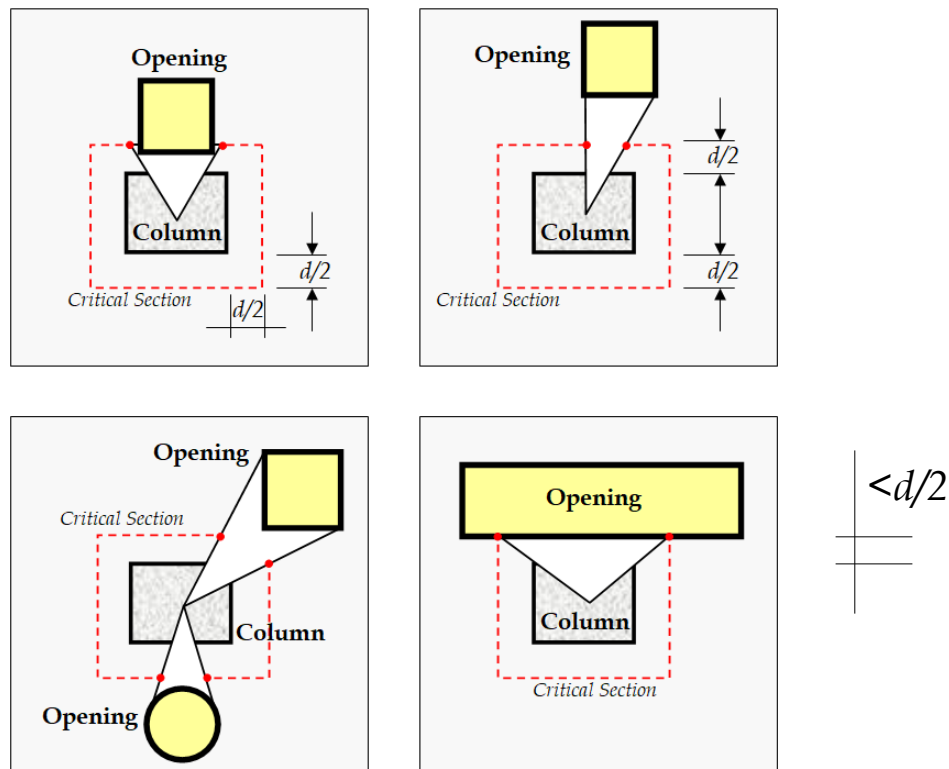


FIG. 54 TRUNCATED CRITICAL SECTIONS (IN PLAN) FOR SHEAR FORCE IN FLAT SLABS OWING TO PRESENCE OF OPENINGS ADJOINING COLUMNS

a) One-way Shear

At any support, under a concentrated load or when there is a change in thickness, one-way shear at the critical section extending in a plane across design strip shall satisfy requirement of **9.2.5.2.1(b)**. In the absence of more detailed analysis, one-way shear shall be distributed between the *column strip* and *middle strips* in proportion to the design negative moments in each strip.

b) Two-way Shear

Slab shall be designed for two-way shear at support, concentrated load and reaction areas at critical sections specified in **9.2.5.2.1(b)**.

Design Shear Stress Demand τ_d shall be computed from following equation,

$$\tau_d = \frac{V_D}{b_o d} + \left(\frac{(1 - \alpha_x) M_{ubx} C_x}{J_{xx}} \right) + \left(\frac{(1 - \alpha_y) M_{uby} C_y}{J_{yy}} \right)$$

where

V_D = Absolute value of Design shear force on critical section,

M_{ubx} = Absolute value of unbalanced column moment about centroidal x-axis of critical section,

M_{uby} = Absolute values of unbalanced column moment about centroidal y-axis of critical section,

b_o = Perimeter of critical section,

d = Average of effective depths of slab in two orthogonal directions,

$(1 - \alpha_x)$ = Fraction of unbalanced column moment transferred by shear about centroidal x-axis of critical section,

$(1 - \alpha_y)$ = Fraction of unbalanced column moment transferred by shear about centroidal y-axis of critical section,

α_x = Fraction of unbalanced column moment transferred by flexure about centroidal axes x-axis of critical section,

α_y = Fraction of unbalanced column moment transferred by flexure about centroidal axes y-axis of critical section,

C_x = Distance of point of interest from centroidal x-axis of critical section,

C_y = Distance of point of interest from centroidal y-axis of critical section,

J_{xx} = Property of critical section analogous to polar moment of inertia about centroidal x-axis of critical section, and

J_{yy} = Property of critical section analogous to polar moment of inertia about centroidal y-axis of critical section.

c) Two-way Shear

Shear reinforcement shall be provided as per the following:

(1) When $0 < \tau_D \leq \tau_{LCp}$:

No shear reinforcement need be provided;

(2) When $\tau_{LCp} < \tau_D \leq 1.5\tau_{LC}$:

Shear reinforcement shall be provided at the critical section as specified hereunder. Shear stresses shall be investigated at successive sections more distant from the support and shear reinforcement shall be provided up to a section where *Design Shear Stress Demand* τ_D does not exceed $0.5\tau_{LC}$. While designing the shear reinforcement, concrete shall be assumed to carry a shear stress of $0.5\tau_{LC}$, and reinforcement the balance of the *Design Shear Stress Demand* τ_D , i.e., $(\tau_D - 0.5\tau_{LC})$, and

(3) When $1.5\tau_{LC} < \tau_D$:

The Flat Slab shall be re-designed to increase its shear area and/or reduce the shear stress demand τ_D ;

where

τ_{LCp} = Design Punching Shear Strength of concrete specified in **9.2.6.1(d)**,

and

τ_{LC} = Design Shear Strength of concrete specified in **9.2.6.1(d)**.

(4) *Slab Reinforcement*

a) Longitudinal Reinforcement

The spacing of longitudinal bars in a Flat Slab shall not exceed 2 times the slab thickness, except where a slab is of cellular or ribbed construction.

The reinforcement within effective slab width b_{eff} shall be curtailed at a minimum distance of $D + l_d$ beyond critical section for two-way shear specified in **9.2.5.2.1**.

When $\frac{V_D}{b_0 d} > \frac{0.17}{\gamma_M} \lambda_a \lambda_s \sqrt{f_{ck}}$ on critical section for two-way shear surrounding column, concentrated load or reaction area, reinforcement within effective slab width b_{eff} , which is designed for moment αM_{ub} , shall not be less than $A_{s,min}$,

$$A_{s,min} = \frac{20 V_d b_{eff}}{3 \alpha_s d f_y},$$

where V_d , b_{eff} and α_s are as defined in 8.7.3.2.

When drop panels are used, the thickness of drop panel for determination of area of reinforcement shall be the lesser of:

- (a) Thickness of drop, and
- (b) Thickness of slab plus $\frac{1}{4}$ th the distance between edges of drop and column & capital.

The length of longitudinal reinforcement in flat slabs shall:

- (1) Have at least the lengths specified in **Figure 55**.
- (2) Be provided with lengths larger, when required by analysis,
- (3) Extensions in negative reinforcement beyond each face of the common column based on the longer span, where adjacent spans are unequal, and
- (4) Bent bars shall be permitted only where the depth-span ratio permits use of bends of 45° or less, and

The above are specified for Flat Slabs that are braced against lateral sway. Flat Slabs shall be built only in structures that are not restrained against lateral sway.

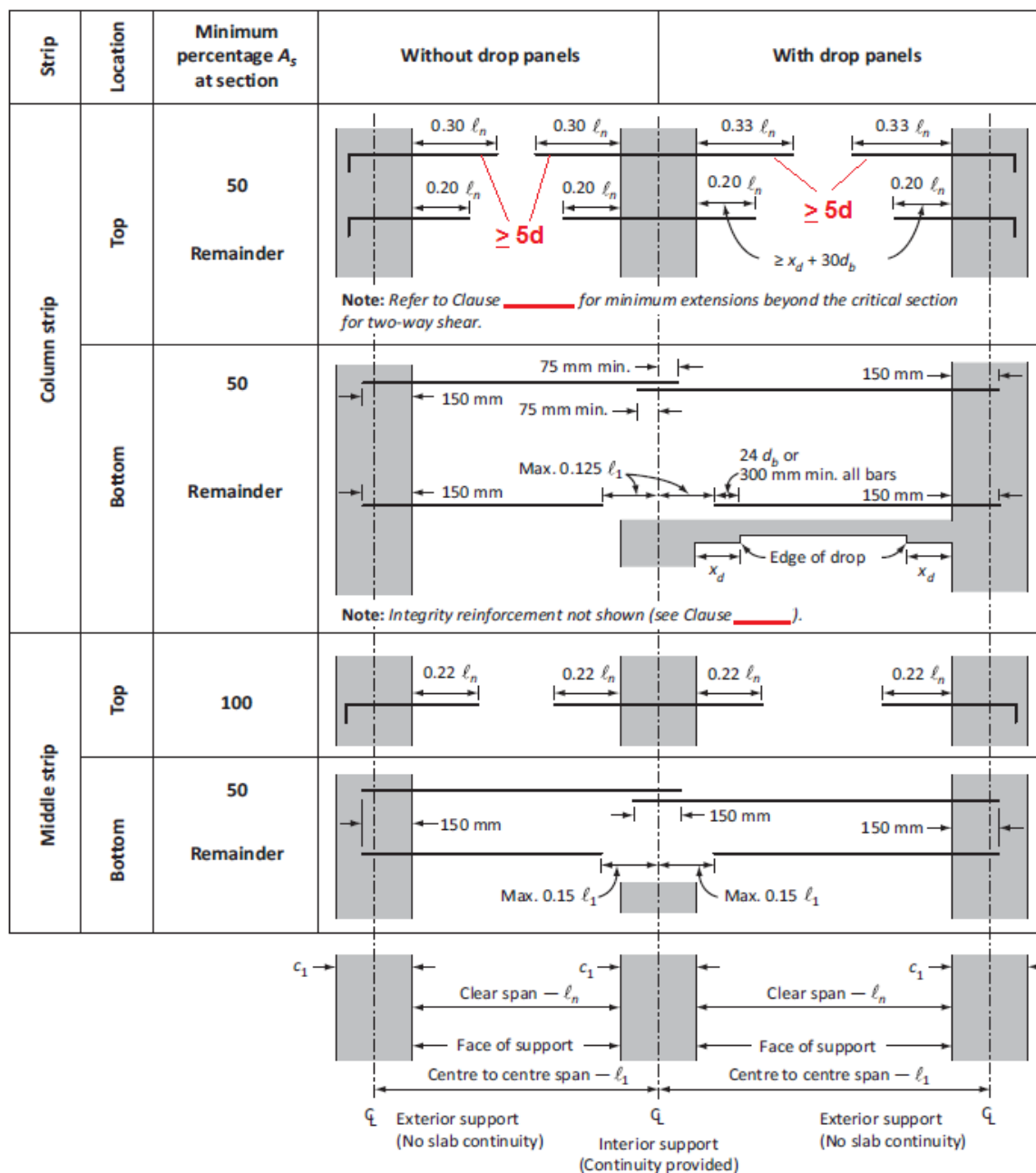


FIG. 55 MINIMUM LENGTH OF REINFORCEMENT IN FLAT SLABS

Bottom *integrity* reinforcement shall be provided in drop panels, which passes through the column in either direction. Such reinforcement shall:

- Have at least 2 bars passing continuously from the slab into the column,
- Have no splicing over a length equal to the development in tension as estimated above, from the face of the drop panel,

- c) Have sufficient development beyond the face of the column in tension (estimated using the ultimate tensile strength of the steel reinforcement used),
- d) Be sized based on the vertical symmetric punching force corresponding to the combination of (1.2DL+0.5IL), and
- e) The sum of area of continuous bottom *integrity* reinforcement passing through column or column capital core in each span direction shall satisfy the following requirement:

$$\sum_{i=1}^{N_b} \{ (0.87 f_y) A_{s,i} \} + \sum_{j=1}^{N_p} \{ f_p A_{sp,j} \} \geq P_{D, floor}$$

where

$A_{s,i}$ = Area of *non-prestressed bottom reinforcement i* passing through column or in the width of the column capital in the plan direction considered; only half of the area of reinforcement shall be considered, if not continuous and not anchored on opposite face of column;

$A_{sp,j}$ = Area of *prestressed tendons j* passing through column or column capital core in each span direction; only half of the area of prestressed reinforcement shall be considered, if not continuous and not anchored on opposite face of column;

f_y = Characteristic strength of non-prestressed reinforcement;

f_p = Stress in prestressed reinforcement under design loads; and

$P_{D, floor}$ = Design *axial load* on column from the floor slab.

- f) The integrity reinforcement as specified above shall satisfy following requirements:

- 1) In *RC flat slabs*, at least 2 bars of 10 mm diameter shall be provided in each direction. Also, $\sum_{j=1}^{N_p} \{ f_p A_{sp,j} \} = 0$.

- 2) In *PT flat slabs*,

- (a) At least 2 *tendons* of 12.7 mm diameter shall be provided in each direction. Contribution of *non-prestressed reinforcement* can be considered.
- (b) Outside the *column* or *column capital*, the tendon in each direction shall pass under tendon in other (orthogonal) direction in adjacent span. If this requirement is not satisfied, necessary integrity reinforcement shall be provided using non-prestressed reinforcement as specified in (a) above.

- (c) Non-prestressed reinforcement shall be anchored at *exterior support or column* with development length calculated using characteristic strength of reinforcement f_y .
- (d) Prestressed tendons shall be anchored at exterior support at or beyond exterior face of support or column.
- (e) If required, non-prestressed reinforcement shall be spliced with mechanical, welded or lap splice at column or within $0.3l_n$ from face of column in flat slabs *without drops* and within $0.33l_n$ from face of column *in flat slab with drops*. When lap splices are used, the splice length shall be at least 1.3 times development length calculated using characteristic strength of reinforcement f_y . Alternatively, additional bottom bars can be provided as *integrity reinforcement* passing through column or column capital, and overlapping with bars in adjoining span for at least 2 times development length calculated using characteristic strength of reinforcement f_y .

The spacing of longitudinal bars in a Flat Slab shall meet the following:

- (1) Positive or negative reinforcement shall not be spaced more than 2 times the slab thickness, or 450 mm, whichever is less, except where a slab is of cellular or ribbed construction.
- (2) Reinforcement within effective slab width b_{eff} shall not be spaced more than 1.5 times the slab thickness, or 250 mm, whichever is less.
- (3) Positive reinforcement shall not be spaced less than 150 mm, except that when fully welded transverse mesh is placed directly over the support, this length can be reduced to $\frac{1}{2}$ the width of the support or 50 mm, whichever is greater.
- (4) Negative reinforcement shall be spaced such that the design stress developed at the internal face is in accordance with 8.7.3.5.3.

Also, when the Flat Slab is not supported by a spandrel beam or wall, or when it cantilevers beyond the support, the anchorage shall be obtained within the Flat Slab.

b) Shear Reinforcement

Single leg, single-U, multiple-U or closed stirrups, (Fig. 56) can be used as shear reinforcement, provided they meet the following requirements:

- (a) $d \geq 150$ mm and
- (b) $d \geq 16d_b$, where d_b is diameter of stirrups.

The design shear strength of these stirrups V_s shall be estimated as:

$$V_{Ls} = 0.87 f_y \left(\frac{A_{sv}}{b_o s_v} \right),$$

where

f_y = Characteristic strength of stirrups which shall not exceed 415 MPa,

A_{sv} = Sum of cross-sectional area of all legs of stirrups on a concentric line parallel to the perimeter of column,

b_o = Perimeter of critical section,

s_v = Spacing of stirrups measured perpendicular to the column face which shall not exceed $d/2$.

Distance from column face to first stirrups, perpendicular to column face shall not exceed $d/2$. Distance between vertical legs of stirrups, parallel to column face shall not exceed $2d$.

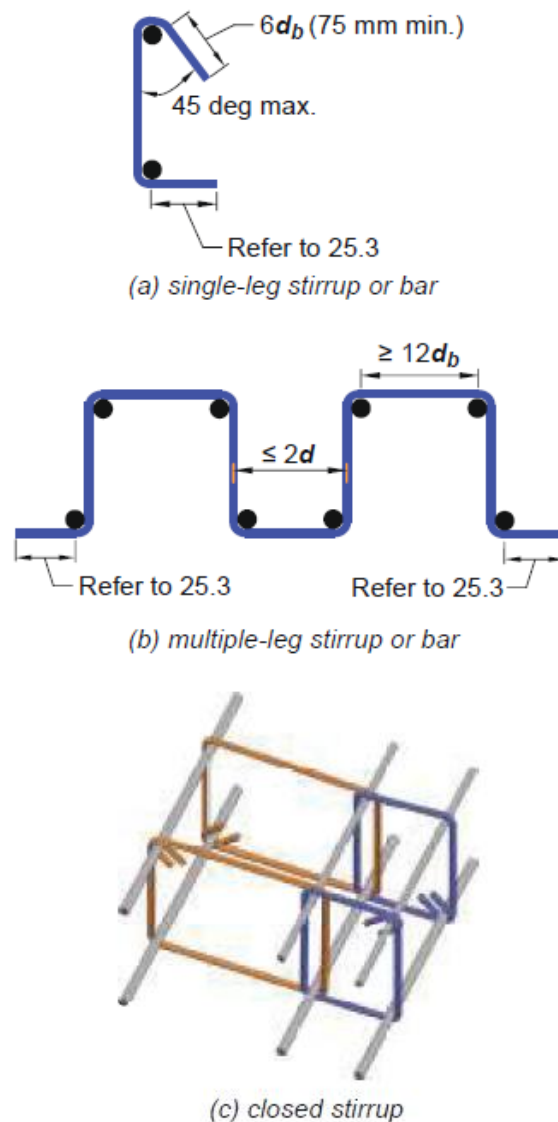


FIG. 56 SINGLE LEG, MULTIPLE-U AND CLOSED STIRRUPS SHEAR REINFORCEMENT IN FLAT SLABS

(5) Openings in Flat Slabs

Openings of any size may be provided in the Flat Slab, if it is shown by detailed analysis that the requirements of strength and serviceability are met. But, when openings conform to the following, no special analysis is required.

- a) Openings of any size may be placed within the *middle strip* in each direction, provided the total amount of reinforcement required for the middle strip without the opening is provided in the remaining width.

- b) In the area common to two parts of the column strips from the adjacent panels, not more than $\frac{1}{8}$ th the width of column strip in either span shall be interrupted by the openings. The area of reinforcement interrupted shall be added on each side of the opening in both directions.
- c) In the area common to one column strip and one middle strip, not more than $\frac{1}{4}$ th the reinforcement in either strip shall be interrupted by the openings. The equivalent of reinforcement interrupted shall be added on all sides of the openings.
- d) Shear design requirements shall be satisfied as specified in **8.7.3.2**.

(e) Ribbed, Hollow Block and Voided Slabs

In ribbed slabs, in-situ ribs shall be not less than 65 mm wide. Their centers shall be spaced not more than 1.5 m apart, and their depth (excluding any topping) shall not be more than 4 times their width. Generally, ribs shall be formed along each edge parallel to the span of one way slabs. When the edge is built into a wall or rests on a beam, a rib at least as wide as the bearing shall be formed along the edge.

Subject to meeting the fire-requirements in **9.2.2.2.2**, the void shall not be considered in the estimation of the voided slab. But, the minimum strength needed for construction shall be ensured.

In slabs made of blocks and formers, the hollow clay tiles shall conform to IS 3951 (Part 1). When required to contribute to the structural strength of a slab, they shall:

- 1) Be made of concrete or burnt clay; and
- 2) Have a crushing strength of at least 14 MPa measured on the net section, when axially loaded in the direction of compressive stress in the slab.

When contribution of void formers to the structural strength is not required:

- (1) Void former shall be of plastic or other equivalent material;
- (2) Analysis and design of voided slab shall consider:
 - (a) Reduction in weight due to introduction of voids. Density of voided portion can be modified to take into account reduction in weight.
 - (b) Reduction in stiffness due to introduction of voids. Reduced stiffness of voided slab can be mathematically calculated considering size and shape of void former in proposed thickness.
 - (c) Reduction in shear capacity due to introduction of voids. Void formers shall be located in areas where design shear force is less than or equal to shear resistance of voided part. In punching area, solid area shall extend at least by $2d$ beyond critical section or beyond last row of punching shear reinforcement. Along the support edges and free slab edges, minimum solid zone equal to slab thickness shall be provided.
 - (d) when compression zone remains above the void former (i.e. neutral axis is within top concrete cover of void former), flexural resistance of voided slab in voided part shall be similar to that of solid slab. When neutral axis is within void former, reduction in flexural resistance shall be considered.

- (e) Sum of top and bottom continuous solid part (*i.e.*, sum of top and bottom cover of void former) shall be considered to determine rigidity of diaphragm as per IS 1893 (Part 1).
- (f) Code specified minimum reinforcement in voided part shall be based on equivalent thickness of voided slab. Equivalent thickness D_{eq} of voided slab shall be calculated as below:

$$D_{eq} = \left(1 - \frac{V_v}{V_s}\right) D$$

where

V_v = Volume of void per m² area,

V_s = Volume of solid slab of similar thickness per m² area,

D = Thickness of slab.

- (g) Fire resistance of voided slab shall take into account effective reinforcement cover, spacing of reinforcement, clear bottom cover to void former and thickness of slab. Equivalent slab thickness D_{eq} shall be considered to determine minimum thickness required for particular fire resistance.
- (h) When voided slab is casted in two working steps to secure void formers from buoyancy, necessary joint shear reinforcement shall be provided at construction joint.

The maximum distance between bars given in 9.2.2.5 applies to areas of solid concrete in this form of construction also. The curtailment, anchorage and cover to reinforcement shall be as described below:

- (a) At least 50 percent of the total main reinforcement shall be carried through at the bottom on to the bearing and anchored in accordance with 9.2.2.8.
- (b) Where a slab, which is continuous over supports, has been designed as simply supported, reinforcement shall be provided over the support to control cracking. This reinforcement shall have a cross-sectional area of not less than 25 percent that required in the middle of the adjoining spans and shall extend at least 1/10th of the clear span into adjoining spans.
- (c) In slabs with permanent blocks, the side cover to the reinforcement shall not be less than 10 mm. In all other cases, cover shall be provided according to 9.2.1.5.

9.3.3.2 Beams

The general requirements for geometry shall be provided as specified in 9.2.

a) Flexure-dominant Beams

(1) DESIGN

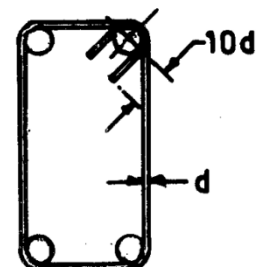
Beams shall be designed for *Shear Force V* and *Bending Moment M* to resist all forces transferred through them under the combination of loads specified in Table 3 for *Strength Criteria*. The capacity of the Beams shall be estimated as specified in **9.3.2.4** and **9.3.2.5** at least at the critical sections specified in **9.2.5**.

(2) DETAILING

The general requirements for reinforcement shall be provided as specified in **9.2**. In addition, the requirements given hereunder shall be complied with. Longitudinal steel in beams shall be enclosed within closed loop transverse reinforcement for effective lateral restraint. The arrangement of transverse reinforcement shall be as below:

(i) Longitudinal Compression Reinforcement

Whenever compression reinforcement is introduced in a beam to resist additional moments (and not merely as hanger bars to hold the longitudinal reinforcement bars), it should be restrained effectively within closed loop transverse reinforcement. Also, in flanges of T-and L-beams, where flanges are in tension, a part of the main tension reinforcement shall be distributed over the effective flange width or a width equal to $1/10^{\text{th}}$ of the span, whichever is smaller. If the effective flange width exceeds $1/10^{\text{th}}$ of the span, nominal longitudinal reinforcement shall be provided in the portions of the flange beyond $1/10^{\text{th}}$ of the span.



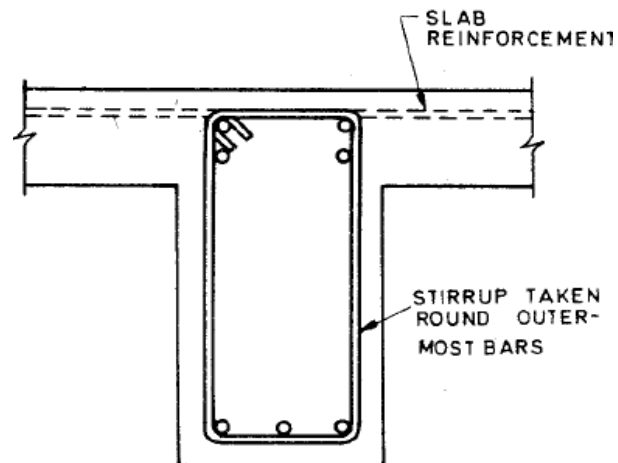


FIG. 57 TRANSVERSE REINFORCEMENT IN *FLANGED BEAMS* RESISTING SHEAR FORCE AND TORSIONAL MOMENT

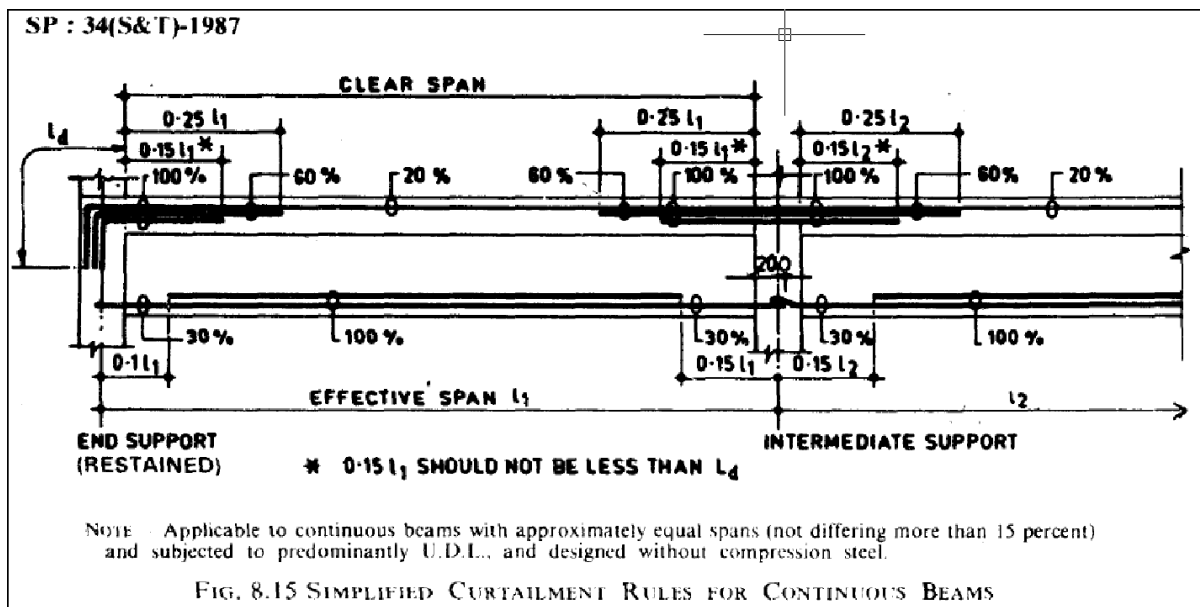


FIG. 58 Simplified Curtailment Rules For Continuous Beams

(ii) Side Face Reinforcement

Where the depth of the clear length of the web in a beam exceeds 750 mm, side face reinforcement shall be provided along both two faces of the clear length of the web. The total area of such reinforcement shall be not less than 0.05% of the web area on each face. This steel shall be distributed equally a spacing not exceeding 300 mm or web thickness, whichever is less.

(iii) Transverse reinforcement in beams for Shear and Torsion

The transverse reinforcement in beams shall be taken around the outer-most tension and compression longitudinal bars. And,

- (a) The transverse reinforcement for torsion shall be rectangular closed stirrups placed perpendicular to the axis of the member. The spacing of the stirrups shall not exceed $\text{Min}\left[x_1; \frac{x_1 + y_1}{4}; 300\text{mm}\right]$, where x_1 and y_1 are the short and long dimensions of the stirrup, respectively.

- (b) The longitudinal reinforcement shall be placed as close as is practicable to the comers of the cross-section.
- (c) There shall be at least one longitudinal bar in each comer of the ties.
- (d) When the cross-sectional dimension of the member exceeds 450 mm, additional longitudinal bars shall he provided to satisfy the requirements of minimum reinforcement and spacing given in **9.2.2.6**.
- (e) In *T-beams* and *I-beams*, such reinforcement shall pass around longitudinal bars located close to the top and bottom faces of the flange.

(b) Compression-dominant Deep Beams

A beam shall be deemed to be a *deep beam*, when the ratio l/d of effective span to overall depth is less than:

- (1) 2.0 in simply supported beams; and
- (2) 2.5 in continuous beams.

A *Deep Beam*, which complies with the requirements pertaining to detail of steel reinforcement given hereunder, shall be deemed to satisfy the provisions for *design* for *shear force* as laid down in this standard:

(1) DESIGN

The lever arm z shall be determined as below:

- (a) For simply supported deep beams:

$$z = \begin{cases} 0.2(l + 2D) & 1 \leq \frac{l}{D} \leq 2 \\ 0.6l & \frac{l}{D} < 1 \end{cases}, \text{ and}$$

- (b) For continuous deep beams:

$$z = \begin{cases} 0.2(l + 1.5D) & 1 \leq \frac{l}{D} \leq 2.5 \\ 0.5l & \frac{l}{D} < 1 \end{cases},$$

where $l = \text{Min}[l_o; 1.15l_{clear}]$, in which l_o is the center-to-center span between supports and l_{clear} the clear span between supports.

The area of the longitudinal steel shall be determined as below:

$$A_{st} = \frac{M_D}{(0.87 f_y) z},$$

where M_D is the design moment estimated from structural analysis under the action of loads corresponding to the *basic load combinations* specified in **Table 3** for *strength criteria*.

(2) DETAILING

The following reinforcement details shall be complied with:

(1) Positive Reinforcement:

The tensile reinforcement provided to resist positive bending moment in any span of a deep beam shall:

- (a) Extend without curtailment between supports;
- (b) Be embedded beyond the face of each support, so that at the face of the support it shall have a development length not less than $0.8L_{dt}$, where L_{dt} is the development length (as per **9.2.2.7**) required to mobilize the *design stress* in the reinforcing steel bars; and
- (c) Be placed within a zone of depth equal to $(0.25D - 0.05l_e)$ adjacent to the tension face of the beam.

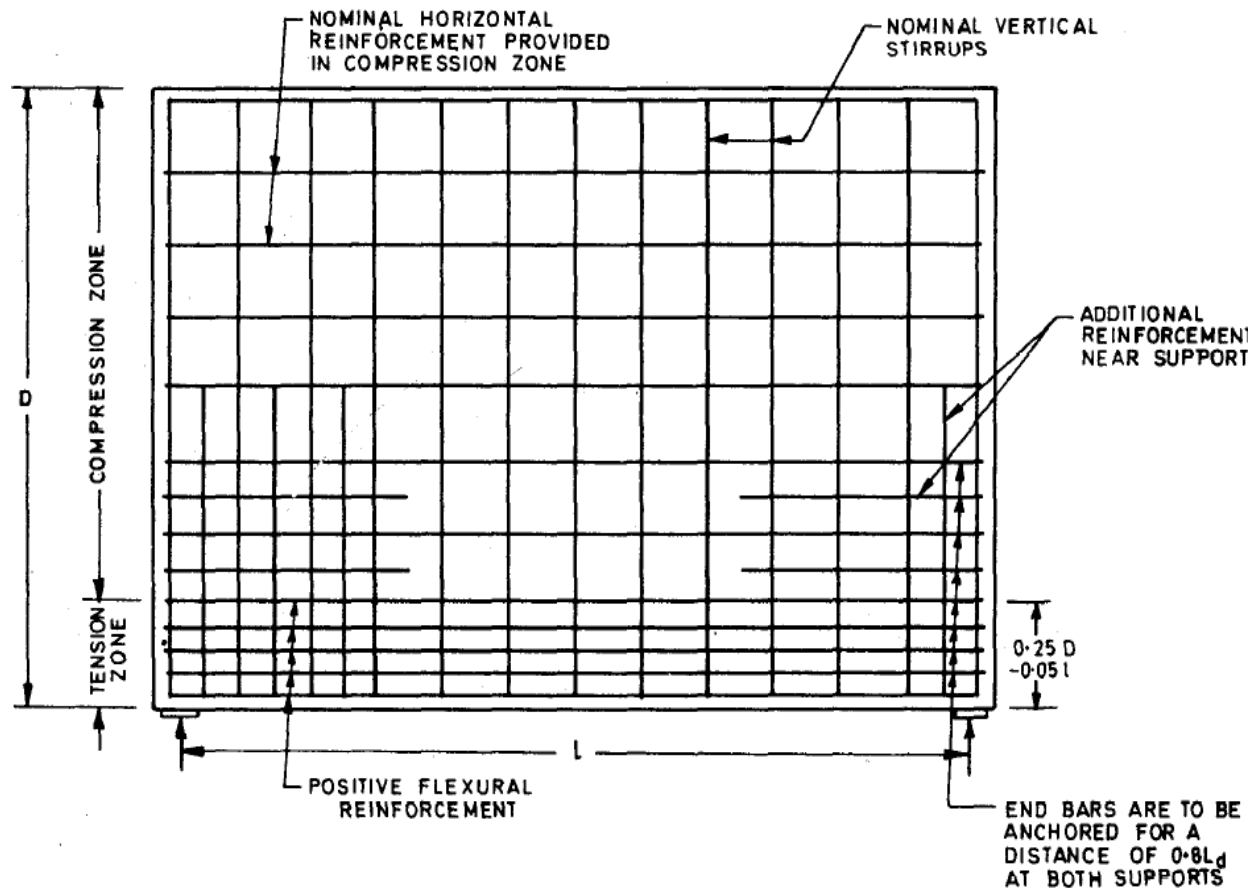


FIG. 59 POSITIVE REINFORCEMENT DETAIL IN SIMPLY SUPPORTED *DEEP BEAMS*

(2) Negative Reinforcement:

The tensile reinforcement provided to resist positive bending moment in any span of a Deep Beam shall:

- (a) *Termination of reinforcement* – The tensile reinforcement required to resist negative bending moment over a support of a deep beam shall be permissible to terminate not more than half of the reinforcement at a distance of $0.5D$ from the face of the support, and the remainder shall extend over the full span.
- (b) *Distribution of reinforcement* - When ratio (l_{clear}/D) is in the range 1.0-2.5, the tensile reinforcement over a support of a deep beam shall be placed in two zones comprising:

(i) A zone of depth $0.2D$ adjacent to the tension face shall contain $0.5\left(\frac{l_{clear}}{D} - 0.5\right)$ times the area of the tension steel:

(ii) A zone of depth $0.3D$ on either side of the mid-depth of the beam shall contain the remainder of the tension steel, evenly distributed.

When ratio $(l_{clear}/D) < 1.0$, the entire tension steel shall be evenly distributed over a depth of $0.8D$ measured from the tension face.

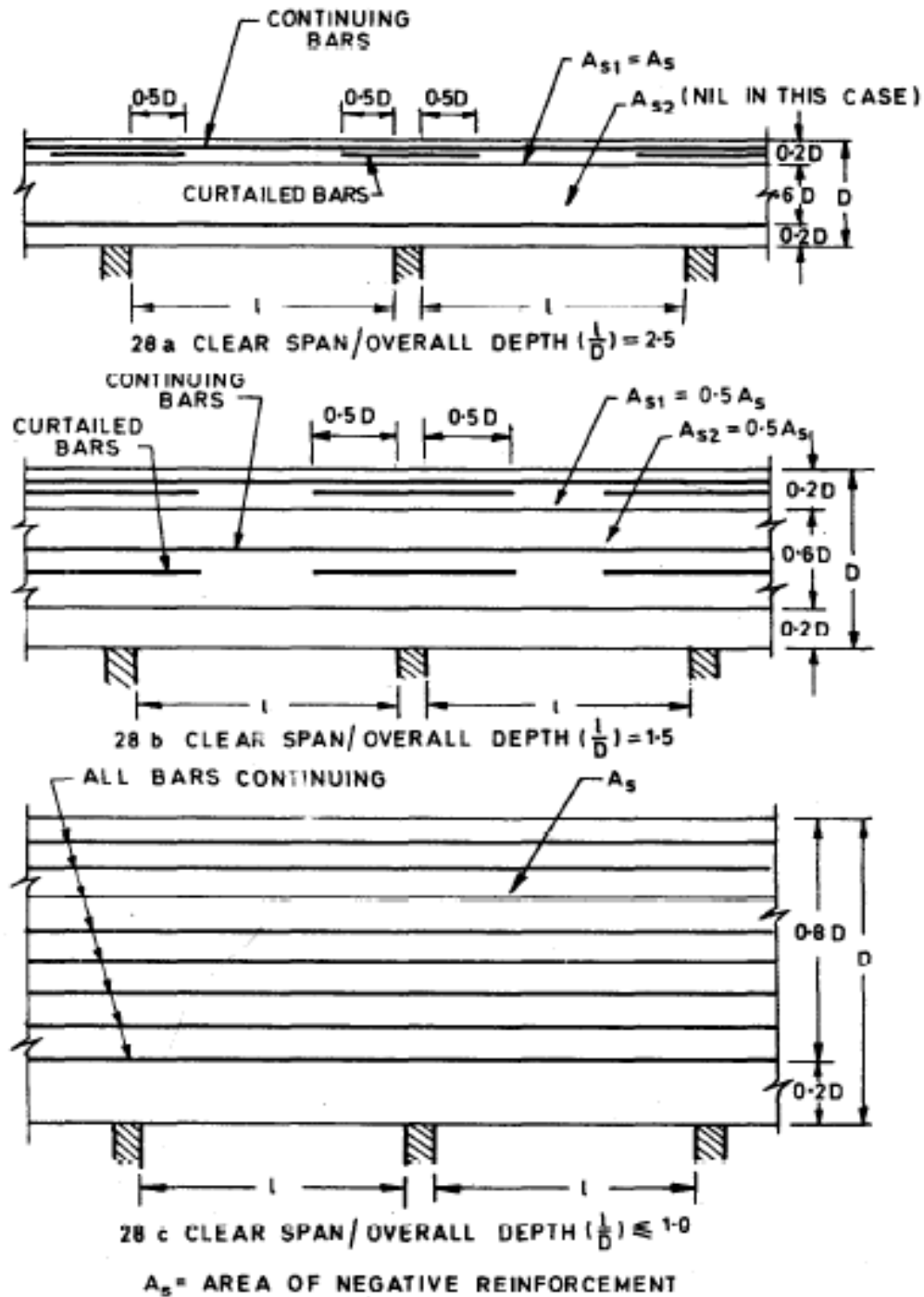


FIG. 60 NEGATIVE REINFORCEMENT DETAIL IN CONTINUOUS *DEEP* BEAMS

(3) Vertical Reinforcement:

If forces are applied to a Deep Beam in such a way that hanging action is required, then bars or suspension stirrups shall be provided to carry all the forces concerned.

(4) Side Face Reinforcement:

Side face reinforcement shall comply with requirements of minimum reinforcement of walls (as per 9.2.4.1(a)(4)).

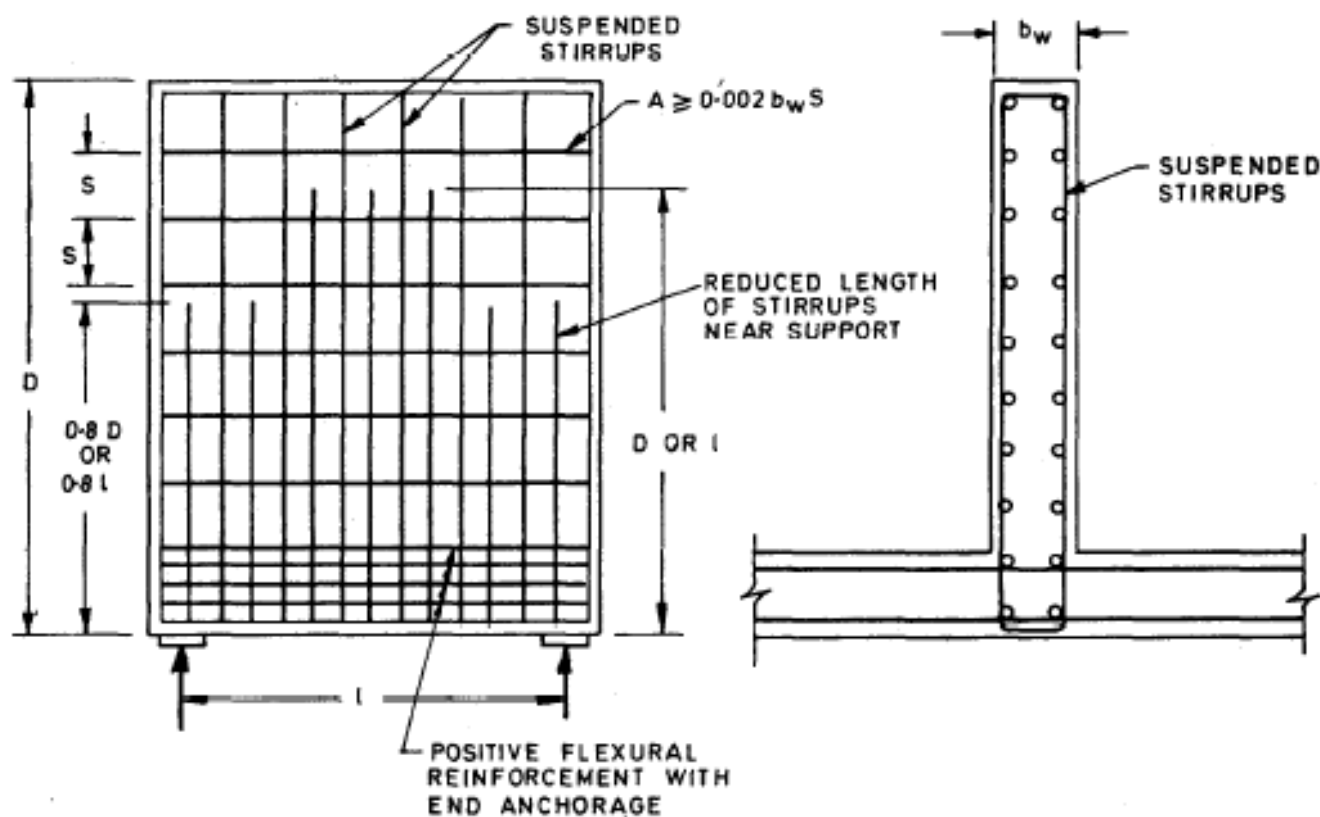
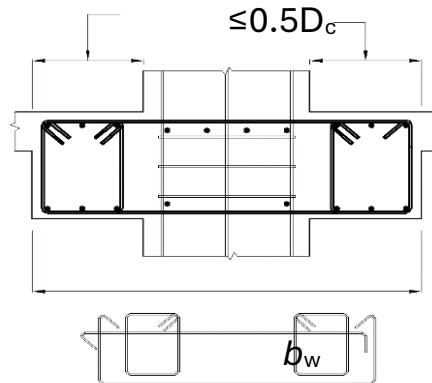


FIG. 61 SUSPENDED BARS IN SIMPLY SUPPORTED DEEP BEAMS

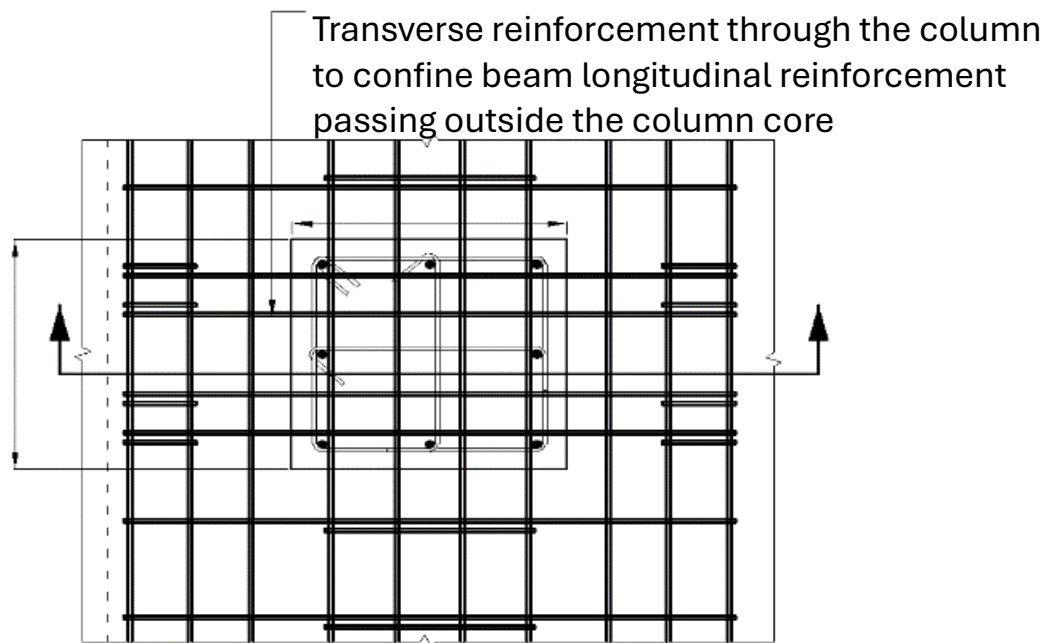
(c) Wide Beams

Beams with width more than that of the column it is framing into may be used, such that the width of beam b_w does not exceed twice the width of the column. But, in no case the projected portions of the wide beam on either side shall exceed 50% of the width of the column (Fig. 62).

When the width of the beam exceeds width of the column c_2 , transverse reinforcement shall be provided throughout the beam span including within the beam column joint.



Section A-A
(a)



Plan
(b)

FIG. 62 MAXIMUM EFFECTIVE WIDTH OF WIDE BEAM AND REQUIRED TRANSVERSE REINFORCEMENT

9.3.3.3 Columns and Beam-Columns

The general requirements for geometry shall be provided as specified in **9.2**.

a) Design

Columns shall be designed for axial force, shear force and bending moment to resist all forces transferred through them under the combination of loads specified in Table 3 for *Strength Criteria*. The capacity of the columns shall be estimated as specified in **9.3.2.4 and 9.3.2.7** at least at the critical sections specified in **9.2.5**.

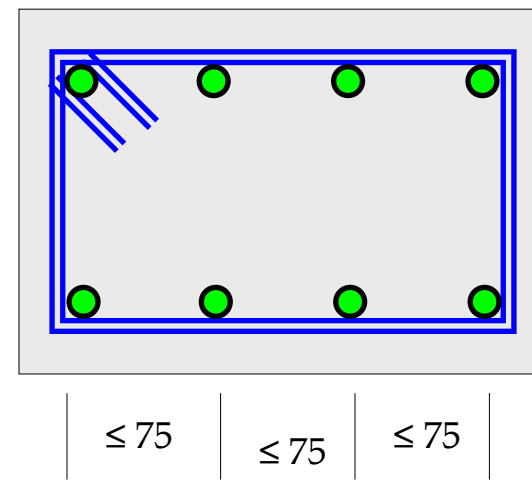
b) Detailing

The general requirements for reinforcement shall be provided as specified in **9.2**. In addition, the requirements given hereunder shall be complied with.

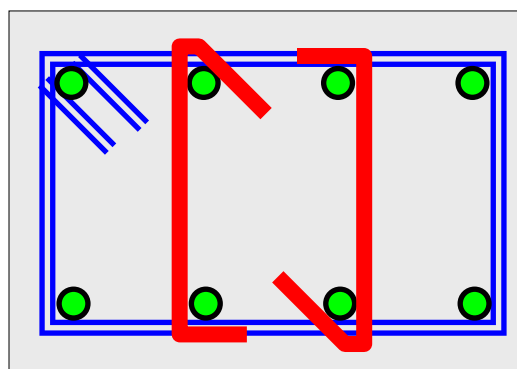
A RC compression member shall have

- (1) Longitudinal steel in columns enclosed within closed loop transverse reinforcement for effective lateral restraint, in compliance with the general requirements specified in **9.2**.
- (2) Transverse or helical reinforcement so disposed that every longitudinal bar nearest to the compression face has effective lateral support against buckling subject to the following provisions:
 - (a) The effective lateral support is given by transverse reinforcement either in the form of circular rings capable of taking up circumferential tension or by polygonal links (lateral ties) with internal angles not exceeding 135° . The ends of the transverse reinforcement shall be properly anchored.
 - (b) If the longitudinal bars are not spaced more than 75 mm on either side, transverse reinforcement need only to go round corner and alternate bars for the purpose of providing effective lateral supports (**Figure 63(a)**).
 - (c) If the longitudinal bars spaced at a distance of not exceeding 48 times the diameter of the tie are effectively tied in two directions, additional longitudinal bars in between these bars shall be tied in one direction by open ties (**Figure 63(b)**).

- (d) Where the longitudinal bars in a compression member are grouped (not in contact) and each group adequately tied with transverse reinforcement as specified in 9.2.2, the transverse reinforcement for the compression member as a whole may be provided on the assumption that each group is a single longitudinal bar for purpose of determining the pitch and diameter of the transverse reinforcement as specified in 9.2.2. The diameter of such transverse reinforcement need not exceed 20 mm.
- (3) Helical reinforcement shall be used only in piles, and not in columns, and when used, they shall comply with the following:
- (a) *Pitch* – Helical reinforcement shall be of regular formation with the turns of the helix spaced evenly and its ends shall be anchored properly by providing one and a half extra turns of the spiral bar. Where an increased load on the column on the strength of the helical reinforcement is allowed for, the pitch of helical turns shall be not more than 75 mm, nor more than $1/6^{\text{th}}$ of the core diameter of the column, nor less than 25 mm, nor less than 3 times the diameter of the steel bar forming the helix. In other cases, the requirements of 9.2.2 shall be complied with.
- (b) *Diameter* – The diameter of the helical reinforcement shall be in accordance with 9.2.2.1.



(a)



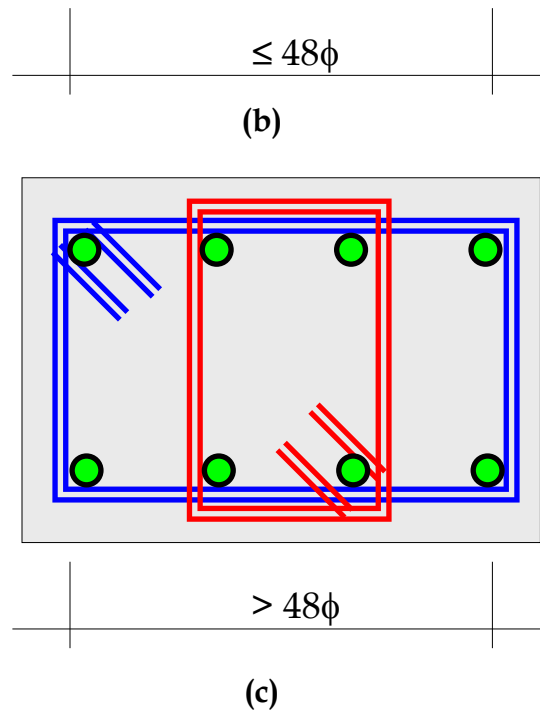


FIG. 63 Arrangement of transverse in reinforcement in columns

(4) In columns where longitudinal bars are offsets at a splice,

- (a) the slope of the inclined portion of the bar with the axis of the column shall not exceed 1 in 6, and the portions of the bar above and below the offset shall be parallel to the axis of the column.
- (b) Adequate horizontal support at the offset bends shall be treated as a matter of design, and shall be provided by metal ties, spirals, or parts of the floor construction. Metal ties or spirals so designed shall be placed near (not more than 8 times bar diameter from) the point of bend. The horizontal thrust to be resisted shall be assumed as 1.5 times the horizontal components of the nominal stress in the inclined portion of the bar.
- (c) Offset bars shall be bent before they are placed in the forms. Where column faces are offset 75 mm or more, splices of vertical bars adjacent to the offset face shall be made by separate dowels overlapped as specified in 9.2.2.8 (Figure 64).

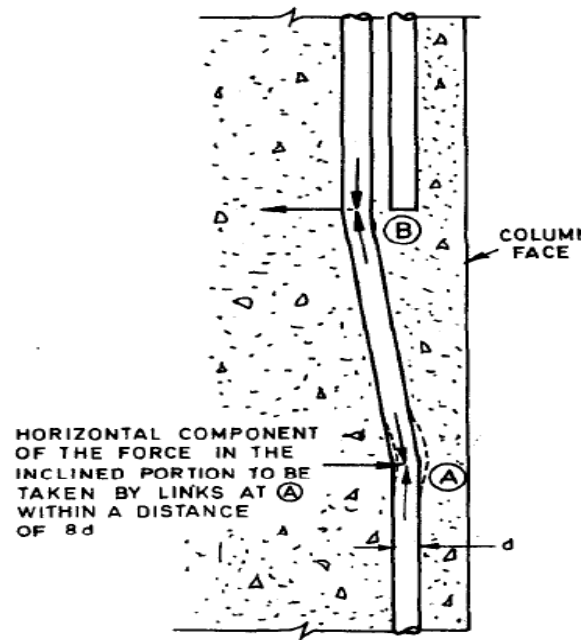
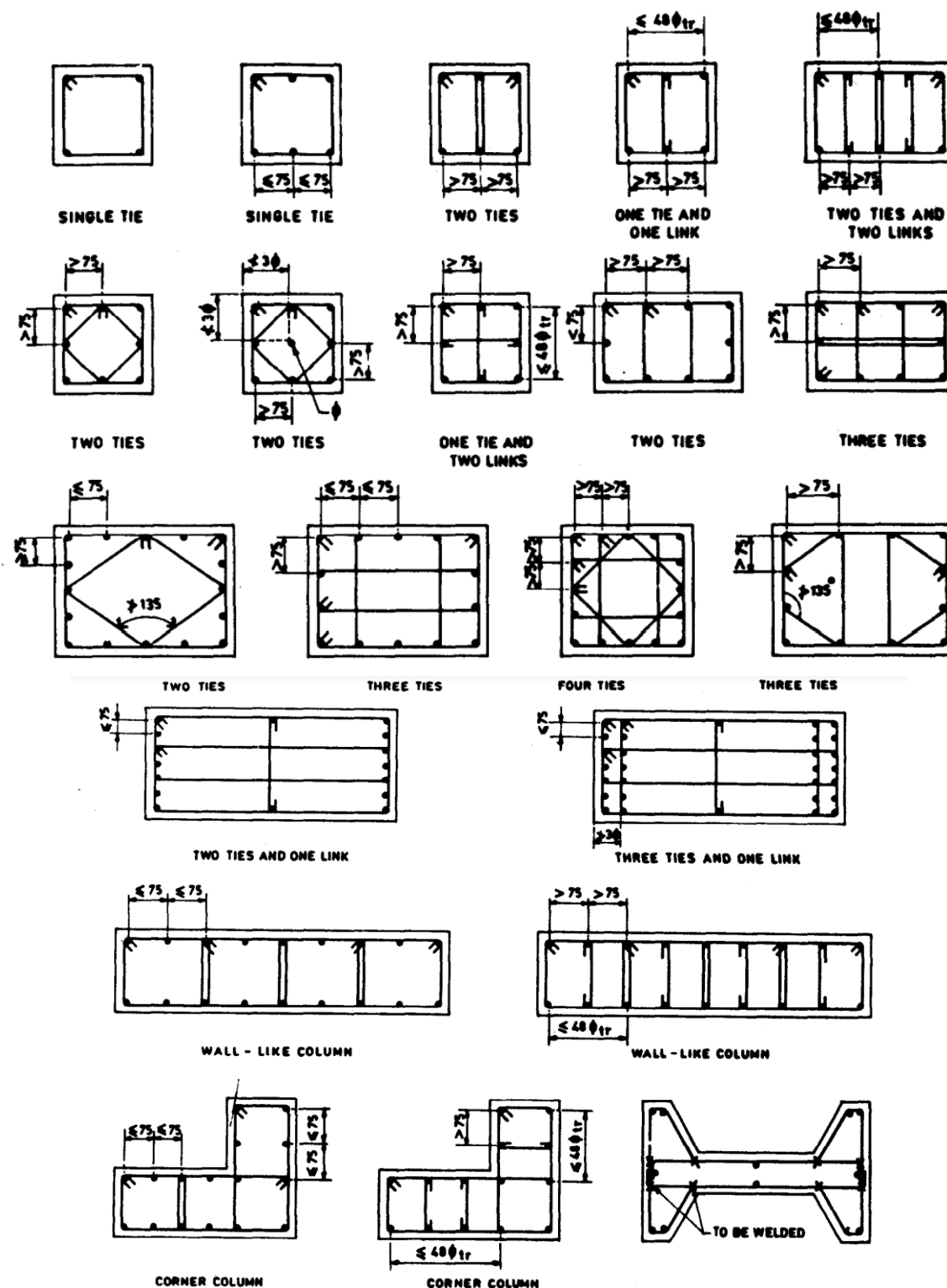


FIG. 64 Unbalanced force at location of cranking of reinforcement bar in a column



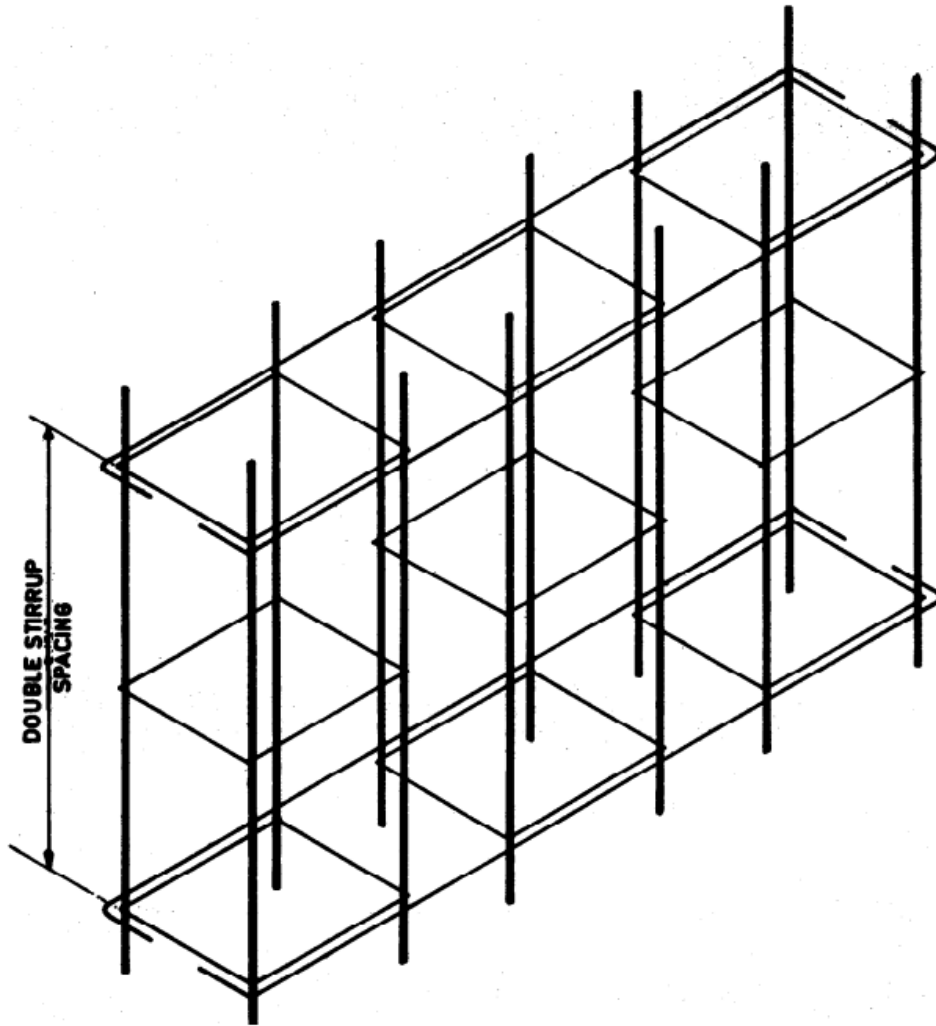


FIG. 66 WRONG DETAILING OF REINFORCMENT IN WIDE COLUMNS

9.3.3.4 *Structural walls*

When structural walls are provided in a structure, they shall be provided in both plan directions. Along each plan direction, they shall be so located and proportioned that they do not create themselves undue torsional eccentricity in plan of the structure.

In a structure with only concrete walls, walls other than those designated to resist the lateral load, need not meet these requirements. But, if displacement-compatibility induced damages are incurred in such walls, the structure shall meet all requirements of integrity criteria laid down in 5.2.5. In a structure with only concrete walls, the coupling beams between the structural walls shall be designed to resist all forces transferred thorough them under the combination of loads specified in Table 3 for strength criteria.

The general requirements for geometry and reinforcement specified in 9.2 shall be complied with. In addition, the additional requirements specified hereunder shall be.

9.3.3.4.1 *Buildings with no openings*

9.3.3.4.1.1 *Stress-resultant demands*

- a) A structural wall shall be designed to resist the design lateral forces acting on the structure in proportion to their in plane stiffness, unless required otherwise. It shall be capable of carrying axial load, bending moment and shear force imposed on it along both plan directions. Thus, these walls shall be designed and detailed (with vertical and horizontal reinforcement on each face) for bending about both major and minor axis.
- b) The structural wall shall have the bending moment diagram about its major axis along the height like that of a vertical cantilever, with small jumps at the floor levels; the bending moment at the roof level need not be zero, owing to the presence of a beams at that level. When the bending moment diagram (about its major or minor axis) of a vertical member along the height changes sign at any level along its height, then it shall be designed like a column.
- c) Structural Walls shall be designed for axial force P , shear force V and bending moment M to resist all forces transferred thorough them under the combination of loads specified in Table 3 for strength criteria. The capacity of the structural walls shall be estimated as specified in 9.3.2.4 and 9.3.2.7 at least at the critical sections specified in 9.2.5.

9.3.3.4.1.2 *Design*

a) *Geometry*

A wall designated as a structural wall shall have at least:

- 1) 200mm thickness, and

- 2) The ratio H_{we}/t_w of effective height to thickness shall not exceed 30, where the effective height of a braced wall shall be given by **8.8.2.2.2**.
- 3) Structural walls shall be built from foundation to roof, unless when they are terminated at a lower height, the structure does not become irregular as per IS 1893 (Part 2).
- 4) 2 curtains of reinforcement, where a curtain implies a pair of vertical and horizontal steel bars on each face of the wall. Concrete Walls with only one curtain of horizontal and vertical bars in the entire cross-section shall not be considered as structural walls.

b) Capacity Estimation

The design of concrete structural walls without any opening or with nominal openings (of area less than 10 percent of the wall area in each storey) shall comply with the provisions given hereunder:

1) Design for axial force and bending moment

- i. Structural walls shall be designed for the combined action of axial force and bending moment shall be estimated as specified in **9.3.2.7**.
- ii. the design bending moment capacity M_L of a structural wall shall be estimated as specified in **9.3.2.4**. the area of horizontal reinforcement so estimated shall not be less than the minimum area specified in **9.2.4.1(b)(3)**.

2) Design for horizontal shear force

- i. The critical section for maximum shear force in a structural wall in its in-plane direction shall be taken at a height from the base of $Max[0.5L_w; 0.5H_w]$, where L_w and H_w are the overall length and overall height of the structural wall.
- ii. The nominal shear stress τ_D in a structural wall in its in-plane direction shall be estimated as:

$$\tau_D = \frac{V_D}{(0.8L_w)t_w},$$

where

V_D = Design shear force on the wall obtained from combinations of loads specified in Table 3 for strength criteria;

t_w = Thickness of structural wall, and

L_w = Overall length of the structural wall.

- 3) The nominal shear stress τ_D shall not exceed the maximum design shear strength $(\tau_{Lc,wall})_{max}$ of concrete in structural walls specified in **9.2.6.1(d)**.

- 4) The design shear force capacity V_L of a structural wall shall be estimated as specified in 9.3.2.5. The area of horizontal reinforcement so estimated shall not be less than the minimum area specified in 9.2.4.1(b)(3).

c) *Boundary Elements*

- 1) Boundary elements shall have geometry as specified hereunder:
 - i. The width of the boundary shall be at least the same thickness as that of the wall web (i.e., the boundary element is concealed).
 - ii. The length of the boundary shall be larger of twice the thickness of the wall web, 0.15 times the length of the wall, and 500 mm.
- 2) Boundary elements shall be provided along the vertical boundaries of walls, when the extreme fiber compressive stress in the wall exceeds $0.20f_{ck}$ due to factored gravity loads plus factored earthquake force. Boundary elements may be discontinued at elevations where extreme fiber compressive stress becomes less than $0.15f_{ck}$. Extreme fiber compressive stress shall be estimated using a linearly elastic model and gross section properties.
- 3) A boundary element shall have adequate axial load carrying capacity, assuming short column action, to enable it to carry axial compression arising from factored gravity load and lateral seismic shaking effects.
- 4) The load factor for gravity load shall be taken as 0.8, if gravity load gives higher axial compressive strength of the boundary element.
- 5) The vertical reinforcement in the boundary elements shall not be less than 0.8 percent and not greater than 6 percent; the practical upper limit would be 4 percent to avoid congestion.
- 6) Boundary elements, where required as per 8.3.3.1 (a), shall be provided with special confining reinforcement throughout their height, given by

$$A_{sh} = 0.05s_v h \left(\frac{f_{ck}}{f_y} \right),$$

and have a spacing not more than 1/3 of minimum member dimension of the boundary element; 6 times diameter of the smallest longitudinal reinforcement bars; and 100 mm but may be relaxed to 150 mm if maximum distance between cross-ties/parallel legs of links or ties is limited to 200 mm, but need not be less than 100 mm.

- 7) Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per 8.2.3.3 (c) (1).
- 8) When a beam frames into the minor axis of a wall, then boundary elements shall be provided at such locations along the full height of the wall. The design of such a boundary element shall meet the requirement of minimum

column dimension perpendicular to the minor axis of the walls given in 8.2.1.1 and minimum column-to-beam strength ratio given in 8.2.2.4(a).

d) *Detailing*

- 1) Structural walls intended to resist P_L , V_L and M_L shall have at least two curtains of vertical and horizontal steel on each face.
- 2) The minimum reinforcement in the vertical and horizontal directions specified in 9.2.4.1 may not be sufficient to provide adequate resistance to the effects of shrinkage and temperature.
- 3) The structural reinforcements along vertical and horizontal directions obtained when designed for $P_L - M_L$ and V_L , respectively should account for the effects of shrinkage and temperature also.

9.3.3.4.1.3 *Structural walls with openings*

Structural Walls with openings shall be avoided to the extent possible. When openings are mandatory in Structural Walls, they shall be limited in number and size, and shall be placed symmetrically in the middle of the wall. The design of concrete structural walls with openings of significant size shall comply with the provisions of IS 13920 (Part 2/Sec 3).

9.3.3.4.1.4 *Buildings with only structural walls*

Buildings constructed with only structural concrete walls shall be designed such that there is no tension in them under *in-plane* deformations arising from combined design axial force and design bending moment demands acting under the effects of combinations of loads specified in 6.4. For the purpose of checking the tension in structural walls, provisions of 8.8.2.2.2 shall be used.

9.3.3.5 *Foundations*

9.3.3.5.1 *Spread Footings (Individual and Combined)*

Footings shall be designed to:

- a) Sustain design vertical load P_L , horizontal load V_L and bending moment M_L imposed by the structure above, and the reaction induced from the soil underneath, by providing adequate *flexural strength, shear strength and development lengths*;
- b) Ensure that any *settlement* that may occur shall be as nearly uniform as possible, and

- c) Ensure that the *bearing pressure* induced in soil does not exceed the safe bearing capacity of the soil (as required in IS 1904).

a) *Geometry of Footing*

Footings can be flat or stepped. Footings are designed to act as a unit, and hence shall be constructed to assure the same. In PC or RC footings, the thickness at the edge shall be not less than 150 mm for footings on soils, and the thickness above the tops of pile cap not less than 300 mm for footings on piles.

The cross-section effective in bending compression shall be limited to the area above the axis of bending. In stepped footings, the angle of slope or depth and location of steps shall be such that the design requirements are satisfied at every section.

In plain concrete pedestals, the angle α between line joining extreme bottom edge of footing and outer edge of column resting on the pedestal and the horizontal (Fig. 67) shall comply with the condition:

$$\tan \alpha > 0.9 \sqrt{1 + \frac{100q_0}{f_{ck}}}$$

where

q_0 = Maximum bearing pressure induced in soil underneath the pedestal, and

f_{ck} = Characteristic strength of concrete.

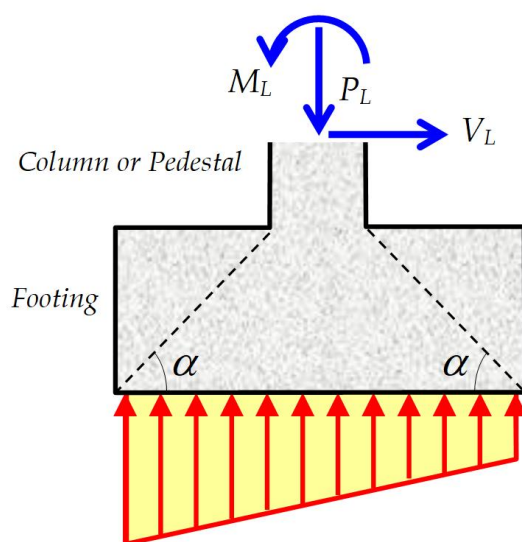


FIG. 67 PERMISSIBLE ANGLE α CONTAINED BY THE GEOMETRY OF PC PEDESTALb) *Bending Moments and Shear Forces*

The bending moment at any vertical sectional plane of the footing shall be determined as the moment of the forces acting over the entire area of the footing to the cantilever side of that sectional plane (which extends completely across the footing).

The critical section for estimating the design bending moment of an isolated footing which supports a column, pedestal or wall, shall be taken:

- 1) At the face of the column, pedestal or wall, in footings supporting a concrete column, pedestal or wall;
- 2) Halfway between the centerline and the edge of the wall, in footings under masonry walls; and
- 3) Halfway between the face of column or pedestal and the edge of gusseted base, in footings under gusseted bases.

When doing so, bending moments and shear forces shall be calculated assuming the reaction from any pile to be a concentrated force at the centre of the pile, in footings resting on piles; and the critical section for bending moment and shear force shall be taken at the face of the column or pedestal taken as a square inscribed within the perimeter of the circular or octagonal columns or pedestals, in footings that support circular or octagonal columns or pedestals.

The design for shear force of a footing is governed by the more severe of the following conditions:

- 1) *Beam Shear*: One-way action of the footing with a potential diagonal crack extending in a plane across the entire width, with the critical section located at a distance equal to the effective depth of footing from the face of the column, pedestal or wall supported by it;
- 2) *Punching Shear – Symmetrical*: Two-way action of the footing with potential diagonal crack extending along the surface assumed to be a symmetrically oriented truncated *cone* around circular column above or truncated pyramid around rectangular column above, with the critical section located at a distance equal to half the effective depth of the footing from the face of the column that carries only axial load.

- 3) *Punching Shear – Unsymmetrical*: Two-way action of the footing with potential diagonal crack extending along the surface assumed to be two anti-symmetrically oriented half truncated cones one around each side around circular column above or half truncated pyramids one around each side rectangular column above, with the critical section located at a distance equal to half the effective depth of the footing from the face of the column that carries both axial load and bending moment.

c) *Tensile Reinforcement*

The total tensile reinforcement at any section (across the entire width of the footing) shall provide a *limiting* bending moment capacity M_L at least equal to the design bending moment M_D on the section. The total tensile reinforcement so obtained shall be distributed:

- 1) In one-way reinforced footings, uniformly across the full width of the footing along each of the two directions in plan;
- 2) In two-way reinforced footings, uniformly across the full width of the footing, along each of the two directions in plan; and
- 3) In two-way reinforced footings, uniformly across the full width of the footing, along the long direction in plan and non-uniformly across the full width of the footing, along the short direction in plan as below:
 - i. A fraction β of the total reinforcement uniformly distributed across a central band equal to the width of the footing marked along the length of the footing, and
 - ii. The remaining fraction $(1 - \beta)$ of the total reinforcement uniformly distributed in the outer portions of the footing, where:

$$\beta = \frac{2}{1 + \left(\frac{L}{B}\right)},$$

where

L = Longer length of the footing in plan, and

B = Shorter length of the footing in plan.

d) *Bearing Stress on top of the Footing*

The maximum bearing stress $f_{b,\max}$ demand in concrete at the top of the footing (or at the base of a column or pedestal) under the action of factored design forces (obtained as per load combinations given in Table 3) shall comply with the condition:

$$f_{b,\max} < \text{Min} \left[2f_{br}; f_{br} \sqrt{\frac{A_1}{A_2}} \right],$$

where

f_{br} = Design Bearing Strength of concrete given by 9.2.6.1(a),

A_1 = Area of the base of the frustum of a cone or pyramid, contained wholly within the footing, having column or pedestal as its upper area, and having a slope of 1 vertical to 2 horizontal (Fig. 68), and

A_2 = Area of the column or pedestal.

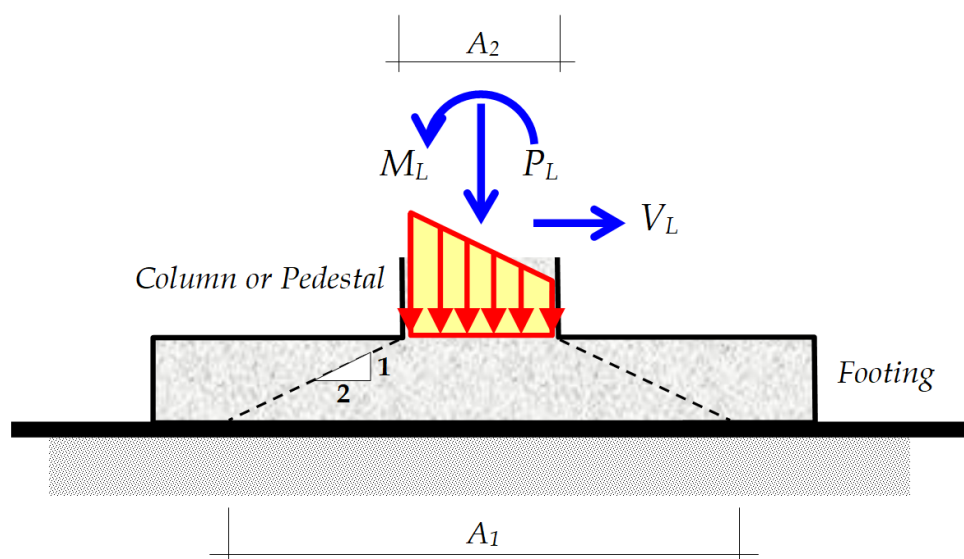


FIG. 68 BEARING STRESS ON TOP OF THE FOOTING

The maximum bearing stress $f_{b,\max}$ does not comply with the above the condition:

- 1) Reinforcement shall be provided for developing the excess force by additional dowels in the footing;
- 2) Development length of such dowel reinforcement shall be sufficient to transfer the compression or tension to the footing;
- 3) Such dowel reinforcement shall
 - i. Be at least 0.5 percent of the cross-sectional area of the column or pedestal,
 - ii. Have a minimum of four bars, and
 - iii. Have diameter not exceeding the diameter of the column or pedestal bars by more than 3 mm. When column or pedestal bars are of diameter 36 mm or larger, dowel bars shall be of diameter smaller than that of the column or pedestal bars.

9.3.3.5.2 Raft Foundations

Raft foundations shall comply with the following:

- a) Longitudinal flexural reinforcement in raft foundations shall be placed in two curtains, one on top face and another on the bottom face, along both plan directions. This shall meet the minimum longitudinal reinforcement requirements specified in **9.2.4.1(a)(6)**.
- b) In one-way rafts, flexural shear shall be designed for across the full width of the raft.
- c) In two-way rafts, both flexural shear and punching shear shall be designed for mandatorily.

9.3.3.5.3 Piles and Pile Caps

The requirements specified in IS 2911 (Part 1/Sec 1) shall be complied with.

9.3.3.5.4 Piled-Rafts

The requirements specified in IS 2911 (Parts 1 and 3) shall be complied with.

9.3.3.5.5 Wells and Well Caps

The relevant specifications of Indian Roads Congress shall be complied with.

9.3.3.6 Special Structural Elements

Adequate end 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 beams and members in which the tension reinforcement is not parallel to the compression face.

9.3.3.6.1 Staircases

The recommendations related to strength, deflection and crack control for beams and slabs shall be applicable for staircases also. Further, the following shall be applicable.

a) Distribution of loads on stairs

In stairs with open wells, where spans partly crossing at right angles occur, the load on areas common to any two such spans may be taken as 50 percent in each direction as shown in Fig. 69(a). Where flights or landings are embedded into walls for a length of not less than 110 mm and are designed to span in the direction of

the flight, a 150 mm strip may be deducted from the loaded area and the effective breadth of the section increased by 75 mm for purposes of design [see Fig. 69(b)].

b) Depth of section

The depth of section shall be taken as the minimum thickness perpendicular to the soffit of the staircase.

c) Modeling of staircases

Staircases shall be modeled to capture the stiffness introduced between two successive floor slabs by the inclined slab and/or inclined spine beams. But, when sliding joint detailing is adopted at the junction of the floor slab and inclined slab and/or inclined spine beams, the analytical model shall be modified accordingly to not introduce any stiffness between two successive floor slabs.

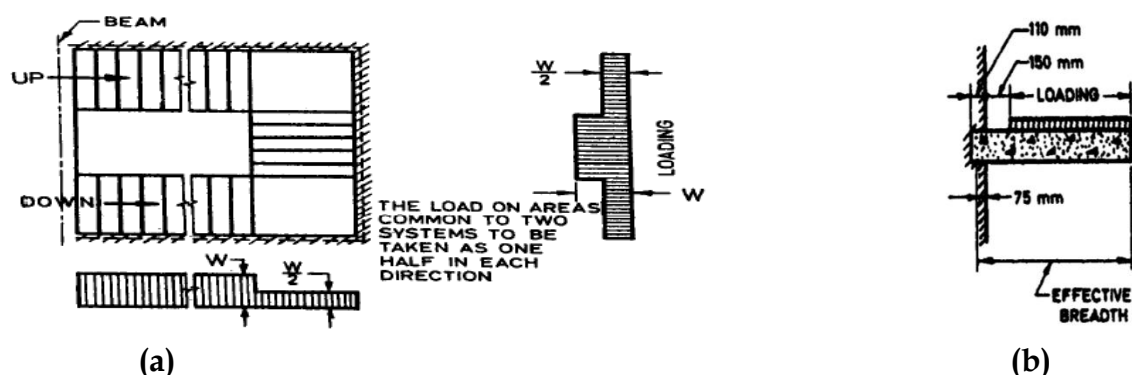


FIG. 69 ASSESSMENT OF LOADS ON STAIR SLABS: (A) OPEN WELL STAIRCASE, AND (B) STAIRS BUILT INTO WALL

9.3.3.6.2 Corbels

9.3.3.6.2.1 Corbels may be designed by using strut & tie method. When doing so, the following shall be complied with:

- The inclination of strut with respect to the main tie shall lie between 45° and 68° .
- The strut shall be designed such that the concrete compression stress in it does not exceed that given in 9.3.6.
- The reinforcement, corresponding to the ties shall be fully anchored beyond the node under the bearing plate, by using U-hoops or anchorage devices such as couplers, unless a length $l_{b,net}$ is available between the face of the vertical member and the front of the corbel. $l_{b,net}$ shall be measured beyond the full width of

compressive strut, and shall be fully anchored at the other end of the vertical member to which the corbel is attached.

9.3.3.6.2.2 Alternately, a corbel may be designed by the following prescriptive method, if the following conditions are satisfied:

- a) The distance a_v of application of factored design vertical force V_F is less than its effective depth d determined at the face of the vertical member (Fig. 70),
- b) The Corbel is subjected to a net downward load V_F , and
- c) The Corbel is subjected to factored design horizontal force H_F acting away from the vertical member is less than V_F .

9.3.3.6.2.3 In the prescriptive method, a corbel shall be designed in accordance with the following provisions:

- a) The design of the Corbel shall depend on the geometry of the corbel. If:
 - 1) $a < 0.5D$: The Corbel shall be provided with closed horizontal stirrups with area A_{SH} shall be provided in addition to the main tension reinforcement A_h at the top of the Corbel. Here, A_{SH} shall comply with:

$$A_{SH} > 0.5A_h,$$

which shall be provided in upper two-third of the corbel depth d .

- 2) $0.5D < a < D$: The Corbel shall be provided with closed vertical stirrups with area A_{SV} shall be provided in addition to the main tension reinforcement at the top of the Corbel and the closed horizontal or inclined stirrups. Here, A_{SV} shall comply with:

$$A_{SV} = \frac{0.5V_F}{0.87f_y}.$$

Also, the Corbel shall be designed such that the concrete compression stress in the strut does not exceed that given in **9.3.6**.

- 3) $a > D$: The Corbel shall be designed as a cantilever flexural member.
- b) If the corbel has varying depth, then the depth d_0 at the outside edge of the corbel shall not be less than $0.5d$, half the depth at the face of the support. In such a case, The favourable contributions from inclined compression chord shall not be considered when resisting the factored loads V_F and H_F .
- c) Critical section at the face of the support shall be designed to resist simultaneously V_F , H_F , and a factored moment $V_F a_v + H_F(h - d)$.

- d) Design of shear friction reinforcement $A_{s,vf}$ to resist V_F shall be in accordance with ____.
- e) The nominal shear stress τ_v at the face of the support due to factored design vertical force V_F shall be less than the design maximum shear strength $\tau_{c,max}$ of concrete given in **Table** ____.
- f) Reinforcement A_{st} required to resist the moment $V_F a_v + H_F(h - d)$ shall be estimated as per ____.
- g) Horizontal forces H_F , when applied in addition to the vertical load V_F , shall require additional reinforcement in the tie. Reinforcement A_h to resist H_F shall be determined from $0.87 f_y A_h > H_F$. Here, H_F shall not be taken less than $0.2V_F$, unless provisions are made to avoid H_F from acting outwards from the vertical member. H_F shall be regarded as *Imposed Load*, even if it acts outwards from the vertical member owing to restraint of creep, shrinkage or temperature change.
- h) Area of primary reinforcement A_{sc} shall be such that

$$A_{sc} \geq \text{Max} \left[(A_{st} + A_h); \left(\frac{2}{3} A_{s,vf} + A_h \right) \right].$$

- i) Total area A_{sv} of closed stirrups or ties parallel to primary reinforcement shall not be less than $0.5(A_{sc} - A_h)$. Distribute A_{sv} uniformly within a distance $\frac{2}{3}d$ adjacent to the primary reinforcement.
- j) A_{sv}/bd shall not be less than that specified in **9.2.4.1(b)**.
- k) At front face of the corbel, primary tension reinforcement shall be anchored by:
 - 1) Bending primary tension reinforcement back to form a horizontal loop; OR
 - 2) Any other means of positive anchorage.
- l) Bearing area on the corbel shall not project beyond straight portion of primary tension reinforcement, nor project beyond interior face of transverse anchor bar, if provided.
- m) The effects of torsion, if any, shall be catered to in accordance with **8.7.1.6**.

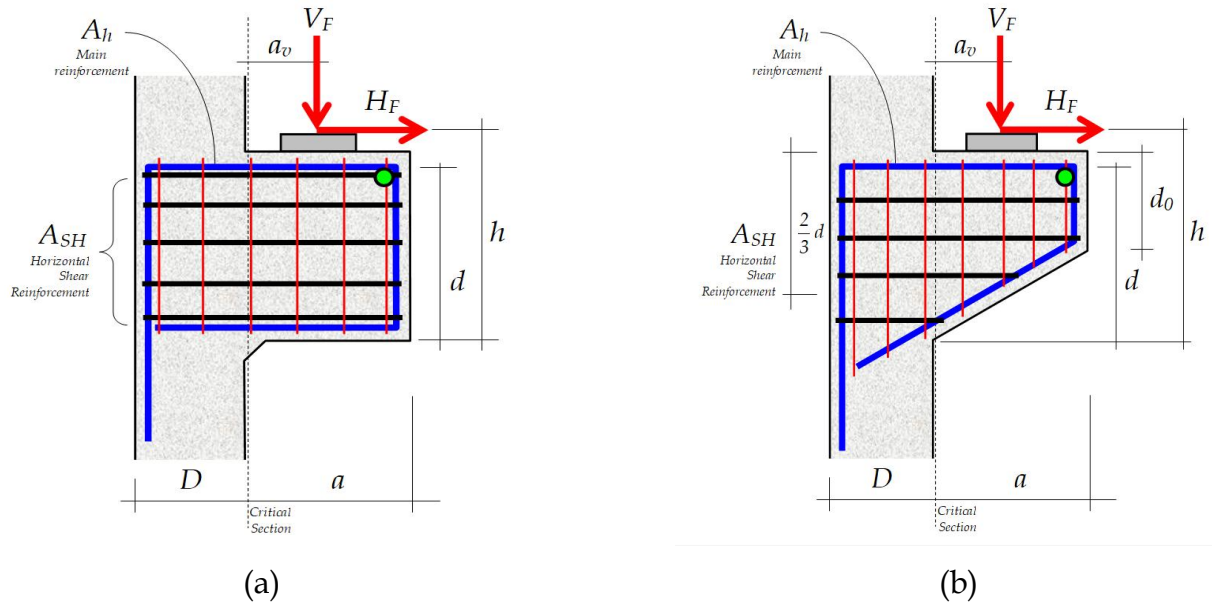


FIG. 70 GEOMETRY OF CONCRETE CORBEL WITH: (A) UNIFORM DEPTH, AND (B) VARYING DEPTH

9.3.3.7 Joints

The..... (to be discussed).

(a) Beam-to-Beam Joints

The

(b) Beam-to-Slab Joints

The

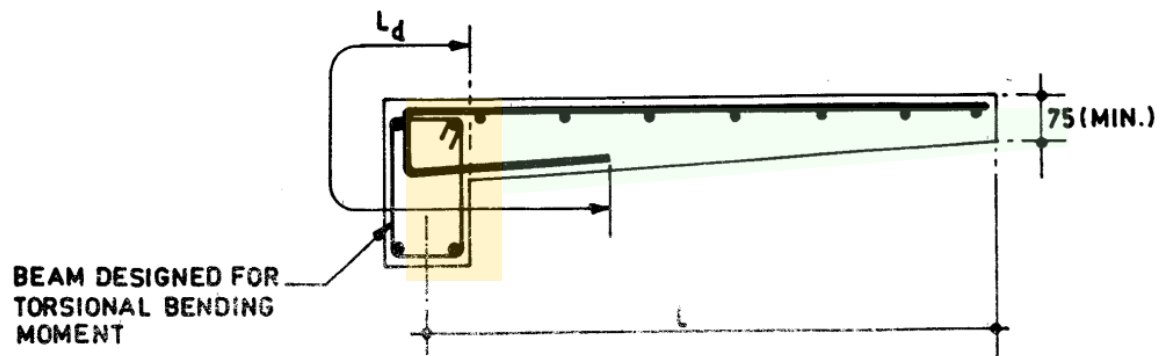


FIG. 71 DETAILING OF REINFORCEMENT IN A BEAM-CANTILEVER SLAB JOINT

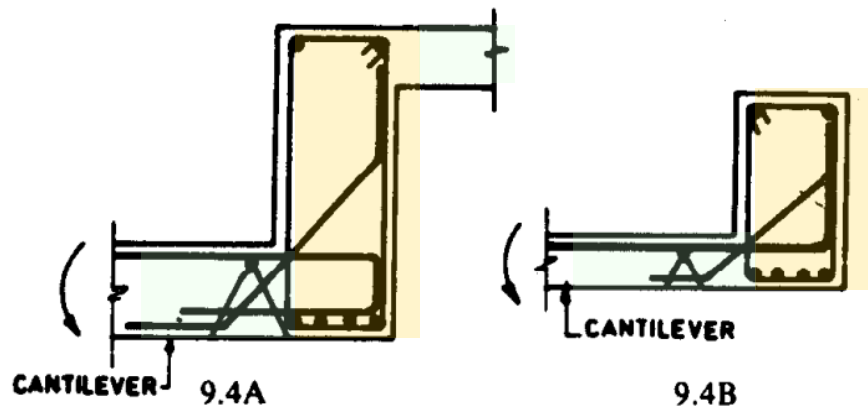
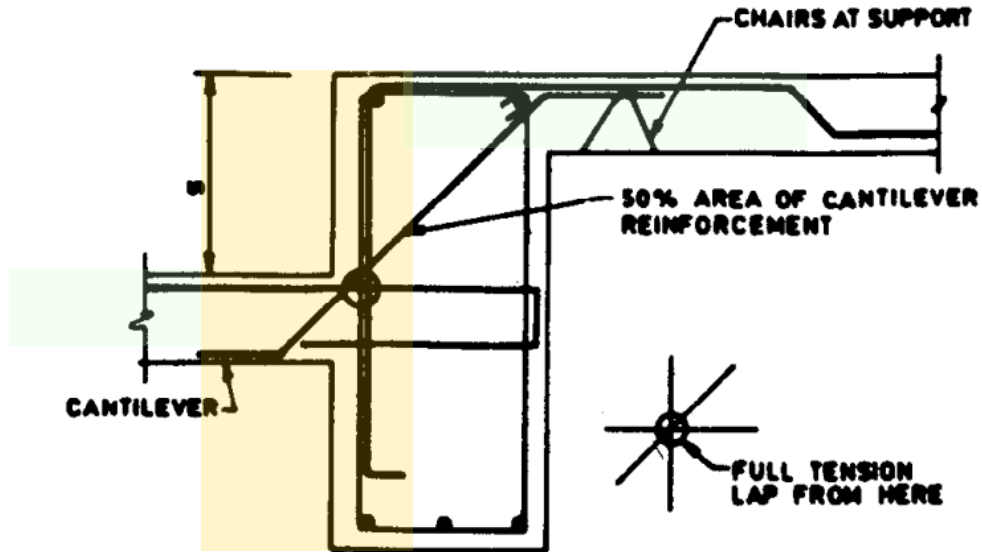


FIG. 72 DETAILING OF REINFORCEMENT IN A BEAM-SUNKEN SLAB JOINT

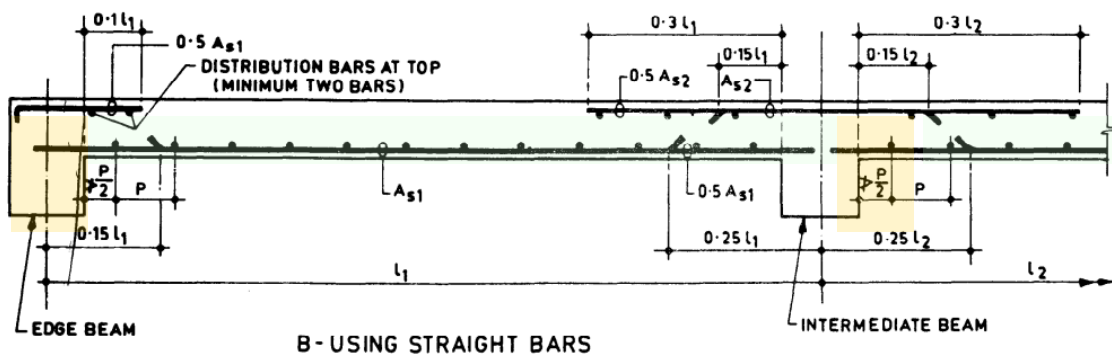


FIG. 73 DETAILING OF REINFORCEMENT IN A END BEAM-SLAB AND INTERIOR BEAM-SLAB JOINTS

(c) Beam-to-Column Joints

The design distortional shear stress demands τ_{DJX} and τ_{DJY} acting horizontally along principal plan directions X and Y of the joint shall be estimated for each load combination as:

$$\tau_{DJX} = \frac{V_{DJX}}{b_1 d_1}, \text{ and}$$

$$\tau_{DJY} = \frac{V_{DJY}}{b_1 d_1},$$

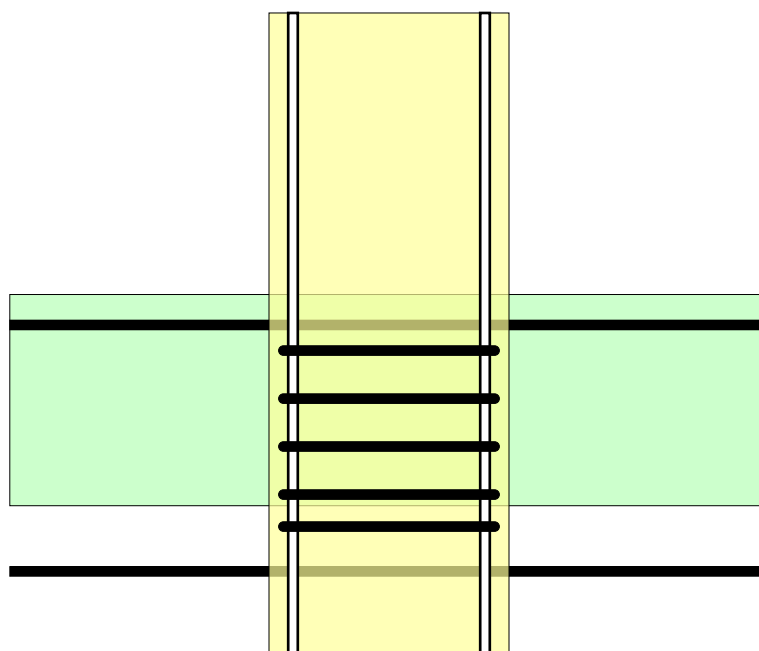
where

b_1 = Center to center distance between corner bars of the column along X-direction, and

d_1 = Center to center distance between corner bars of the column along Y-direction.

τ_{DJX} and τ_{DJY} shall not exceed the values given in 9.2.6.1.

The reinforcement detailing shall be as given in Fig. 74.



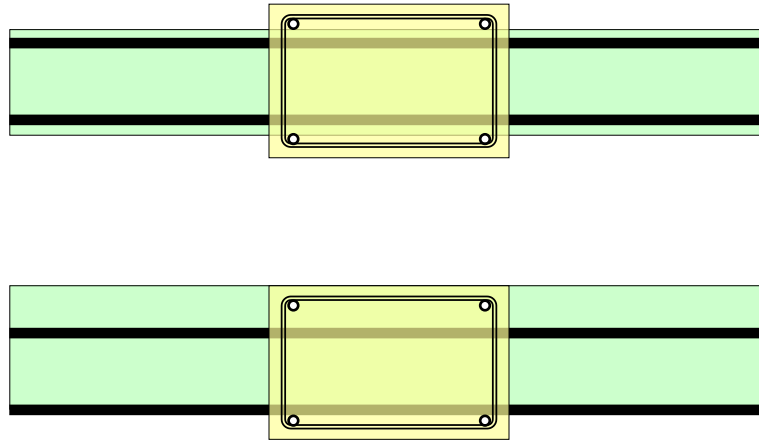


FIG. 74 DETAILING OF REINFORCEMENT IN BEAM-COLUMN JOINTS

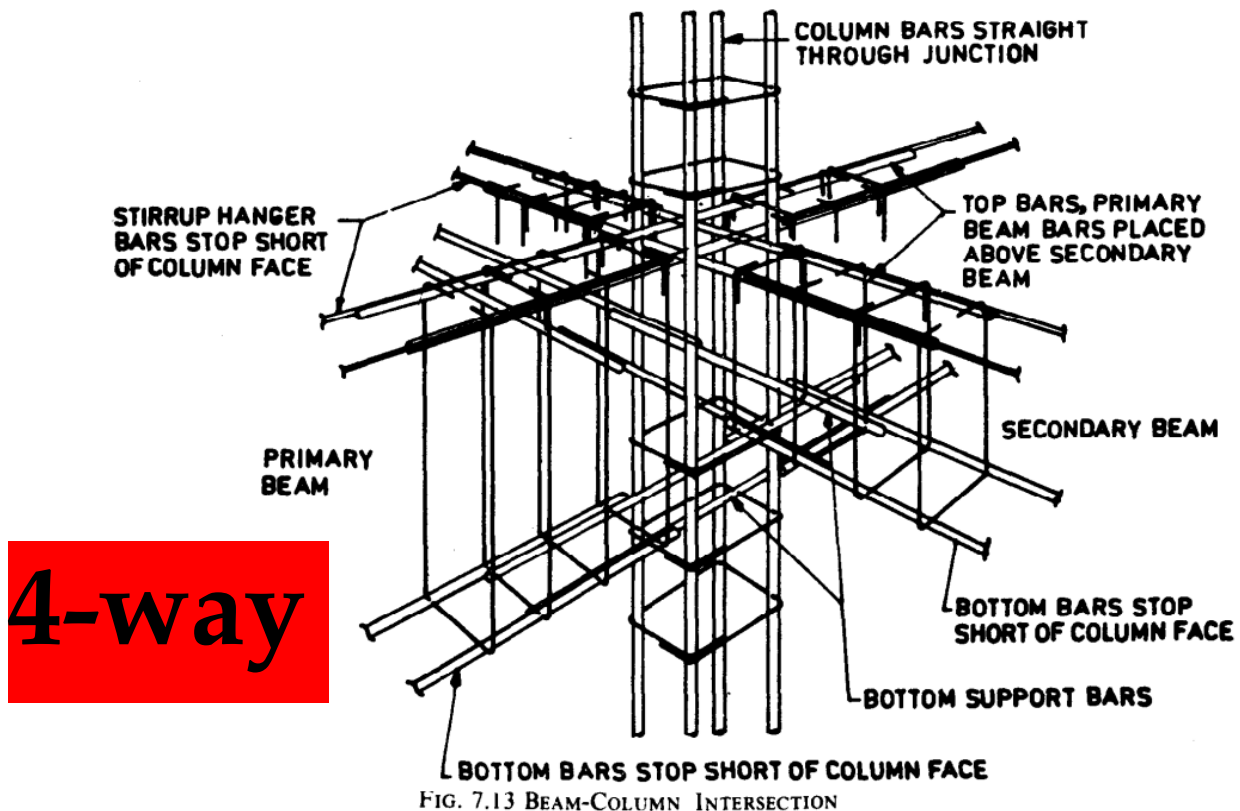


FIG. 75 DETAILING OF REINFORCEMENT IN A 4-WAY BEAM-COLUMN JOINT

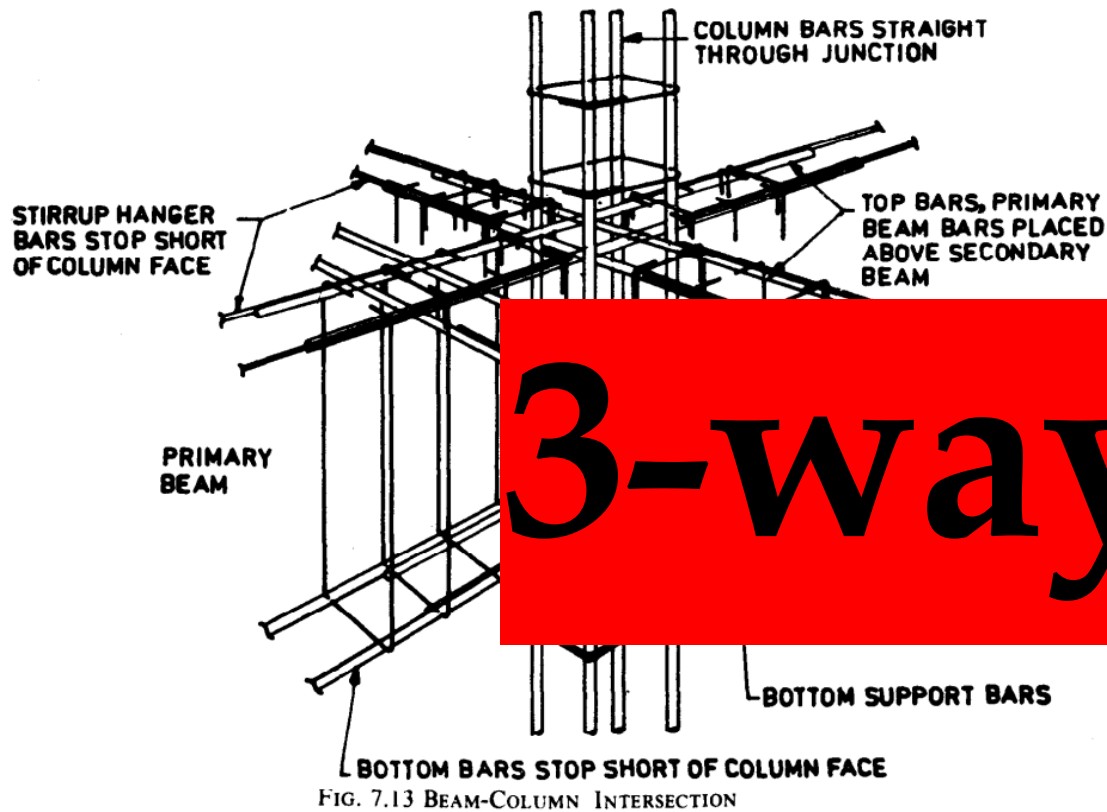


FIG. 76 DETAILING OF REINFORCEMENT IN A 3-WAY BEAM-COLUMN JOINT

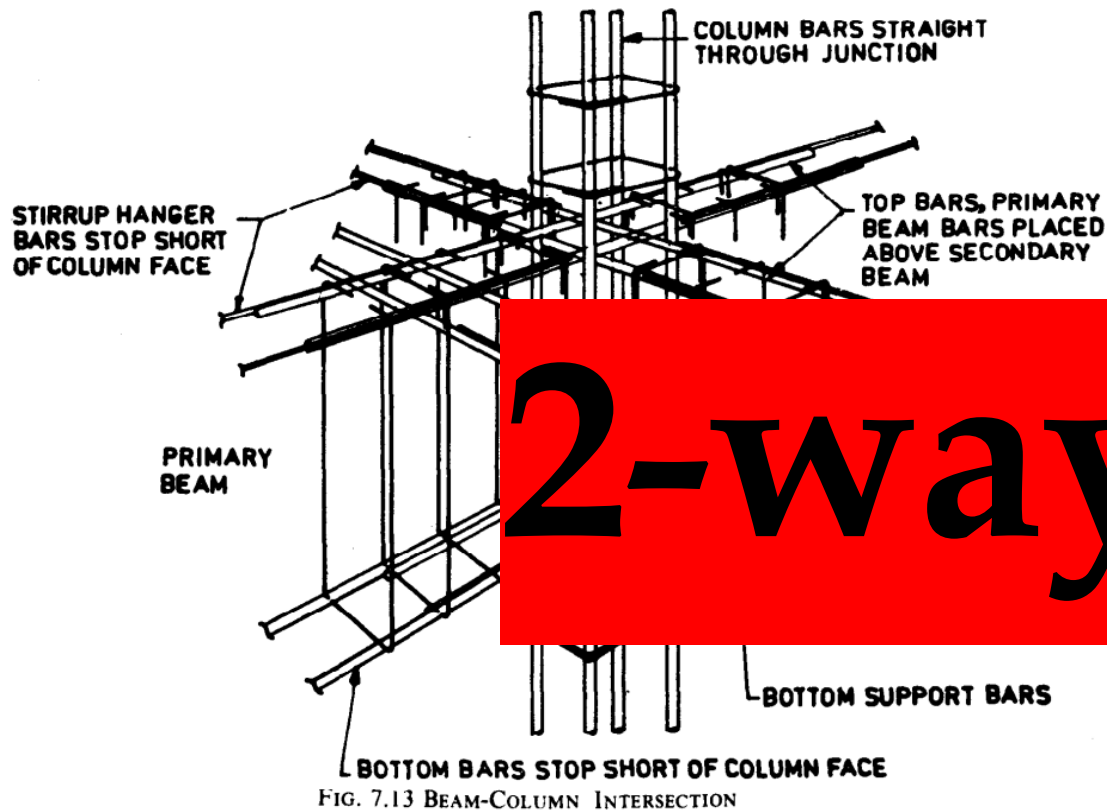


FIG. 77 DETAILING OF REINFORCEMENT IN A 4-WAY BEAM-COLUMN JOINT

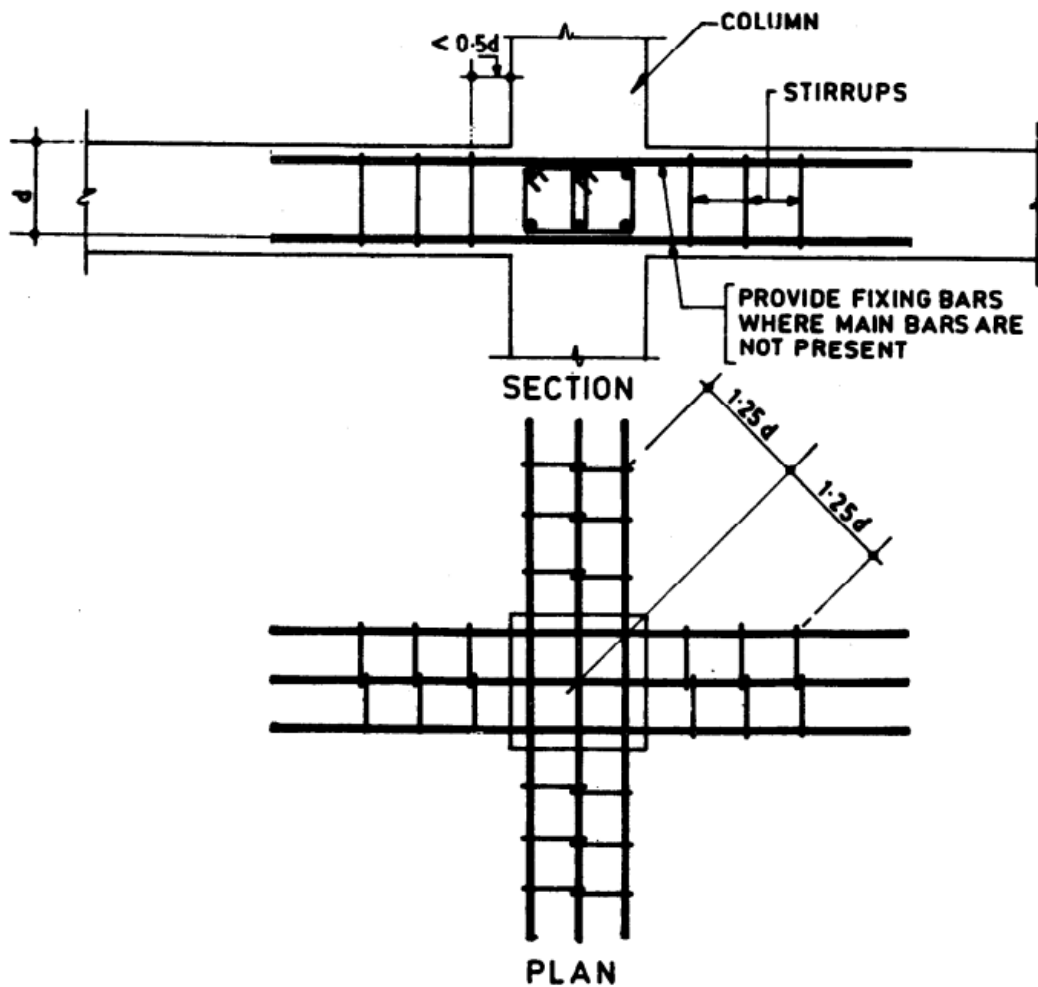
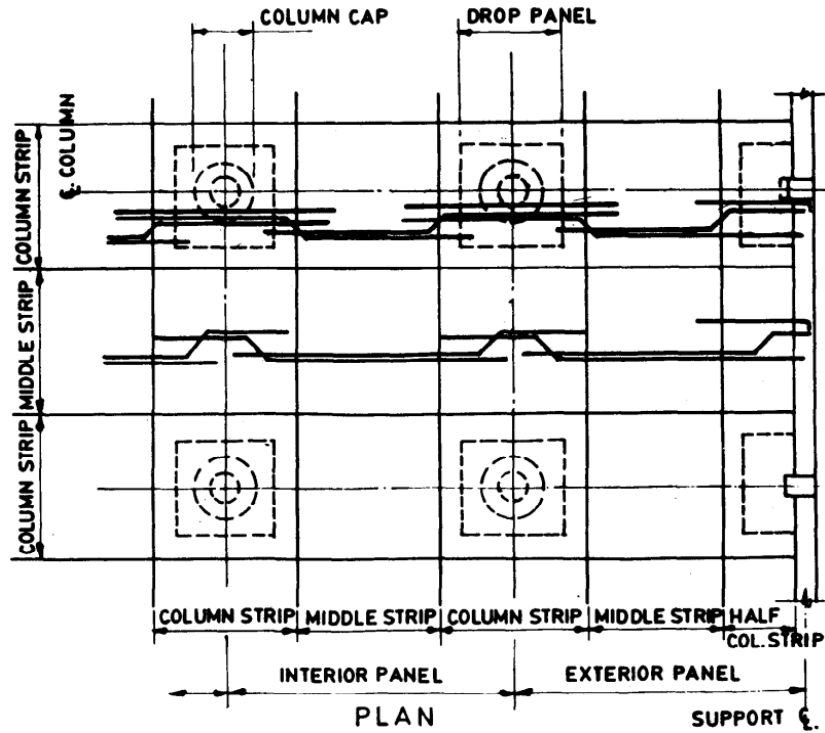


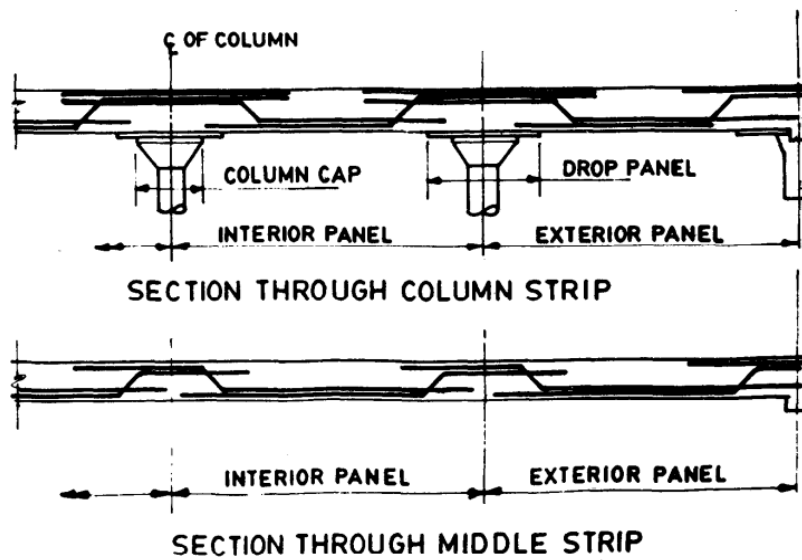
FIG. 78 DETAILING OF REINFORCEMENT IN A 4-WAY BEAM-COLUMN JOINT

(d) Column-to-Slab Joints

The.....(to be discussed)



(a)

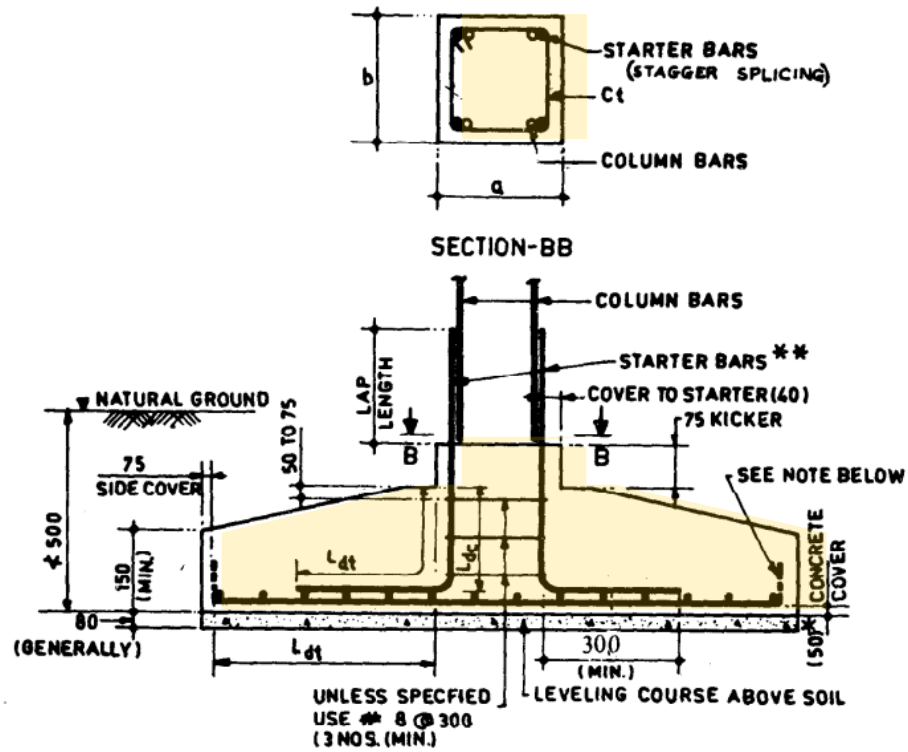


(b)

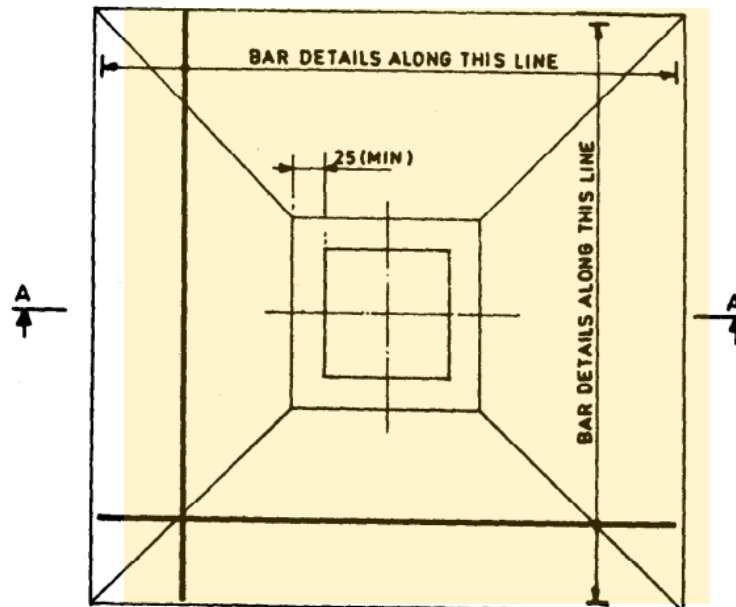
FIG. 79 DETAILING OF REINFORCEMENT IN A SLAB-COLUMN JOINT

(e) Column-to-Foundation Joints

The.....(to be discussed)

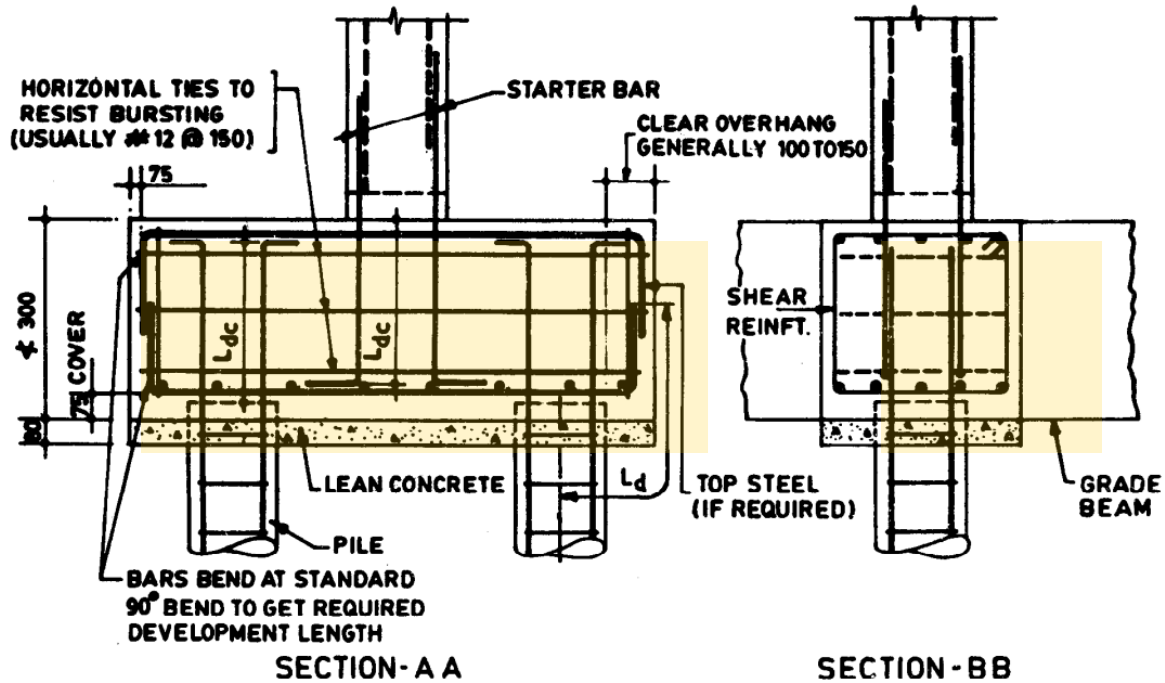


(a)

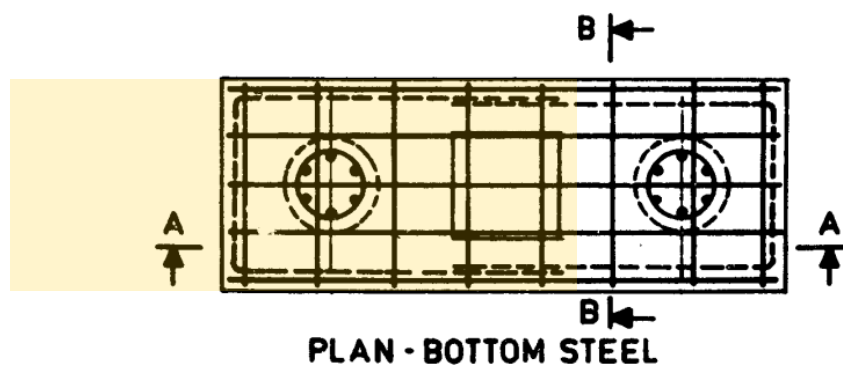


(b)

FIG. 80 DETAILING OF REINFORCEMENT AT A COLUMN TO FOOTING JUNCTION: (A) SECTIONAL ELEVATION, AND (B) PLAN



(a)



(b)

FIG. 81 DETAILING OF REINFORCEMENT AT A COLUMN TO PILE CAP JUNCTION: (A) SECTIONAL ELEVATION, AND (B) PLAN

(f) Construction Joints (Shear Keys)

The.....(to be discussed)

9.3.3.8 Anchors

The.....(to be discussed)

(a) Compression-Shear Anchors

The.....(to be discussed)

(b) Tension-Shear Anchors

The.....(to be discussed)

9.3.3.9 Thermal expansion

The stresses induced by temperature changes in concrete members that restrained against expansion and contraction of members shall be deemed to have been considered if the larger plan dimension of the structure is less than 45m. But, when the larger plan dimension is more than 45m, calculations shall be performed.

When it is desired to not provide such separation joints in a structure, the stresses induced in the structure shall be as estimated as per 8 under the action of loads applied as per the *basic load combinations* specified in **Table 4** corresponding to the *serviceability criteria*, and designed suitably as per 9.3.

9.3.4 Design and Detail Of Prestressed Concrete Members

The general design requirements of this standard for design of reinforced concrete structures shall be as applicable to the design of prestressed concrete members also, except as modified and supplemented hereunder for the design and detailing of prestressed concrete structural components and sub-systems.

a) Structural Design

The grades of materials (namely concrete, untensioned reinforcing steel and prestressing steel) admissible for use in concrete structures shall be as specified in 1.2. Further, in the calculations related to design of untensioned transverse reinforcement, the characteristic yield strength f_y shall not be taken to be more than 415 MPa, irrespective of the grade of untensioned transverse steel used. The admissible combinations of the grades of concrete and prestressing steel in reinforced concrete

members shall comply with those specified by Table 52. The said combinations shall be applicable for both longitudinal and transverse steels.

b) Reinforcement Detailing

The untensioned reinforcement steel shall be admissible combinations of the grades of concrete and longitudinal prestressing steel in prestressed concrete members shall comply with those specified by Table 52.

Table 52 Combinations of Grades of Concrete and Prestressing Steel admissible in reinforced concrete structures (to be discussed)

(Clause 9.3.4)

Grade of Concrete	Grade of Prestressing Steel						
	1200	1375	1470	1570	1715	1865	2010
Strands							
M35							
M40							
M45							
M50							
M55							
M60							
M65							
M70							
Wires							
	1200	1375	1470	1570	1715	1865	2010
M35							
M40							
M45							
M50							
M55							
M60							
M65							

M70							

9.3.4.1 *Effects of prestress*

The effects of prestress shall be accounted for under the action of loads appearing on the structure.

In particular, the contribution of prestressing tendons to the strength resistance of the member shall be limited to that arising out of the additional stress mobilized in them by their further elongation, consistent with the ultimate deformation of the member.

The force in prestress wires, cables and strands shall be considered (at least) at two levels, corresponding to:

- Prestressing force considered 10 percent more than the design value in post-tensioned systems, and 5 percent more than the design value in pre-tensioned systems, and
- Prestressing force considered 10 percent less than the design value in post-tensioned systems, and 5 percent less than the design value in pre-tensioned systems, and

Under the above levels of prestress, the strength and serviceability requirements laid down in this standard shall be met.

9.3.4.2 *Deductions for prestressing steel*

Area and moment of inertia of the cross-section shall be *modified* to account for the presence of prestressing tendons and the position of their centroid. The transformed area of prestress steel shall be taken as m times the area of the prestressing tendon, where m is the modular ratio given by:

$$m = \frac{E_p}{E_{ck}},$$

where

E_p = Modulus of Elasticity of prestressing steel specified by 7.3.1.2.1, and

E_{ck} = Modulus of Elasticity of concrete specified by 7.1.4.1.

In multi-directional prestressing, reduction may be made of the full area of the ungrouted ducts and sheaths placed at an angle to the prestressing tendons under consideration.

9.3.4.3 *Maximum initial prestress*

At the time of initial tensioning, the maximum prestress f_{pi} in a prestressing *rod, wire or strand* immediately behind the anchorage shall be taken as:

$$f_{pi} = \text{Min}[0.8f_{py}; 0.76f_{pu}] ,$$

where

f_{py} = 0.2 percent proof tensile stress of the prestressing steel, and

f_{pu} = Ultimate tensile stress of the prestressing steel.

The maximum prestressing force applied to tendon at active end shall not exceed:

- (1) $0.90f_{proof} A_p$ during stressing, when accuracy of measurement is ensured to be beyond ± 5 percent but within ± 10 percent, and $0.95f_{proof} A_p$ during stressing, when accuracy of measurement is ensured to be within ± 5 percent; and
- (2) $\text{Min}[0.75f_{pk} A_p; 0.85f_{proof} A_p]$ after stressing and after losses due to elastic shortening and anchorage slip,

where f_{proof} is proof stress of the prestressing steel at 0.1 percent strain.

9.3.4.4 Loss of Prestress

While assessing the stresses in concrete and steel during tensioning operations and later in service, due regard shall be paid to all losses and variations in stress arising from creep and shrinkage of concrete. Relaxation of steel, shortening (elastic deformation) of concrete at transfer, slip in anchorage, and friction. Unless otherwise determined by actual tests, allowance for these losses shall be made as specified hereunder.

9.3.4.4.1 Creep of concrete

The loss of prestress due to creep of concrete shall be determined under all permanently applied loads. The creep loss due to live load stresses, erection stresses and other stresses of short duration may be ignored.

The loss of prestress ΔF_C due to creep of concrete shall be obtained as:

$$\Delta F_{pC} = \begin{cases} E_p \varepsilon_{CC} A_C & \text{Bonded Tendons} \\ \Delta f_{CC} A_C & \text{Unbonded Tendons} \end{cases}$$

where

E_p = Modulus of elasticity of the prestressing steel;

ε_{CC} = Creep strain of concrete integrated along the line of centroid of the prestressing tendon at the section at which creep-loss is being calculated. For all practical purposes, it shall be the creep strain due to sustained stress equal to the average of the stresses at the beginning and end of the period in focus;

A_C = Area of cross-section of concrete member; and

Δf_{CC} = Integrated and averaged creep stress along the line joining the anchorage points.

9.3.4.4.2 Shrinkage of concrete

The loss of prestress ΔF_S due to shrinkage of concrete shall be obtained as:

$$\Delta F_S = E_p \varepsilon_{CS} A_C,$$

where

E_p = Modulus of elasticity of the prestressing steel; and

ε_{CS} = Shrinkage strain of concrete estimated as per 7.1.4.5, and

A_C = Area of cross-section of concrete member.

9.3.4.4.3 Relaxation of steel

The loss of prestress ΔF_R due to relaxation in prestressing steel shall be obtained as:

$$\Delta F_R = \Delta f_R A_p,$$

where

Δf_R = Relaxation losses (stress) in prestressing steel, which varies with type of steel, initial prestress, age and temperature; and

A_p = Area of cross-section of prestressing steel.

Δf_R shall be determined from experiments. When experimental values are not available, the relaxation losses shall be taken 3 times the values of $\Delta f_{R,1000}$ given in Table 53.

Table 53 Relaxation Losses $\Delta f_{R,1000}$ in Prestressing Steel at 1,000 h at (20°C ± 2°C)

(Clause 9.3.4.4.3)

Sl No.	Initial Stress	Relaxation Loss (in percent)	
		Normal Relaxation	Low Relaxation
(1)	(2)	(3)	(4)
i)	$0.5 f_p$	0.0	0.0
ii)	$0.6 f_p$	3.0	1.0
iii)	$0.7 f_p$	5.0	2.5
iv)	$0.8 f_p$	8.0	4.5

f_p is the characteristic strength of prestressing steel.

For tendons at higher temperatures or tendons subjected to large lateral loads, relaxation losses greater, the relaxation loss shall be taken as specified by the engineer-in-charge. No reduction in Δf_R shall be made in tendons with a load equal to or greater than the relevant jacking force applied for a short time prior to the anchoring of the Tendon.

9.3.4.4.4 Elastic shortening of concrete

The loss of prestress ΔF_S due to elastic shortening of concrete shall be obtained as:

$$\Delta F_E = 0.5mf_{Ci}A_C,$$

where

m = Modular ratio of the prestressing steel and concrete chosen; and

f_{Ci} = Initial stress in concrete, estimated assuming that tendons are located at their centroid:

- (a) *Pre-tensioning* – Initial stress in concrete adjacent to the tendons averaged along their lengths
- (b) *Post-tensioning* – Tendons are not stressed simultaneously, and there is a progressive loss of prestress during transfer due to gradual application of prestressing forces. Alternatively, the loss of prestress may be computed more accurately considering the sequence of tensioning;

and

A_C = Area of cross-section of concrete member.

9.3.4.4.5 Slip in Anchorage

The loss of prestress ΔF_S due to slip of wires during anchoring shall be obtained as:

$$\Delta F_A = \left(\frac{\Delta s}{L_p} \right) E_p A_p,$$

where

Δs = Slip in the anchorage of prestressing steel,

L_p = Length of the prestressing steel,

E_p = Modulus of elasticity of the prestressing steel, and

A_p = Area of cross-section of prestressing steel.

9.3.4.4.6 Friction

The loss of prestress $\Delta F_F(x)$ due to friction in wires away from the stressing end, at x metres from tensioning end and acting in the direction of the tangent to the curve of the cable, shall be obtained as:

$$\Delta F_F(x) = P_0 \left(1 - e^{\mu\alpha + kx} \right),$$

where

P_0 = Prestressing force in prestressed steel at the tensioning end acting in the direction of the tangent to the force of the cable,

μ = Coefficient of friction in curve, as given in Table 54,

α = Cumulative angle (rad) through which the tangent to the cable profile has turned between any two points under consideration, and

k = Coefficient for wave effect as given in Table 54.

Table 54 Values of μ and k for estimating loss of prestress due to friction

[Clause 9.3.4.4.6]

Type of High Tensile Steel	Type of Duct or Sheath	Values Recommended to be used in Design	
		k (/m)	μ
Wire cables	Bright metals	0.0091	0.25
	Galvanized	0.0046	0.20
	Lead coated	0.0046	0.18
Uncoated stress-relieved strand	Bright metals	0.0046	0.25
	Galvanized	0.0030	0.20
	Lead coated	0.0030	0.18
	Corrugated HDPE	0.0020	0.17

9.3.4.5 Transmission length

The following shall be considered when estimating the transmission length:

- The strength of concrete at transfer, the size and type of tendon, and the surface deformations of the tendon. The transmission length of the tendon towards the top of a unit is usually greater than that at the bottom.

- b) The transmission length shall be taken as:

Plain and indented wire 100ϕ ,

Crimped wires 65ϕ , and

Strands 30ϕ ,

if concrete is well-compacted, detailed as per the relevant provisions of this standard, and its strength at transfer at least 35 MPa, and the tendon is released gradually. In the above, ϕ is the diameter of the tendon.

- c) The development of stress in the tendon shall be assumed to vary parabolically along the length of the member.
- d) At the support of a simply supported beam, half the transmission length shall overhang from the support. And, at a fixed support, the entire transmission length shall overhang.
- e) The transmission length may vary depending on the site conditions. Therefore, it should be determined from field tests carried out under the most unfavourable conditions.

9.3.4.6 Anchorage zones

The local stresses generated in anchorage zones shall be accounted for.

9.3.4.6.1 Bearing Stress

The bearing stress $f_{bearing}$ in concrete adjoining the anchorage shall comply with:

$$f_{bearing} \leq \begin{cases} \text{Min} \left[0.8f_{ci}; 0.48f_{ck} \sqrt{\frac{A_{bearing}}{A_{punching}}} \right] & \text{After accounting for losses due to} \\ & \text{relaxation of steel, elastic shortening} \\ & \text{and seating of anchorages} \\ \text{Min} \left[f_{ci}; 0.60f_{ck} \sqrt{\frac{A_{bearing}}{A_{punching}}} \right] & \text{During tensioning} \end{cases},$$

where

f_{ci} = Cube strength at transfer of prestress,

$A_{punching}$ = Contact area of bearing of the plate in full contact at the anchorage end, which if circular in shape, shall be replaced by a square of equivalent area, and

$A_{bearing}$ = Maximum area of that portion of the prestressed member, which is geometrically similar and concentric to the punching area. Where anchorages are placed close to each other, it shall be the non-overlap area. Where stressing of cables is done in stages, the adjacent unstressed anchorages shall be neglected when determining the bearing area.

Further:

- When adequate hoop reinforcement is provided around the anchorages in the anchorage zone as recommended by the manufacturer, the values limiting the bearing stress may be increased suitably.
- When the anchorages are embedded in concrete (with reinforcement details, concrete strength, cover and other dimensions conforming to manufacturer's specifications), the bearing stress shall be investigated after accounting for the surface friction between the anchorage and the concrete.
- Where compressive stress already exists in the bearing area (such as, when anchorage zone is placed in the body of a structure), $f_{bearing}$ shall not exceed the values limiting the bearing stress specified above.

9.3.4.6.2 Bursting tensile force

The load F_0 in the tendon shall be estimated considering the tendon jacking load, in bonded post-tensioned members and the tendon jacking load or the load in the tendon at the strength criteria, whichever is greater, in unbonded post-tensioned members.

The bursting tensile force $F_{bursting}$ in an individual square end block loaded by a symmetrically placed square anchorage or bearing plate, shall be estimated as:

$$\frac{F_{bursting}}{F_0} = 0.32 - 0.3 \left(\frac{y_{p0}}{y_0} \right),$$

where

y_{p0} = Side of loaded area, and

y_0 = Side of end block.

Further:

- $F_{bursting}$ shall be distributed over the length $0.1y_0 - y_0$, from the loaded face of the end block to the other end of the tendon. Area of untensioned steel reinforcement required to resist $F_{bursting}$ shall be estimated considering smaller of:

- 1) Design strength stress of reinforcing bars of $0.87 f_y$, and
 - 2) Stress in reinforcing steel corresponding to a strain of 0.001, when the concrete cover to the reinforcement is less than 50 mm.
- b) In rectangular end blocks, the bursting tensile forces in the two principal directions should be assessed as above. When circular anchorage or bearing plates are used, the side of the equivalent square area shall be used. Where groups of anchorages or bearing plates occur, the end blocks shall be divided into a series of symmetrically loaded prisms and each prism treated in the above manner. For designing end blocks having a cross-section different in shape from that of the general cross-section of the beam, specialist literature shall be referred to when designing the same.
- c) The design for resisting the bursting tensile force does not account for the tensile strength of concrete. If the same needs to be accounted for, specialist literature shall be referred to consider the same.
- d) Design of anchorage zone shall account for the *Spalling Tensile Stresses* that occur in end blocks, where the anchorage or bearing plate is highly eccentric, and are maximum at the loaded face.

9.3.4.7 Detail of untensioned reinforcement

The detail of reinforcement in prestressed concrete shall generally conform to the requirements given for reinforced concrete members in this standard. In addition, the requirements specified hereunder shall be satisfied.

a) Longitudinal Reinforcement

In PSC beams,

- 1) When the depth D_w of the web exceeds 500 mm, longitudinal distribution reinforcement not less than 0.05 percent of the area of the web shall be provided on each face of the web. The spacing of the individual bars of such vertical reinforcement in the web shall not exceed 200 mm.
- 2) All un-tensioned longitudinal reinforcement in shall be restrained in the lateral direction, by placing them inside the closed-loop transverse reinforcement.

b) Transverse Reinforcement

Reinforcement in the form of closed-link or helix shall be provided perpendicular to the line of heavy compression or shock loading to resist the induced tensile stresses.

9.3.4.8 *Additional requirements*

The following additional requirements shall be complied with when designing prestressed concrete members.

a) *Continuity*

In the design of continuous prestressed concrete structures, due consideration shall be given to the effects of support restraints on both the external moment and the moment due to prestressing.

b) *Butted Assembly*

Where a butted assembly is used, or where conditions of abuttal are employed, proper provision shall be made to transfer all shear stresses. Wherever the shear stresses exceed the limits specified in _____, this provision shall include keying of all abutting faces.

9.3.4.9 *Prestressed members*

- a) When prestressed concrete members are subjected to loads reversals, it shall be demonstrated that the reversal of loads does not jeopardize the safety of the members and the associated structures.
- b) When beams are prestressed, which are parts of moment resisting frames:
 - 1) Their widths shall not be more than that of the columns that support them; and
 - 2) The requirement of strong column – weak beam specified in **13.2.2.4(a)** of IS 13920 (Part 2) shall be examined with the larger of the two beam moments at the face of the column during reversal.
 - 3) The effects of vertical acceleration specified in **7.5.2** of IS 1893 (Part 2) shall be examined in all earthquake zones.
- c) The effects of elastic shortening arising out of prestressing (either pre-tensioning or post-tensioning), if any, of members shall be accounted for in the structural analysis.
- d) When slabs are prestressed in flat slab systems, the effect of prestressing shall be examined on the punching shear in the flat slabs.

(a) Maximum Prestressing Force

The... (to be discussed)

(b) Anchorages and Couplers for Prestressing Tendons

The... (to be discussed)

(c) Reinforcement Detail

The... (to be discussed)

9.3.5 Design of Concrete Structures with Prestressed Members

When concrete structures are prestressed, the following shall apply:

- a) In buildings with prestressed slabs and beams, the lateral drift of the building shall meet the requirement given in 7.1.4 of IS 1893 (Part 2).
- b) Buildings with prestressing shall not be used in slabs and beams, the lateral drift of the building shall meet the requirement given in 7.1.4 of IS 1893 (Part 2).

9.3.6 Design and detail by strut and tie method

Local parts and junctions of members of concrete structures can be analysed and designed by the strut and tie method. When doing so, the provisions hereunder shall be complied with.

Structural concrete members or local regions in such members shall be:

- a) Modelled as an idealized truss consisting of struts and ties as per 8.2.1,
- b) Analysed as a truss subjected to the factored loads as per 8.2.3,
- c) Designed to resist the forces so obtained as per 9.3.6.1, and
- d) Detailed with reinforcement bars to meet the intent of design as per 9.3.6.2.

9.3.6.1 Design

The design limiting strength F_L of a strut, a tie and a node shall comply with:

$$F_L \geq F_D,$$

where

F_D = Design Load on the strut, tie and node, respectively, obtained when loads are applied as per combinations specified in Table 3 for strength criteria.

a) *Struts*

The design compressive strength F_{LS} of a strut considering only concrete shall be taken as:

$$F_{LS} = A_{cS} f_{cS},$$

where

A_{cS} = Smallest cross-sectional area along the length of the Strut, and

f_{cS} = Effective compressive strength of concrete in the Strut

$$= \text{Min}[f_{cS}; f_{cN}],$$

in which

f_{cS} = Characteristic compressive strength of concrete in the **Strut**,

$$= \frac{\beta_1 (0.68 f_{ck})}{\gamma_M},$$

β_1 = Factor accounting for the geometry and level of confinement of concrete in the Strut

$$= \begin{cases} 1.00 & \text{Prismatic Strut} \\ 0.75 & \text{Fan - shaped Strut with transverse reinforcement confining concrete} \\ 0.60 & \text{Fan - shaped Strut without transverse reinforcement confining concrete ,} \\ 0.40 & \text{Struts that can go into tension} \\ 0.60 & \text{All other Struts} \end{cases}$$

f_{ck} = Characteristic compressive strength of concrete,

γ_M = Partial Safety Factor for concrete in compression, taken as 1.5,

f_{cN} = Characteristic compressive strength of concrete at the *Node*,

$$= \frac{0.68 \beta_2 f_{ck}}{\gamma_M}, \text{ and}$$

β_2 = Factor accounting for the level of confinement of concrete at the Node

$$= \begin{cases} 1.0 & \text{Node where only compression struts and bearing areas meet} \\ 0.8 & \text{Node where at least one tie meets} \\ 0.6 & \text{Node where two or more ties meet} \end{cases}.$$

The design compressive strength F_{LS} of a strut *considering concrete and longitudinal compression reinforcement steel* shall be taken as:

$$F_{LS} = A_{cS} f_{cS}(\epsilon_{c,lim}) + A_{sc} \left(\frac{f_{sc}(\epsilon_{c,lim})}{\gamma_M} \right),$$

where

A_{sc} = Cross-sectional area of compression steel along the full length of the Strut,

$f_{sc}(\epsilon_{c,lim})$ = Characteristic tensile stress of steel reinforcement bars, and

γ_M = Partial Safety Factor for steel, taken as 1.15.

b) Ties

The design tensile strength F_{LT} of a reinforced concrete tie shall be taken as:

$$F_{LT} = A_{st} \left(\frac{f_y}{\gamma_M} \right),$$

where

A_{st} = Cross-sectional area of tension steel along the length of the tie, and

f_y = Characteristic tensile stress of steel reinforcement bars, and

γ_M = Partial Safety Factor for steel, taken as 1.15.

The design tensile strength F_{LT} of a prestressed concrete tie shall be taken as:

$$F_{LT} = A_{st} \left(\frac{f_y}{\gamma_M} \right) + A_{sp} \left(f_p^* + \frac{f_{sp}^*}{\gamma_M} \right),$$

where

A_{st} = Cross-sectional area of reinforcing steel along the length of the Tie, and

A_{sp} = Cross-sectional area of prestressing steel along the length of the Tie, and

f_y = Characteristic tensile strength of reinforcing steel reinforcement bars, and

f_p^* = Stress in prestressing steel after losses,

f_{sp}^* = Design Strength of prestressing steel, in MPa,

$$= \begin{cases} 415 & \text{for Bonded Prestressing Steel} \\ 70 & \text{for Unbonded Prestressing Steel} \end{cases}, \text{ and}$$

γ_M = Partial Safety Factor for steel, taken as 1.15.

c) *Nodes*

The nodes shall be designed to resist the difference between the Tie force on one side of the node and the tie force on the other side. At nodes anchoring one tie, the tie force shall be developed at the point where the centroid of the reinforcement in a tie leaves the node and enters the span. and, at nodes anchoring two or more ties, the tie force in each direction shall be developed at the point where the centroid of the reinforcement in the tie leaves the node.

The design compression strength F_{LN} of a Node shall be taken as:

$$F_{LT} = f_{cN} A_N,$$

where

f_{sp}^* = Effective compressive strength of the concrete in the Node,

$$= \frac{0.68\beta_2 f_{ck}}{\gamma_M},$$

β_2 = Factor as defined in 9.3.6.1(a), and

A_N = Area of concrete at the Node

= Smaller of:

- (a) Areas on the face of the Node on which the individual design strut force F_{DS} and tie force F_{DT} act on the Node, taken perpendicular to their line of action; and
- (b) Area of a section through the node, taken perpendicular to the line of action of the resultant force F_{DN} on the section.

9.3.6.2 Detailing

The requirements given hereunder shall be complied with when detailing the struts, ties and nodes of a concrete structure.

a) *Struts*

The reinforcement required in a strut shall be placed along:

- (i) Two mutually orthogonal directions at any angle to the axis of the strut, or
- (ii) One direction at an angle α ($\geq 40^\circ$) to the axis of the strut.

The compression reinforcement in a Strut shall be anchored properly, parallel to the axis of the strut, located within the strut, and enclosed in ties or spirals, as per _____.

Unless a detailed analysis provides accurate values, the angle θ of inclination of potential with the axis of the member (with low level of inelasticity), may be estimated as:

$$\theta = \frac{\pi}{4} + \frac{1}{2} \tan^{-1} \left(\frac{P}{V} \right) - \frac{1}{2} \tan^{-1} \left(\frac{L}{D} \right).$$

b) Ties

The axis of the reinforcement provided in a tie shall coincide with the axis of the Tie considered in the strut and tie model.

The longitudinal reinforcement in a *Tie* shall be anchored by mechanical devices, standard hooks, or straight bar development as per **9.2.2.7**, and the transverse reinforcement as per **9.2.2.7**.

c) Nodes

The...(to be discussed).

9.4 Serviceability Criteria

Under the action of the combinations of service loads specified in **6.4**, of the effects (namely stress-resultants and deformations) shall be estimated by linear structural analysis as specified in **8.1.1** considering the reduced moments of inertia of the member cross-sections specified in **8.2.1.1(a)**. The maximum deformations namely deflections, rotations, width of cracks, vibrations (such as displacements, velocities and accelerations) and relative movement at joints shall be estimated and checked to be within the limits specified hereunder.

The serviceability criterion of the structure or part thereof shall be assessed from:

- a) *Material remaining within permissible stresses at critical sections of members:*

The stress demand induced at critical sections of members produced by the probable most unfavourable combination of loads on the structure (arrived at by

using the appropriate partial safety factors γ_L for loads specified by 6.4 in Table 5) shall not exceed the corresponding permissible stress in the material (arrived at by using appropriate factor of strength κ_M on material strengths) as specified in hereunder; and

b) *Structure remaining within permissible deformations at critical locations of the structure:*

The deformation demand induced at critical sections of the structure produced by the probable most unfavourable combination of loads on the structure (arrived at by using the appropriate partial safety factors γ_L for loads specified by 6.4 in Table 5) shall not exceed the corresponding permissible stress in the material (arrived at by using appropriate factor of strength κ_M on material strengths) as specified in hereunder.

Before the members, joints and connections are designed, a structure shall meet the provisions of stability given in 9.3.1.

In concrete structures, where torsional moment is required to maintain equilibrium, members shall be designed for torsion in accordance with the provisions given in 9.3.2.6. But, for such indeterminate concrete structures, where torsional moment can be eliminated by releasing redundant restraints, without compromising the safety of the member and thereby of the structure, no specific design for torsional moment is necessary, provided torsional stiffness is neglected in structural analysis when calculating the internal stress-resultants.

9.4.1 *Deformation Checks*

The deformations estimated as per requirements of structural analysis specified in 8 shall be limited to the values specified hereunder.

9.4.1.1 *Deflections*

The estimation and limits on the deflections in individual members and the structure as whole shall be considered as provided hereunder. When the maximum slenderness limits specified in 9.2.3.1 are complied with, the members shall be deemed to have satisfied the deflection limits specified hereunder; in such cases, no additional calculations are required as specified hereunder.

a) *Estimation of deflections*

Deflections in individual members and the structure as whole shall be estimated by structural analysis performed as per 8 considering the load combinations specified in 6.4 as per Table 4 for serviceability criterion.

1) *Beams and slabs*

The vertical deflections of beams and slabs shall be estimated as per 8.10.1.1.

2) *Columns, structural walls and buildings*

The lateral deflections of columns, walls and buildings shall be obtained from linear structural analysis of the building, considering cracked moments of inertia given in 8.1.1(a).

3) *Imposed load to be considered when estimating deflections due to creep*

When computing the deflections due to creep, only dead load and a part of the imposed load that is likely to be permanent shall be considered. No specific recommendation in this regard is specified in the provisions. In the absence of detailed estimates, the proportion (depending upon the type of the occupancy) of the characteristic imposed loads specified in Table 55 shall be taken to be permanent. Changes, if any, of occupancy or use, which are likely to be made later during the life of the structure, shall be considered.

The deflections owing to creep and shrinkage in beams and slabs shall be calculated as given in 9.3.4.4(a) and 7.1.4.5, respectively. But, in ordinary buildings, when the slenderness ratios (that is span-to-depth ratios specified in 9.2.3.1) are complied with in beams and slabs, the effects of creep may be ignored in design of these members.

Table 55 Imposed Load to be Considered on Floors in Estimation of Vertical Deflection due to Creep

[Clause 9.4.1.1(a) and Clause 9.4.1.1(a)(3)]

SI No.	Location of Floors	Fraction of imposed load to be considered
(1)	(2)	(3)
i)	Residential Buildings	0.7
ii)	Office Buildings	0.6

iii)	Hospital Buildings (wards and corridors)	0.5
iv)	Theatres and Library Buildings	0.8
v)	School Buildings	0.5
vi)	Assembly Structures	0.5
vii)	Storage and Industrial Structures	1.0

b) *Permissible deflections*

Deflections estimated in **9.4.1.1(a)** shall be less than the values specified hereunder.

1) *Beams and slabs*

Vertical deflections of a RC or PSC slab or beam, which is estimated by the method specified in **8.10.1.1** and measured from the as-cast level of the supports of floors, roofs and all other horizontal members, shall be limited as per the following:

- i) The vertical deflection due to all loads (including the effects of temperature, creep and shrinkage) shall not exceed (Effective Span/250).
- ii) The vertical deflection occurring after erection of partitions and application of finishes (including the effects of temperature, creep and shrinkage) shall not exceed the smaller of (Effective Span/350) and 20 mm.

When specific requirements place more stringent limits, the same shall be used.

If finishes are to be applied to PSC members, the total vertical deflection shall not exceed span/300, unless uniformity of camber between adjoining units (that confine the finishes on the other sides) can be ensured. The above vertical deflection limits shall be deemed to be satisfied, if the effective span-to-depth (l/d) ratios are not greater than the values specified in **9.2.3.1**.

The said permissible deflections are applicable to solid slabs as well as to ribbed, hollow block and voided slabs.

2) *Columns, structural walls and buildings*

Lateral sway at the top of a building and in individual storeys, which are estimated from the above analyses, shall not exceed $H/500$ in load combinations involving design wind load; and $H/250$ or the value specified in the latest revision of IS 1893, in load combinations involving design earthquake load where H is the total height of the building measured from its base. Here, the base of the building on flat ground shall be taken as the top of the foundation, and of a building on sloped ground as the top of its uppermost foundation.

9.4.1.2 Rotations

The estimation and limits on the rotations in individual members and the structural as whole shall be considered as provided hereunder.

a) Estimation of rotations

Rotations in individual members and the structural as whole shall be estimated by structural analysis performed as per 8 considering the combinations of loads specified in 6.4 as per Table 4 for serviceability criterion.

b) Permissible rotations

Elastic rotation at the ends of the beams, slabs, walls and columns, and measured from the as-cast level of the members, shall be limited to the values specified in Table 56. These rotation limits shall be deemed to be satisfied, if the effective span-to-depth (l/d) ratios are not greater than the values specified in 9.2.3.1.

The rotations in the foundations shall not exceed the values given in Table 56.

Table 56 Maximum Rotation permitted at the end of the Member to meet the requirements of Serviceability Criterion Due To Rotation
[Clause 9.4.1.2(b)]

Sl No.	Length L of Member or Building	Maximum Rotation
(1)	(2)	(3)
i)	$100 \text{ m} < L$	$1/2000$
ii)	$50 \text{ m} < L < 100 \text{ m}$	$1/1000$
iii)	$25 \text{ m} < L < 50 \text{ m}$	$1/750$
iv)	$10 \text{ m} < L < 25 \text{ m}$	$1/500$

v)	$L < 10 \text{ m}$	$1/400$

Table 57 Permissible Values of Rotation in Foundation Members

<i>Foundation Type</i>	<i>Maximum Rotations (rad)</i>
Footing - <i>Individual</i>	??
Footing - <i>Combined</i>	??
Raft Foundation	??
Pile Foundation	??
Well Foundation	??
Other Foundation	??

Comment:
the above
be discussed
meeting.
members are
suggest values based on their expertise.

The values for
table needs to
during the
However,
encouraged to

9.4.1.3 Crack widths

The estimation and limits on the width of surface crack in individual members shall be considered as provided hereunder.

a) Estimation of Crack widths

Crack widths shall be estimated considering the combinations of loads specified in 6.4 as per **Table 4** for serviceability criterion. In members subjected to:

- 1) Bending moment demand M_{WD} , it shall be estimated as per as per 9.3.2.7.7, and
- 2) Axial load demand P_{SD} and bending moment demand M_{SD} , it need not be checked if $P_{SD} > 0.2f_{ck}A_g$,

where f_{ck} is the characteristic compressive strength of concrete and A_g the area of the gross section of the member, and shall be estimated as per 9.3.2.7.7, if:

$$P_{SD} \leq 0.2f_{ck}A_g.$$

b) Permissible crack widths

Two types of design are permissible in concrete members, namely no crack design and cracked design.

1) *No crack design*

The bending moment demand at the cross-section is less than the cracking bending moment M_{cr} estimated using section modulus of the gross transformed section and modulus of rupture f_{cr} of concrete specified in 7.1.4.6.4.

2) *Cracked design*

For reinforced concrete members, crack width (estimated as per 9.4.1.3) in beams, slabs, walls, columns, footings and rafts shall be limited to the values specified in Table 58. When the values of maximum spacing of longitudinal bars specified in Table 56 are complied with, the member shall be deemed to have satisfied the crack width requirement. For prestressed concrete members, crack width (estimated as per 9.4.1.3) in beams, slabs, walls, columns, footings and rafts shall be limited to the values specified in Table 59. The maximum permissible crack width in concrete members shall be taken as per Table 60.

Table 58 Allowable Crack Width in Reinforced Concrete Members to Serviceability Sub-Criterion on Cracking, Based on a Reference Cover Value of 25 mm
[Clause 9.4.1.3(b)]

Sl No.	Structural member	Maximum width (in mm) of crack permitted in Concrete Members
(1)	(2)	(3)
i)	Slabs	0.10
ii)	Beams	0.10
iii)	Columns or Piers	0.05
iv)	Walls	
	(a) Liquid Retaining Walls	0.00
	(b) Others	0.05
v)	Footings	0.05
vi)	Rafts	0.05
vii)	Piles	0.05
viii)	Structure retaining special gases or liquid	?? (to be discussed)

Table 59 Allowable crack width in Prestressed Concrete Members to meet Serviceability Sub-Criterion on Cracking
[Clause 9.3.4(a) and (b)]

Sl No.	Member Type	Condition	Maximum Permissible Width (mm) of Crack
(1)	(2)	(3)	(4)
i)	1	No tensile stress is allowed.	0
ii)	2	Tensile stresses are allowed, but not visible cracking.	0.10
iii)	3	Cracking is allowed, but should not affect the appearance or durability of the structure.	0.10

Table 60 Maximum permissible crack width (mm) in concrete members (to be discussed)
[Clause 9.3.4(b)]

		Bar Diameter (mm)													
		8	10	12	14	16	18	20	22	25	28	32	26	40	45
Bar Spacing (mm)	100														
	120														
	140														
	160														
	180														
	200														
	220														
	240														
	260														
	280														
	300														

9.4.1.4 Vibrations

When the structure has machines, long spans and flat slabs, and where wind vibrations are significant, the provisions specified hereunder shall apply.

a) Estimation of vibrations

The vibrations shall be estimated by dynamic structural analysis performed as per 8 considering the design loads combined as per the load combinations corresponding to serviceability criteria specified in Table 4.

b) Permissible vibrations

Peak displacement, peak velocity and peak acceleration values at any location in the structure shall be within the allowable values specified in Table 61.

Table 61 Allowable deformations at different locations in a structure to meet Serviceability Sub-Criterion on Vibration
[Clause 9.4.1.4(b)]

Sl No.	Vibration Quantity	Peak Vibration
(1)	(2)	(3)
i)	Displacement	10 mm
ii)	Velocity	
	(a) Commercial Buildings and Industrial Buildings	50 mm/s
	(b) Residential Buildings	20 mm/s
	(c) Special Buildings	10 mm/s
iii)	Acceleration	
	(a) Load Combinations including Wind Forces	
	(i) Commercial Buildings and Industrial Buildings	250 mm/s ²
	(ii) Residential Buildings	180 mm/s ²
	(b) Load Combinations <i>not</i> including Wind Forces	25 mm/s ²

9.4.1.5 Relative movement at joints

Relative movement between two parts of the same structure or two adjoining structures shall be accommodated by providing for separation joints as specified hereunder.

a) Estimation of relative movement

The relative movement shall be estimated by structural analysis performed as per 8 under the action of loads applied as per the basic load combinations specified in Table 4 corresponding to the serviceability criteria.

Structures in which marked changes in plan dimensions take place abruptly shall be provided with expansion on joints at the section where such changes occur. The

two parts of the structure shall be separated completely with no reinforcement (or any stiffening element) extending across the expansion joint, so that the necessary movement occurs with no or a minimum resistance at the joint, even though the foundation is common for two parts of the structure that is separated above ground. Also, the vertical structural members (such as columns and walls) adjacent to the joint shall be strengthened.

The length of a structure in which where expansion joints have to be provided shall be determined after considering various factors, for example temperature, exposure to weather, and time and season of the laying of the concrete. Normally structures exceeding 45m in length are designed with one nor more expansion joints. But, in view of the large number of factors involved in deciding the location, spacing and nature of expansion joints, the provision of expansion joint in RC structures shall be left to the discretion of the designer. IS 3414 gives design requirements that need to be provided for.

b) *Permissible relative movement*

1) *Sliding joints*

The relative displacement at the interface of members within a structure that slide on each other shall be permitted with as little friction at the interface as possible.

2) *Expansion and contraction joints*

The deformations between two parts of the same building or two buildings built adjacent to each other, which are estimated under the action of service load combinations involving Temperature Loads, shall be provided for in the form of gaps between the said two parts or buildings. In particular, the non-structural elements passing from one part or building to the other shall be designed to accommodate these deformations, without causing disruption in the services provided by the non-structural elements.

Gaps between two adjoining parts of a structure or two adjoining structures shall be the maximum of:

- i) As specified in Table 53.

- ii) $H/250$ under the action of wind loads, and
- iii) From detailed structural analysis performed to obtain the lateral displacement of the structure subjected to wind and earthquake load combinations.

9.4.1.6 Settlements of foundations

The uniform and relative vertical settlements in foundations shall be within the allowable settlements specified in Table 62.

The tilting of a structure shall be limited to $H/2000$ arising from the relative vertical settlements across the width of the foundation under the action of loads corresponding to basic load combinations specified in Table 3.

Table 62 Allowable Deflections in Foundations

[Clause 9.4.1.6]

Sl No.	Foundation Type	Maximum limits of Vertical Deflection mm	
		Member supporting normal structures	Member supporting or overlying structures sensitive to deflections
(1)	(2)	(3)	(4)
i)	Footings		
	(a) Individual Footings	40	25
	(b) Combined Footings	40	25
ii)	Raft Foundations	40	25
iii)	Piled-Raft Foundation	40	25
iv)	Well Foundation	40	25

9.4.2 Permissible Stresses

Permissible stresses in concrete, steel and their interface in concrete members and joints consistent with requirements of serviceability criterion shall be limited to the values given hereunder.

9.4.2.1 Concrete

In the assessment of adequacy of a concrete sections and members, lengths of lap splices and development lengths (including design lengths of anchorage) to meet the requirements of serviceability criterion, seven permissible strengths of concrete shall be considered as specified hereunder:

- a) Permissible bearing strength σ_{pbr} ,
- b) Permissible bond strength τ_{pbd} ,
- c) Permissible axial strength σ_{pac} in axial compression, and permissible axial strength σ_{pat} in axial tension,
- d) Permissible shear stress strength τ_{pc} , and permissible maximum shear strength $\tau_{pc,max}$, and permissible punching shear strength τ_{pcp} , and
- e) Permissible flexural strength σ_{pbc} in bending compression and permissible flexural strength σ_{pbt} in bending tension.

Designers may use values lower than those specified hereunder.

The factor of strength κ_M used in arriving at the above permissible strengths of concrete are given in Table 63.

Table 63 Factor of Strength κ_M for Concrete

[Clause 9.4.2.1]

Sl No.	Permissible Strength of Concrete	Factor of Strength κ_M
(1)	(2)	(3)
i)	Permissible bearing strength σ_{pbr}	2.25
ii)	Permissible bond strength τ_{pbd}	2.70
iii)	Permissible axial strength	
	(a) σ_{pac} in axial compression	2.70
	(b) σ_{pat} in axial tension	3.00

iv)	Permissible shear strength	
	(a) τ_{pc} in flexural shear	2.70
	(b) $\tau_{pc,max}$ in maximum flexural shear	2.25
	(c) τ_{pcp} in punching shear	2.70
v)	Permissible flexural strength	
	(a) σ_{pbc} in bending compression	2.25
	(b) σ_{pbt} in bending tension	3.00

a) *Permissible bearing strength σ_{pbr}*

The permissible bearing strength σ_{pbr} of concrete shall be taken as:

$$\sigma_{pbr} = \frac{0.68 f_{ck}}{\kappa_M} = 0.272 f_{ck},$$

where

f_{ck} = Characteristic compressive strength of concrete, and

κ_M = Factor of strength for concrete in bearing compression = 2.5 .

b) *Permissible bond strength τ_{pbd}*

The permissible bond strength τ_{pbd} for reinforcing bars embedded in concrete shall be taken as:

$$\tau_{pbd} = \begin{cases} \frac{0.300(f_{ck})^{0.6}}{\kappa_M} = 0.090(f_{ck})^{0.6} & \text{For Fe250 bars in Tension} \\ \frac{0.480(f_{ck})^{0.6}}{\kappa_M} = 0.144(f_{ck})^{0.6} & \text{For Fe415, Fe500 and Fe5500 bars in Tension} \\ \frac{0.375(f_{ck})^{0.6}}{\kappa_M} = 0.113(f_{ck})^{0.6} & \text{For Fe250 bars in Compression} \\ \frac{0.600(f_{ck})^{0.6}}{\kappa_M} = 0.180(f_{ck})^{0.6} & \text{For Fe415, Fe500 and Fe5500 bars in Compression} \end{cases}$$

where

f_{ck} = Characteristic compressive strength of concrete, and

κ_M = Factor of strength for concrete in *bond* = 3.0.

In case of earthquake loading, the values given in Table 61 for bars in tension shall be used for bars on both faces.

c) *Permissible axial strengths σ_{Pac} and σ_{Pat}*

The permissible axial strength σ_{Pac} of concrete in axial compression shall be taken as:

$$\sigma_{Pac} = \frac{0.68f_{ck}}{\kappa_M} = 0.25f_{ck},$$

where

f_{ck} = Characteristic compressive strength of concrete, and

κ_M = Factor of strength for concrete in compression = 2.7.

The permissible axial strength σ_{Pat} of concrete in axial tension shall be taken as:

$$\sigma_{Pat} = \frac{0.30f_{ck}}{\kappa_M} = 0.10f_{ck},$$

where

f_{ck} = Characteristic Compressive Strength of concrete, and

κ_M = Factor of Strength for concrete in tension = 3.0.

This shall be used for estimating the bending moment capacity and the axial force capacity in tension of a concrete section or member.

d) *Permissible shear strengths τ_{Pc} , $\tau_{Pc,max}$ and τ_{Pcp}*

The permissible shear strength τ_{Pc} of concrete in concrete members provided with steel transverse reinforcement shall be taken as:

$$\tau_{Pc} = \left(\frac{0.36 \lambda_a \lambda_s \lambda_g \sqrt[3]{(p_t f_{ck})}}{\kappa_M} \right) \delta = (0.16 \lambda_a \lambda_s \lambda_g \sqrt[3]{(p_t f_{ck})}) \delta, \text{ and}$$

where

$$\lambda_a = \text{aggregate type factor} = \begin{cases} 1.00 & \text{Normal weight aggregate} \\ 0.75 & \text{Light weight aggregate} \end{cases},$$

$$\lambda_s = \text{cross-section Size factor} = \begin{cases} 1.00 & \text{Effectivedepthd} \leq 400\text{mm} \\ \left(\frac{400}{d} \right)^{0.25} & \text{Effectivedepthd} > 400\text{mm} \end{cases},$$

$$\lambda_g = \text{cross-section Geometry factor} = \begin{cases} 0.83 & \text{Rectangular or square cross - section} \\ 0.70 & \text{Circular cross - section} \end{cases},$$

κ_M = factor of Strength for concrete in Shear = 2.7,

f_{ck} = characteristic Compressive Strength of concrete (MPa), and

p_t = fraction A_{st}/bd of tension steel provided in the beam,

$$\delta = \begin{cases} \left(1 + \frac{3P_D}{A_g f_{ck}} \right) \leq 1.5 & \text{for compressiv e axial load} \\ \left(1 - \frac{12P_D}{A_g f_{ck}} \right) \geq 0 & \text{for tensile axial load} \end{cases},$$

in which

A_{st} = area of longitudinal tensile steel in the *beam*,

b = breadth of the beam parallel to the axis of bending,

d = effective depth of the beam perpendicular to the axis of bending,

P_D = design Axial Load on member obtained from factored loads applied on the structure,

A_g = gross area of the concrete column, and

f_{ck} = characteristic compressive stress capacity of concrete.

e) *Permissible flexural strengths* σ_{pbc} and σ_{pbt}

The permissible flexural strength σ_{pbc} of concrete in compression shall be taken as:

$$\sigma_{pbc} = \frac{0.68 f_{ck}}{\kappa_M} = \frac{1}{3} f_{ck},$$

where

f_{ck} = Characteristic compressive strength of concrete, and

κ_M = Partial safety factor for concrete in compression = 2.25.

The permissible flexural strength σ_{pbt} of concrete in tension shall be taken as:

$$\sigma_{pbt} = \frac{1.5\sqrt{f_{ck}}}{\kappa_M} = 0.5\sqrt{f_{ck}},$$

where

f_{ck} = Characteristic compressive strength of concrete, and

κ_M = Factor of strength for concrete in tension = 3.0.

This value shall be used, when estimating the bending moment capacity of a concrete section or member.

9.4.2.2 Steel reinforcement

At critical sections of members, the permissible stresses in steel reinforcement bars shall be limited to the values specified in Table 64 for the load actions mentioned therein.

In flexural members, the value of permissible stress σ_{st} in tension in Table 64 is applicable at the centroid of the tensile reinforcement, subject to the condition that when more than one layer of tensile reinforcement is provided, the stress at the centroid of the outermost layer shall not exceed by more than 10 percent of this value.

Table 64 Minimum Gap Required to Be Provided to Meet the Requirements of Serviceability Criterion due to Relative Movements

[Clause 9.4.2.2]

Sl No.	Load Action	Permissible Stress (MPa)		
		Mild Steel of Grade I conforming to IS 432 (Part 1)	Medium Tensile Steel conforming to IS 432 (Part 1)	High Yield Strength Deformed Steel, Thermo-Mechanically Treated Steel and others steels conforming to IS 1786
(1)	(2)	(3)	(4)	(5)
i)	σ_{st} in Tension			

	$\phi \leq 20mm$	140	$0.5f_y \leq 190MPa$	$0.55f_y$
	$\phi > 20mm$	130		$0.55f_y$
	Welded wire fabric conforming to IS 1566			230
ii)	σ_{sc} in Compression in Columns	130	130	$0.45f_y$
iii)	σ_{sc} in Compression in Beams and Slabs			
	- When compressive resistance of concrete is considered	$1.5m\sigma_{csc} \leq 130MPa$	$1.5m\sigma_{csc} \leq 130MPa$	$1.5m\sigma_{csc} \leq 0.45f_y$
	- When compressive resistance of concrete is <i>not</i> considered			
	$\phi \leq 20mm$	140	$0.5f_y \leq 190MPa$	$0.55f_y$
	$\phi > 20mm$	130	$0.5f_y \leq 190MPa$	$0.55f_y$
iv)	τ_{sv} in Shear	130	130	230
NOTE: When mild steel of grade II conforming to IS 432 (Part 1) is used: 1. In new designs, Permissible Stresses shall be 90% of the values given above for Mild Steel of Grade I conforming to IS 432 (Part 1); and 2. In designs that are completed on the basis of Mild Steel of Grade I conforming to IS 432 (Part 1), the area of reinforcement shall be increased by 10% of that required for Mild Steel of Grade I conforming to IS 432 (Part 1). 3. The Modular Ratio m shall be taken as specified in 9.3.4.2.				

9.4.2.3 Increase in permissible stresses

No increase shall be permitted in permissible stresses when stresses due to wind load and earthquake load are combined individually as per Table 4 with those due to dead load, imposed load and impact load.

9.4.3 Stress Checks and Stress-Resultant Checks

Under the action of loads applied in combinations specified for serviceability criterion as per Table 4, the cross-section of a concrete member can be designed to be either uncracked or cracked. This depends on the philosophy of design specified for different concrete

structures in the respective standard; in the absence any guidance in any such standard, Table 65 shall be binding.

The structure shall be deemed to have met the strength-related serviceability requirements, if:

- a) *Stress Checks* — The stress induced at the extreme fibers at the critical sections shall not exceed the permissible values specified in 9.4.2, and
- b) *Stress-Resultant Checks* — The serviceability design stress-resultant demands generated at the critical sections of members under the action of loads applied as per combinations specified in Table 4 for serviceability criterion, shall not exceed the corresponding serviceability design capacities specified hereunder.

In doing so, when cracked section is considered, the following may be considered:

- (1) General design requirements specified in 9.1, and 9.2.1 to 9.2.5, and
- (2) Redistribution of Moments specified in 8.9.

Table 65 Philosophy of Serviceability Check with respect to Permitting Cracks
[Clause 9.4.3]

Sl No.	Type of structure	Uncracked Section	Cracked Section
(1)	(2)	(3)	(4)
i)	Reinforced concrete structures	Crack width check not required	Crack width check required as per durability requirements given in 9.5.4.4.6.
ii)	Prestressed concrete structures		
	(a) Type 1 : Uncracked and no tension	Crack width check not required	Not permitted
	(b) Type 2 : Uncracked, but tension is within <i>Modulus of Rupture</i>		
	(c) Type 3 : Cracked		Crack width check required under the action of service

			loads given in Table 81
iii)	Special structures		
	(a) Liquid retaining structures	Crack width check required as per respective Standard	
	(b) Gas retaining structures		
	(c) Radiation containing structures		
	(d) Structures with special performance requirement		

9.4.3.1 Assumptions

The following assumptions shall be made in the estimation of serviceability design stress-resultant capacities of RC members:

- At any cross-section, plane sections plane and normal to the axis of bending before bending, remain plane and normal to the axis of bending even after bending.
- All tensile stresses are resisted by reinforcing bars and none by concrete, except as otherwise specifically permitted.
- The stress-strain curve is linear of both steel and concrete under serviceability level loads.
- The modular ratio m of reinforcement bars in tension shall be taken as $m = 280 / f_{ck}$, and of those in compression as $1.5m = 420 / f_{ck}$, where f_{ck} is taken in MPa.

9.4.3.2 Axial force capacity P_s at serviceability criterion

The axial force capacity P_s at serviceability criterion of RC members provided hereunder shall be applicable under pure axial load condition.

9.4.3.2.1 Member in tension

The design tensile axial force demand P_{SDT} on the member under the action of external serviceability loads shall be less than the serviceability design tensile axial force capacity P_{ST} permissible in a member in tension estimated as:

$$P_{PT} = (A_c + mA_{st})\sigma_{cat},$$

where

A_c = area of concrete in the cross-section (excluding any finishing material),

A_{st} = area of steel in the cross-section,

m = modular ratio for the grades chosen of steel and concrete,

$$= \frac{280}{f_{ck}}, \text{ and}$$

σ_{cat} = permissible stress in concrete under axial tension specified in 9.4.2.1.

9.4.3.2.2 Pedestals and short columns in compression

- a) The design compression axial force demand P_{SDC} on a pedestal or short column [complying with 9.2.3.1(c)] in compression under the action of external serviceability loads shall be less than the serviceability compressive axial force capacity P_{SC} estimated as:

$$P_{SC} = (A_c \sigma_{cac} + A_{sc} \sigma_{sc}) \lambda,$$

where

- A_c = area of concrete in the cross-section (excluding any finishing material),
 A_{st} = area of steel in the cross-section,
 σ_{cac} = permissible stress in concrete under axial compression specified in 9.4.2.1,
 σ_{sc} = permissible stress in steel bars in axial compression specified in 9.4.2.2, and
 λ = $\begin{cases} 1.00 & \text{For lateral ties consisting of closed loops} \\ 1.05 & \text{For lateral ties consisting of helical spirals} \end{cases}$

- b) For pedestals and short columns:

- 1) The above estimate of P_{SC} shall not be applicable when the lateral ties in members do not have closed loops or helical spirals; and
- 2) The minimum eccentricity requirement in 9.3.2.2 shall be deemed to be complied with.

9.4.3.2.3 Long columns in compression

9.4.3.2.3.1 The design compression axial force demand P_{SDC} on a long column (complying with 9.2.3.1(c)) in compression under the action of external serviceability loads shall be less than the serviceability compressive axial force capacity P_{SC} estimated as:

$$P_{SC} = (A_c \sigma_{cac} + A_{sc} \sigma_{sc}) C_r,$$

where

- A_c = area of concrete in the cross-section (excluding any finishing material),
 A_{st} = area of steel in the cross-section,

$$\begin{aligned}
\sigma_{cac} &= \text{permissible stress in concrete under axial compression specified in 9.4.2.1,} \\
\sigma_{sc} &= \text{permissible stress in steel bars in axial compression specified in 9.4.2.2, and} \\
C_r &= \text{slenderness Factor} \\
&= \text{Min} \left[\left(1.25 - \frac{l_e}{48b} \right); \left(1.25 - \frac{l_e}{160r_{\min}} \right) \right],
\end{aligned}$$

wherein

$$\begin{aligned}
b &= \text{least lateral dimension of column; for columns with helical spirals, it shall} \\
&\quad \text{be taken as the diameter of core concrete,} \\
l_e &= \text{effective length of column, and} \\
r_{\min} &= \text{least radius of gyration of the cross-section.}
\end{aligned}$$

In the above, l_e/b and $l_e/r_{\min}$ shall be more than 12 and 40, respectively.

9.4.3.2.3.2 For composite columns, refer to IS 11384.

9.4.3.3 Serviceability design bending moment capacity M_s

The design capacities of RC members provided hereunder shall be applicable under pure bending moment without axial load.

9.4.3.3.1 Uncracked section

The design bending moment demands M_{SDX} and M_{SDY} on the member under the action of external serviceability loads shall be less than the serviceability design bending moment capacities M_{SX} and M_{SY} permissible in a member for bending about X- and Y-axes, respectively, estimated as:

$$\begin{aligned}
M_{SX} &= \text{Min} [Z_{tX} \sigma_{cbt}; Z_{cX} \sigma_{cbc}], \\
M_{SY} &= \text{Min} [Z_{tY} \sigma_{cbt}; Z_{cY} \sigma_{cbc}],
\end{aligned}$$

where

Z_{tX} = Section Modulus of the gross cross-section corresponding to the bottom-most fiber of concrete in tension (considering the transformed section) for bending about X-axis,

Z_{tY} = Section Modulus of the gross cross-section corresponding to the bottom-most fiber of concrete in tension (considering the transformed section) for bending about Y-axis,

Z_{cX} = Section Modulus of the gross cross-section corresponding to the bottom-most fiber of concrete in compression (considering the transformed section) for bending about X-axis,

Z_{cY} = Section Modulus of the gross cross-section corresponding to the bottom-most fiber of concrete in compression (considering the transformed section) for bending about Y-axis,

σ_{cbt} = Permissible stress in concrete under bending tension specified in **9.4.2.1**, and

σ_{cbc} = Permissible stress in concrete under bending compression specified in **9.4.2.1**.

In the above, the transformed area of the cross-section shall be estimated by considering transformed area of steel reinforcing bars in tension as mA_{st} and that of steel reinforcing bars in compression as $1.5mA_{sc}$.

Also, the absolute value of tensile stress f_{cbt} in concrete under bending tension shall satisfy the conditions:

$$f_{cbt} = \frac{M_{SDX}}{Z_{tX}} \leq 1.25 \left(\frac{M_{SDX}}{Z_{cX}} \right) \text{ for uniaxial bending about major X-axis, and} \\ \leq 0.75 f_{cr,7days}$$

$$f_{cbt} = \left(\frac{M_{SDX}}{Z_{tX}} + \frac{M_{SDY}}{Z_{tY}} \right) \leq 1.35 \left(\frac{M_{SDX}}{Z_{cX}} + \frac{M_{SDY}}{Z_{cY}} \right) \text{ for biaxial bending about X- \& Y-} \\ \leq 0.75 f_{cr,7days}$$

axes, where

$f_{cr,7days}$ = modulus of rupture of concrete measured on 7-days after casting, different from the 28-day value given in **7.1.4.6.4**.

9.4.3.3.2 Cracked section

9.4.3.3.2.1 If the requirements given in **9.4.3.4.2** are not satisfied, the stresses in concrete and steel shall be calculated by the theory of cracked RC section, in which the tensile resistance of concrete is ignored. Again, for the stresses so calculated shall be within the permissible stresses specified in **9.4.2** for the section to be deemed safe.

9.4.3.3.2.2 The maximum stresses in concrete and steel shall be found from first principles based on the cracked section theory satisfying the following requirements:

- a) The stresses and strains at a cross-section satisfy all of the following:
 - 1) The external axial load and bending moment demand applied on the section are in equilibrium with the resultants of the internal stresses;
 - 2) The stress is linearly proportional to strain at any point in both steel and concrete; and

- 3) The strains at a cross-section are compatible in keeping with the assumption of plane sections before bending remain plane and normal even after bending; and

b) Tensile strength of concrete is ignored.

9.4.3.4 Combined serviceability design axial force P_s – bending moment M_s capacity

The provisions hereunder shall be applicable for RC members subjected to combined axial load in compression and bending moment.

9.4.3.4.1 Uncracked section

The absolute value of tensile stress f_{cbt} in concrete under bending tension shall satisfy the conditions:

$$f_{cbt} = \frac{M_{SDX}}{Z_{tX}} \leq 1.25 \left(\frac{P_{SDC}}{A_T} + \frac{M_{SDX}}{Z_{cX}} \right) \text{ for uniaxial bending about major X-axis, and} \\ \leq 0.75 f_{cr,7 \text{ days}}$$

$$f_{cbt} = \left(\frac{P_{SD}}{A_T} + \frac{M_{SDX}}{Z_{tX}} + \frac{M_{SDY}}{Z_{tY}} \right) \leq 1.35 \left(\frac{P_{SDC}}{A_T} + \frac{M_{SDX}}{Z_{cX}} + \frac{M_{SDY}}{Z_{cY}} \right) \\ \leq 0.75 f_{cr,7 \text{ days}}$$

for biaxial bending about X- and Y-axes,

where

A_T = transformed area of the cross-section estimated by considering transformed area of steel reinforcing bars in tension as mA_{st} and that of steel reinforcing bars in compression as $1.5mA_{sc}$, and

$f_{cr,7 \text{ days}}$ = modulus of rupture of concrete measured on 7-days after casting, different from the 28-day value given in 7.1.4.6.4.

Also, the design serviceability axial load demand P_{SDC} (in compression) and the design bending moment demands M_{SDX} and M_{SDY} on the member under the action of external serviceability loads, and the design serviceability axial load capacity P_{SC} and the design serviceability bending moment capacities M_{SX} and M_{SY} of the member for bending about X- and Y-axes, respectively, shall satisfy the following condition:

$$\frac{P_{SDC}}{P_{SC}} + \frac{M_{SDX}}{M_{SX}} + \frac{M_{SDY}}{M_{SY}} \leq 1$$

where

$$P_{PC} = (A_c + 1.5mA_{ac})\sigma_{cac}$$

$$M_{PX} = Z_{cX}\sigma_{cbc}, \text{ and}$$

$$M_{PY} = Z_{cY}\sigma_{cbc}.$$

9.4.3.4.2 Cracked section

9.4.3.4.2.1 If the requirements given in **8.4.3.5(a)** are not satisfied, the stresses in concrete and steel shall be calculated by the theory of cracked RC section, in which the tensile resistance of concrete is ignored. Again, for the stresses so calculated shall be within the permissible stresses specified in **9.4.2** for the section to be deemed safe.

9.4.3.4.2.2 The maximum stresses in concrete and steel shall be found from first principles based on the cracked section theory satisfying the following requirements:

- a) The stresses and strains at a cross-section satisfy all of the following:
 - 1) The external axial load and bending moment demand applied on the section are in equilibrium with the resultants of the internal stresses;
 - 2) The stress is linearly proportional to strain at any point in both steel and concrete; and
 - 3) The strains at a cross-section are compatible in keeping with the assumption of plane sections before bending remain plane and normal even after bending; and
- b) Tensile strength of concrete is ignored.

9.4.3.4.2.3 Members subjected to combined axial load and bending moment shall be checked to comply with the requirements of limit state method of design as per **9.4.3.4**.

9.4.3.5 Serviceability design shear force capacity V_s

9.4.3.5.1 The serviceability design shear force demand V_{SD} on a RC member (generated at the critical sections of members under the action of loads applied as per combinations specified in Table 4 for serviceability criterion) shall not exceed the corresponding serviceability design shear force capacity V_s specified hereunder.

9.4.3.5.2 The serviceability design shear force capacity V_s of a RC member shall be estimated as:

$$V_s = V_{sc} + V_{ss},$$

where

$$\begin{aligned} V_{sc} &= \text{contribution of concrete to } V_L \\ &= \tau_c b d, \end{aligned}$$

$$\begin{aligned} V_{ss} &= \text{contribution of reinforcing steel to } V_L \\ &= \sigma_{yV} A_{SV} \left(\frac{d}{s_V} \right)_{INT}, \end{aligned}$$

in which

$$\begin{aligned} A_{SV} &= \text{total cross-sectional area of transverse steel reinforcement} \\ &= N_{SV} \left[\pi \left(\frac{\phi_{SV}}{2} \right)^2 \right], \end{aligned}$$

$$s_V = \text{spacing of the stirrups along the length of the member,}$$

$$f_{yV} = \text{permissible stress in steel under shear as specified in 9.4.2, shall not be taken more than } 230 \text{ MPa,}$$

$$N_{SV} = \text{number of legs in transverse reinforcement at each section of member,}$$

$$\phi_{SV} = \text{diameter of the bar used as transverse reinforcement, and}$$

$$\left(\frac{d}{s_V} \right)_{INT} = \text{integral value of the number.}$$

9.4.3.5.3 Shear reinforcement shall be provided in RC beams, columns and structural walls as per the following:

a) When $0 < \tau_{SD} \leq \tau_c$:

Minimum shear reinforcement shall be provided as specified in **9.2.4.1(b)**,

b) When $\tau_c < \tau_{SD} \leq \tau_{c,max}$:

Shear reinforcement shall be provided as specified hereunder, and

c) When $\tau_{c,max} < \tau_{SD}$:

The member shall be re-sized to increase its shear area, grade of concrete to be increased, and/or percentage tension reinforcement be increased.

In the above, τ_{SD} is the nominal design shear stress demand at a RC section, given by:

$$\tau_{SD} = \frac{V_{SD} \pm \frac{M_{SD} \tan \beta}{d}}{bd},$$

τ_C = design shear strength of concrete as specified in 9.4.2, and

$\tau_{C,max}$ = maximum design shear stress of concrete as specified in 9.4.2,

in which

b = breadth of member, taken as breadth of web for flanged sections,

d = effective depth,

V_{SD} = serviceability design shear force demand at the critical section for shear,

M_{SD} = serviceability design bending moment demand at the critical section for shear, and

β = angle of inclination of the compression surface of the beam in tapered beams, and taken as 0 in prismatic beams.

In the expression, the *negative* sign applies when the *Bending Moment* M_{SD} increases numerically in the same direction as the *effective depth* d increases, and the *positive* sign when it decreases.

9.4.3.6 Serviceability design torsional moment capacity T_s

9.4.3.6.1 A member in torsion is said to have met the requirements of serviceability, if:

a) *Stress Check*:

- 1) The nominal shear stresses in concrete at all critical sections are less than the shear stress τ_c permissible in concrete (specified in 9.4.2.1), and
- 2) The tensile stresses in reinforcing steel stirrup at those sections are less than the permissible tensile stress σ_{st} .

b) *Strength Check*:

The serviceability design torsional moment demand T_{SD} on a RC member (generated at the critical sections of members under the action of loads applied as

per combinations specified in Table 4 for serviceability criterion) shall not exceed the corresponding serviceability design torsional moment capacity T_s specified hereunder.

9.4.3.6.2 In general, where the torsional stiffness (and strength) of members has not been considered in the analysis of the structure, no specific calculations for torsion shall be necessary. Adequate control in torsional cracking shall be assumed to have been provided by the required nominal shear reinforcement. But, where the torsional stiffness (and strength) of members has been considered in the analysis, the members shall be designed for torsion.

9.4.3.6.3 These provisions shall be applicable only when:

- a) Solid rectangular beams with $D > b$,
- b) Hollow rectangular beams with $D > b$ and wall thickness $t > b/4$,
- c) Solid T-beams and I-beams, whose webs comply with the requirement of $D > b$.

a) *Limiting Stress*

A concrete member shall resist the serviceability design torsional moment demand T_{SD} at the critical section by concrete, transverse reinforcement steel and longitudinal steel. T_{SD} shall be considered to increase both the serviceability design shear force demand V_{SD} and the serviceability design bending moment demand M_{SD} . These additional demands on V_{SD} and M_{SD} shall be taken as specified hereunder.

Under the action of shear force, at the critical section of a RC member:

1) *Contribution of Concrete:*

The nominal shear stress demand τ_{SD} in concrete shall not exceed the permissible shear stress τ_c ; and

(ii) *Contribution of Transverse Reinforcing Steel:*

The tension in transverse reinforcing steel stirrups shall not exceed the *Permissible Tensile Stress* σ_{st} .

Also, in the estimation of the *Serviceability Design Shear Force Capacity* V_s of a section, it shall be assumed that both concrete and transverse steel reach simultaneously these said *permissible stresses*.

Shear reinforcement shall be provided *only* in the form of vertical stirrups. Also, adequate control of torsional cracking shall be provided by the shear reinforcement specified hereunder.

(b) Critical Section

Sections located less than a distance d from the face of the support may be designed for the same torsional moment as computed at a distance d , where d is the effective depth of the member.

(c) RC Beams

(1) Design for Bending Moment –

The Total Design Bending Moment $M_{SD,T}$ acting at the critical section shall be:

$$M_{SD,T} = M_{SD} + M_{SDET},$$

where

M_{SD} = Serviceability Design Bending Moment Demand at the critical section of the member (obtained from Structural Analysis, under the action of loads applied as per combinations specified in Table 4 for *Serviceability Criterion*), and

M_{SDET} = Equivalent Design Bending Moment at the critical section of the member due to Serviceability Design Torsional Moment Demand T_{SD} on the member

$$= T_{SD} \frac{\left(1 + \frac{D}{b}\right)}{1.7},$$

in which

T_{SD} = serviceability design torsional moment demand T_{SD} on the member (obtained from structural analysis, under the action of loads applied as per combinations specified in Table 4 for serviceability criterion),

D = Larger dimension of the cross-section of the concrete member, and

b = Smaller dimension of the cross-section of the concrete member.

Further,

(1) If $M_{SDET} \leq M_{SD}$, then design the member for the total moment M_{DT} acting in the direction of M_D ; and

(2) If $M_{SDET} > M_{SD}$, then design the member for the total moment M_{DT} acting in the direction of M_{SD} and for a moment of $M_{SDET2} = (M_{SD} - M_{SDET})$ acting in the direction opposite to that of M_{SD} .

(2) Design for Shear Force –

The total serviceability design shear force demand $V_{SD,T}$ acting at the critical section shall be obtained as:

$$V_{SD,T} = V_{SD} + V_{SDET},$$

where

V_{SD} = Design Shear Force at the critical section of the member obtained from Structural Analysis, and

V_{SDET} = Equivalent Design Shear Force at the critical section of the member due to Torsional Moment T_D obtained from Structural Analysis

$$= 1.6 \frac{T_{SD}}{b},$$

in which

T_{SD} = serviceability design torsional moment demand T_{SD} on the member (obtained from Structural Analysis, under the action of loads applied as per combinations specified in Table 4 for *Serviceability Criterion*), and

b = Smaller dimension of the cross-section of the concrete member.

The total serviceability design shear stress demand $\tau_{SD,T}$ at the critical section shall be obtained as:

$$\tau_{SD,T} = \frac{V_{SD,T}}{bd}.$$

Shear reinforcement shall be provided as per the following:

(1) When $0 < \tau_{SD,T} \leq \tau_c$:

Minimum shear reinforcement shall be provided as per **9.2.4.1(b)**;

(2) When $\tau_c < \tau_{SD,T} \leq \tau_{c,max}$:

When two-legged closed hoops enclosing the corner longitudinal bars are used, the area A_{SV} of cross section of the 2 legs of transverse reinforcement shall be given by:

$$A_{sv} = \left(\frac{1}{\tau_{yV} \left(\frac{d_1}{s_V} \right)_{INT}} \right) \left[\frac{T_{SD,T}}{b_1} + \frac{V_{SD,T}}{2.5} \right] \geq \left(\frac{1}{\tau_{yV} \left(\frac{d_1}{s_V} \right)_{INT}} \right) (\tau_{SD,T} - \tau_c) bd$$

where

τ_{yV} = Characteristic strength of the bars used as stirrups,

d = Effective depth of member with respect to untensioned steel,

s_V = Spacing of transverse reinforcement along the length of the member, and

b_1 = Centre-to-centre distance between corner longitudinal bars in the direction of the width,

d_1 = Centre-to-centre distance between corner longitudinal bars in the direction of the depth,

$\left(\frac{d_1}{s_V}\right)_{INT}$ = Integral value of the number; and

(3) When $\tau_{c,max} < \tau_{SD,T}$:

The member shall be: (a) re-sized to increase its shear area, (b) grade concrete increased, and/or (c) percentage tension reinforcement be increased;

where

τ_c = Permissible Shear Stress in concrete as specified in **9.4.2**, and

$\tau_{c,max}$ = Maximum Permissible Shear Stress in concrete as specified in **9.4.2**.

9.5 DESIGN FOR DURABILITY

9.5.1 General Criteria

A durable structure shall be designed in such a way that it meets the requirements of strength, service and stability throughout its design service life. The level of deterioration over its design life does not impair the performance of the structure without significant loss of utility with anticipated level of maintenance but without requiring excessive /major maintenance.

The degree of deterioration may be estimated on the basis of analytical calculations, experimental investigation, experience from past constructions, or a combination of these.

9.5.1.1 *Factors affecting durability of concrete*

The factors influencing durability include:

- a) the environmental conditions;
- b) the cover to embedded steel;
- c) the type and quality of constituent materials;
- d) the cement content and water/cement ratio of the concrete;
- e) workmanship, to obtain full compaction and efficient curing;
- f) the shape and size of the member;
- g) design and detailing; and
- h) maintenance.

9.5.1.2 Complexity in structural form, execution and usage generally increase the sensitivity of the structure to an aggressive environment. Hence, to avoid detrimental effects of environment on the structure, the structures shall abide by the following:

- a) The exposed surfaces of structures should be constructed and designed in such a way to promote efficient drainage of water and to avoid standing pools and rundown of water.
- b) Care should be taken to minimize cracks that may collect or transmit water.
- c) Adequate curing to avoid the harmful effects of early loss of moisture.
- d) Member profiles and their intersections with other members shall be designed and detailed in a way to ensure easy flow of concrete and proper compaction during concreting.

Concrete is more vulnerable to deterioration due to chemical or climatic attack when it is in thin sections, in sections under hydrostatic pressure from one side only, in partially

immersed sections and at corners and edges of elements. Cyclic wetting and drying effects strongly accelerate the rate at which deleterious substances enter the concrete. The life of the structure can be lengthened by providing extra cover to steel, by chamfering the corners or by using circular cross sections or by using surface coatings which prevent or reduce the ingress of water, carbon dioxide or aggressive chemicals.

The degree of exposure anticipated for the concrete during its service life together with other relevant factors need to be considered. Maintenance plays an important role in sustained durability of concrete. The concrete mix to provide adequate durability under the conditions should be chosen taking account of the accuracy of current testing regimes for control and compliance as described in this standard.

9.5.2 *Mechanisms of Deterioration of Concrete*

9.5.2.1 Deterioration mechanisms are either chemical or physical in nature and may be originated from within the concrete, or result of the external environmental exposure conditions. Transport mechanism plays a significant role while deciding the rate and extent of deterioration. Some of the potential deterioration mechanism causing durability problems are classified as below:

a) Corrosion of reinforcement due to

- 1) Permeation of atmospheric carbon dioxide in the concrete (carbonation induced corrosion)
- 2) Presence of significant levels of chlorides in concrete (chloride induced corrosion)

b) Physical deterioration — This includes abrasion, freezing and thawing, weathering, shrinkage etc.

c) Chemical deterioration — This includes sulphate attack, alkali aggregate reaction, etc.

9.5.2.2 In most of the situations one type of deterioration mechanism dominate. However, in some circumstances several deterioration mechanisms can act together. This is referred to as synergetic deterioration. Under such circumstances, specialist literature may be referred and a risk-based approach might be adopted for guidance.

9.5.3 Exposure Classes related to Environmental Factors

Exposure classes specified hereunder are based on the chemical and physical conditions to which a concrete structure shall be exposed or been exposed in additions to the mechanical actions. Table 66 to 70 specifies various exposure classes considering different environmental conditions and concrete deterioration mechanisms.

The type of environment is defined by the combination of:

- a) General exposure classes related to the potential deterioration mechanism of corrosion of reinforcement as per Table 66 to 68.
- b) Specific exposure classes relating to other deterioration processes appropriate to each case as per Table 69 and Table 70.

Table 66 Exposure classes for concrete deterioration due to corrosion induced by carbonation (Where concrete containing reinforcement or other embedded metal is exposed to air and moisture)
(Clause 9.5.3)

Class designation	Class name	Description of the climate
(1)	(2)	(3)
C1	Hot- dry	Mean monthly maximum temperature above 30 °C with mean monthly relative humidity below 55%
C2	Warm humid and Temperate	The temperatures usually vary between 25–35 °C in summers with mean monthly relative humidity below 55%; while in winters, temperatures vary between 20–30 °C with monthly relative humidity above 75%
C3	Composite	For the whole year; it has a particular season for more than six months and may experience other seasons for the remaining period.
C4	Cold regions	Mean monthly maximum temperature less than 25 °C for all values of mean monthly relative humidity
NOTE Details of the climatic zone and precipitation zones are provided in Annex B.		

Table 67 Exposure classes for concrete deterioration due to Corrosion induced by chlorides other than from sea water (Where concrete containing reinforcement or other embedded metal is subject to contact with water)

containing chlorides, including deicing salts from sources other than sea water)

(Clause 9.5.3)

Class designation	Class name	Description of the environment
(1)	(2)	(3)
CH1	Wet, rarely dry	Swimming pools, Concrete exposed to water containing chlorides
CH2	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides.

Table 68 Exposure classes for concrete deterioration due to corrosion induced by chlorides from sea water (Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water)

(Clause 9.5.3)

Class designation	Class name	Description of the environment
(1)	(2)	(3)
SW1	Exposed to airborne salt but not in direct contact with sea water	a) Structures located up to 1km from the coast b) Structures located beyond 1 km and up to 10 km from the coast c) Structure located beyond 10 km and up to 50 km from the coast *
SW2	Permanently submerged	Parts of marine structures
SW3	Tidal, splash and spray zones	Parts of marine structures

*to be checked for carbonation also.

Table 69 Exposure classes for concrete deterioration due to sulphate attack

(Clause 9.5.3)

Class designation	Class name	Description of the environment	
		Soil	Water
(1)	(2)	(3)	(4)
S0	No risk	SO ₃ <0.2%	SO ₃ <300 ppm

S1	Moderate	SO ₃ =0.2% to 1.0%	SO ₃ =300 to 2500 ppm
S2	Severe	SO ₃ >1%	SO ₃ >2500 ppm
S3	Very Severe	Magnesium sulphate SO ₃ >1%	SO ₃ >2500 ppm

Table 70 Exposure classes for concrete deterioration due to Freeze/thaw attack (Where concrete is exposed to significant attack from freeze-thaw cycles whilst wet)
(Clause 9.5.3)

Class designation	Class name	Description of the environment
F1	Moderate	Concrete exposed to freezing and thawing cycles and occasional exposed to moisture
F2	Severe	Concrete exposed to freezing and thawing cycles and in continuous contact with moisture
F3	Very severe	Concrete exposed to freezing and thawing cycles and in continuous contact with moisture and exposed to de-icing chemicals

9.5.4 Design for durability

All structures shall be designed for durability for a given design service life by selecting appropriate grade of concrete, type of cement, SCM's, mix proportions and adequate cover to the reinforcement for the given exposure class.

9.5.4.1 Design Service Life

The period for which the structure or structural element is to be used for its intended purpose with anticipated maintenance but no major/substantial repair being necessary.

Design service life to be adopted for a structure may be decided by the owner/structural engineer based on its utility as required. For a preliminary guidance, the design service life for different type of structures are indicated in Table 71.

Table 71 Indicative values for the design service life t_{sl}
(Clause 9.5.4.1)

Sl No.	Type of structure	Design service life t_{sl} (in years)
i)	Temporary structures (structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary)	10
ii)	Replaceable structural parts like gantry girders, bearings, etc	10 – 25
iii)	Normal low rise buildings and other common structures	50-75
iv)	High rise structures, critical and important buildings such as hospitals, health care facilities, etc	75
v)	Monumental buildings structures, bridges, and other civil engineering structures	100 or more

9.5.4.2 Limit state of durability

Limit state of durability shall be decided such that deterioration produced by chemical and physical actions (other than the loads and actions of the structural analysis) does not affect the other five limit states as defined in this standard during its design life. Limit states of durability for deterioration due to corrosion shall be considered as given in this chapter.

To check limit state of durability following condition shall be verified:

$$t_l \geq t_{sl}$$

where,

t_l is time needed for the aggressive agent to produce a significant degradation;

and

t_{sl} is design service life.

9.5.4.3 Carbonation induced corrosion

Carbonation is a physiochemical process involving chemical and physical changes. In concrete, the reaction between hydrated calcium compounds, primarily Ca(OH)_2 , and atmospheric CO_2 causes a chemical change within the concrete that increases the risk of reinforcement corrosion. This process, known as carbonation of the concrete, causes a decrease in the alkalinity of the pore solution to below pH 9. The carbonation starts from the exposed concrete surface and penetrates slowly to the interior of the concrete. As the carbonation reaches the steel reinforcement, the corrosion of reinforcement may initiate.

9.5.4.3.1 In order to ensure adequate resistance towards carbonation of concrete structures, basic requirements of maximum water/cement ratio or water/cementitious ratio, minimum grade of concrete, minimum cementitious content, minimum nominal

cover and durability test requirements for 50 years design service life for normal structures are given in the table below. These values are applicable for normal structures with importance level 1 and quality assurance level 2 and 3 (medium and high). Importance level for different types of structures are given below.

<i>Type of structure</i>	<i>Importance Level</i>
Normal	1.0
Important	1.2
Very important	1.4

However, for other values of importance level or design service life more than 50 yrs and up to 100 years, detail limit state design for durability is to be carried out as per procedure given below. Limiting values given in Table 73 need to be adhered to. For details on quality assurance levels to be considered, refer clause **11.1**.

9.5.4.3.2 The requirements given in Table 72 apply to 20 mm nominal maximum size aggregate. For other sizes of aggregate, they should be changed as given in Table 73.

Table 72 Adjustments to Minimum Cement Contents for Aggregates Other Than 20 mm Nominal Maximum Size
(Clause 9.5.4.3.2)

Sl No.	Nominal Maximum Aggregate Size (mm)	Adjustment to Minimum Cement Contents in Table 7 (kg/m³)
(1)	(2)	(3)
i)	10	+40
ii)	20	0
iii)	40	-30

Table 73 Requirements for Normal Structures Subjected to Carbonation
(Clause 9.5.4.3.2)

Class designation	Description of the environment	Type of member	Type of Cement	Nominal depth of cover (in mm)	Maximum w/c or w/ccm	Minimum Grade of Concrete	Minimum cementitious content (kg/m ³)	Durability Test Requirement*
								Electrical Resistivity (kohm cm) at 28 days
C1	Hot-dry	Internal and external members	OPC	20	0.55	M20	300	> 10
			PPC or OPC+ fly ash	25	0.55	M20	300	> 20
			PSC or OPC+ GGBS	25	0.55	M20	300	> 40
C2	Warm humid and temperate	For internal members	OPC	20	0.52	M25/(M20 for cover ≥ 25 mm)	320 for m25 / 300 for m20	>10
			PPC or OPC+ fly ash	25	0.50	M25	320	>20
			PSC or OPC+ GGBS	25	0.50	M25	320	>40
		For external Members	OPC	25	0.50	M25	320	>10
			PPC or OPC+ fly ash	30	0.50	M25	320	>20
			PSC or OPC+ GGBS	30	0.50	M25	320	>40
C3	Composite	For internal members	OPC	20/	0.52	M25	300	>10
			PPC or OPC+ fly ash	25	0.50	M25	320	>20
			PSC or OPC+ GGBS	25	0.50	M25	320	>40
			OPC	25	0.50	M25	320	>10

		For external members	PPC or OPC+ fly ash	30	0.47	M30	320	> 20
			PSC or OPC+ GGBS	30	0.470	M30	320	> 40
C4	Cold region	Internal members	OPC	20	0.55	M20	300	> 10
			PPC or OPC+ fly ash	20	0.50	M20	300	>20
			PSC or OPC+ GGBS	20	0.50	M20	300	>40
		External members	OPC	25	0.55	M25	300	>10
			PPC or OPC+ fly ash	25	0.50	M25	300	>20
			PSC or OPC+ GGBS	25	0.50	M25	300	>40

NOTE

- 1 Durability test required for class C2 and C3, if grade of concrete is less than M30.
- 2 For QC level 1 and where durability tests are required but have not been conducted, then minimum grade of concrete will be increased to next level and maximum w/c ratio should be reduced by 0.05. However, for important projects, durability tests are recommended in all classes.
- 3 For internal members which may have easy access to water (such as members in and around toilets, kitchens, pantries and other water-handling areas), the values for external members shall be used.
- 4 For internal members of open type structures like parking garages which are exposed to moisture and temperature similar to external environment, the values given for external members will be used.
- 5 For QC tests values of durability tests obtained during mix design stage (which are likely to be stringent than values given above) will be used. Values given in table are values to be achieved at mix design stage. Acceptance criteria given in chapter..... On Quality control.
- 6 For QC purpose, other durability test such as oxygen permeability index test as per IS 516 (Part 2/Sec 3) can also be used provided that the same test shall be conducted at mix design stage to fix the requirements.
- 7 Minimum cement content given for OPC are for 43 grade OPC. For 33 grade OPC, the values shall be increased by 20 kg/m³ and w/c ratio to be reduced by 0.02.
- 8 For members exposed to high moisture content, water cement ratio may be reduced by 0.05.
- 9 "The minimum cement content, maximum free water cement ratio and minimum grade of concrete are individually related to exposure condition."

9.5.4.3.3 Limit state design for carbonation induced corrosion

Limit state of durability for carbonation induced corrosion will be the initiation of cracks due to reinforcement corrosion due to depassivation caused due to carbonation. To ensure this limit state is not reached during the design service life, the calculated service life (t_{cl}) as per procedure given below shall be more than design service life (t_{sl}).

To calculate the service life, the actions and effects to be considered (which cause the carbonation and subsequent cracking of RC structure), the parameters of resistance to carbonation and procedure to verify performance are given below.

9.5.4.3.3.1 Actions and effects

Climatic/environmental factors to be considered are:

- a) Atmospheric CO₂ levels
- b) Type of environment or climate

9.5.4.3.3.2 Characteristic value of CO₂ levels

Based upon geographical conditions, characteristic value of CO₂ (at mean design service life), CO_{2ck} is given below.

<i>Description</i>	<i>Characteristic value of CO₂ levels, CO_{2ck} (in ppm by mass)</i>
For all areas under exposure class C4 and areas in exposure class C1 to C3 but away from urban or industrial areas	350
For internal members of air-conditioned buildings in exposure class C1 to C3	400
External members of urban and non-aggressive industrial area in exposure class C1 to C3	450
NOTE	
Industrial areas where concentration of CO ₂ is high, shall be designed for the actual maximum expected value of CO ₂ during the service life but not less than the values given above.	

9.5.4.3.3.3 Effect of climate

Effect of climate need to be considered in the assessment of service life of concrete structures. The weather coefficient, W takes into account meso-climatic conditions such as humidity and temperature (see Table 74).

Table 74 Climate Coefficient
(Clause 9.5.4.3.3.3)

Class designation	Description of the climate	Type of member	Climate coefficient W
(1)	(2)	(3)	(4)
C1	Hot- dry	Internal and external	0.90
C2	Warm humid and Temperate	Internal	1.00
		External	1.10
C3	Composite	Internal	1.10
		External	1.25
C4	Cold/ mountainous	Internal	0.9
		External	1.0

9.5.4.3.3.4 Resistance to carbonation

Resistance to carbonation of cover concrete can be achieved by using properly designed concrete mixes (having adequate w/c ratio and minimum cement content) and by providing adequate cover to reinforcement. The resistance of concrete mix to carbonation is measured in terms of carbonation coefficient.

9.5.4.3.3.5 Carbonation coefficient

9.5.4.3.3.6 Characteristic carbonation coefficient

The characteristic carbonation coefficient (K_{ck}) for different binder systems can be determined as

$$K_{ck} = \begin{cases} 9.5\rho - 2.75 & \text{OPC} \\ 12.3\rho - 1.6 & \text{PPC and PSC} \end{cases}$$

where ρ is water-cement ratio or water cementitious ratio

NOTE

- 1 For concrete containing fly ash between 25 to 35 percent, K_{ck} as given for PPC is to be used.

- 2 For concrete containing fly ash up to 25 percent, K_{ck} shall be used in between OPC and PPC using linear interpolation.
- 3 For concrete containing GGBS up to 50 percent, K_{ck} shall be used in between OPC and PSC using linear interpolation.
- 4 For concrete containing GGBS more than 50 percent, K_{ck} as given for PSC is to be used.

The values of K_{ck} so determined are minimum values and need to be verified further at the time of mix design using accelerated carbonation or rapid durability tests as per 9.5.4.3.3.9 below. Higher of the two values shall be adopted in durability design.

9.5.4.3.3.7 Design carbonation coefficient

Design carbonation coefficient, K_d can be determined as

$$K_d = K_{ck}\gamma_{m1}$$

where γ_{m1} is the partial safety factor based on level of quality assurance (QA) and design service life as given below:

Level of Quality Assurance	Partial Safety Factor γ_{m1}	
	$t_{sl} \leq 75$ yrs	$t_{sl} > 75$ yrs
(1)	(2)	(3)
1 (normal)	1.15	1.20
2 (medium)	1.07	1.12
3 (high)	1.00	1.05

9.5.4.3.3.8 Nominal minimum cover, C

The characteristic value of cover shall be same as chosen nominal value, as no negative tolerance is permitted on cover. The design concrete cover, C_d (in mm) is given as

$$C_d = C/\gamma_{m2}$$

where

C is the characteristic cover (in mm) as per clause __

γ_{m2} is the partial safety factor based on level of quality assurance (QA) and is given as

Level of Quality Assurance	γ_{m2}
(1)	(2)

1 (normal)	1.10
2 (medium)	1.05
3 (high)	1.00

9.5.4.3.3.9 Verification of K_{ck}

- a) While conducting mix design, accelerated tests (namely electrical resistivity test) should be conducted to determine K_{ck} . Table 75 provides required values of electrical resistivity for different values of K_{ck} and binder type.

Table 75 For concrete made with OPC

[Clause 9.5.4.3.3.9(a)]

Sl No.	Type of Binder					
	OPC		PPC or OPC+Fly Ash		PSC or OPC +GGBS	
	K_{ck}	Electrical Resistivity (in kohm-cm) 28 days	K_{ck}	Electrical Resistivity (in kohm-cm) 28 days	K_{ck}	Electrical Resistivity (in kohm-cm) 28 days
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	1.0	12	≤ 2.5	30	≤ 2.5	52
ii)	1.5	11	3.0	26	3.0	50
iii)	2.0	10	3.5	24	3.5	47
iv)	2.5	9	4.0	22	4.0	44
v)	3.0	8.5	4.5	20	4.5	40
vi)			5.0	18	5.0	38
vii)			5.5	17	≥ 5.5	35

- b) For quality control purposes, other durability test like oxygen permeability test as per IS 516 (Part 2/Sec 3) can also be used provided that the same test shall be conducted at mix design stage to fix the required values. For important structures (particularly structures designed for service life more than 50 year, the mean carbonation coefficient K_m shall be determined through accelerated carbonation test as per IS 516 (Part 2/Sec 4). This test takes longer duration as compared to a) above. However, test as mentioned at a) shall also be conducted for determining the values to be considered for acceptance during quality control. Procedure for determination of K_m using IS 516 (Part 2/Sec 4) is given in Annex D. Thereafter, K_{ck} shall be determined from K_m using the following equation.

$$K_{ck} = K_m \times 1.1$$

Minimum values of K_{ck} shall be as per **9.5.4.3.3.9**.

9.5.4.3.3.10 Verification of performance against carbonation

For verification, it shall be ensured that

$$t_{sl} \leq t_i;$$

where $t_i = t_i + t_p$,

t_i is time for initiation of corrosion and shall be taken as time for carbonation to reach reinforcement front, that is when carbonation depth is equals nominal cover to rebar; and t_p is propagation time, that is time from initiation of corrosion to initiation of crack.

a) Determination of initiation period (t_i)

For determination of initiation period t_i for a given design cover and material properties or for determination of minimum design cover for a required value of t_i , the following equation shall be used:

$$C_d = W K_d \left(\frac{t_i \gamma_f CO_{2ck}}{500} \right) \rho$$

where,

t_i	corrosion initiation period (in years)
C_d	Design value of nominal cover (in mm)
W	Climate coefficient as per Table 9
K_d	Design value of carbonation coefficient
CO_{2ck}	characteristic value of CO_2
ρ	Factor, which depends upon the type of binder = 0.60 for concrete made using OPC = 0.45 for concrete made using PPC or PSC

NOTES

- 1 For concrete containing fly ash between 25 percent to 35 percent, ρ value as proposed for PPC to be used.
- 2 For concrete containing fly ash between 15 percent to 25 percent, linear interpretation to be done between OPC and PPC.
- 3 For concrete containing fly ash less than 15 percent, ρ value will be 0.60.
- 4 For concrete containing GGBS between 25 percent to 50 percent, linear interpretation to be done between OPC and PSC.
- 5 For concrete containing GGBS less than 25 percent, ρ value will be 0.60.

b) Determination of propagation period (t_p)

Considering initiation of corrosion crack as limit state, propagation period depends upon the rate of corrosion, concrete cover and grade of concrete which is a complex phenomenon. For normal building design purposes, values given in Table 11 may be used. For important structures, detailed assessment can be made using available models in literature.

Table 76 Propagation period (in years) for nominal cover of 20 mm

[Clause 9.5.4.3.3.10(b)]

Grade of concrete	Propagation Period (in years)													
	Concrete made using OPC								Concrete made using PPC/PSC or OPC+Fly ash or OPC+GGBS					
	C1	C2		C3		C4		C1	C2		C3		C4	
	I/E*	I	E	I	E	I	E	I/E	I	E	I	E	I	E
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
≤ M35	1	10	5	10	8	8	6	1	10	8	12	10	10	7
Above M35	1	12	7	15	10	10	8	1	12	10	15	12	12	9

* I stands for internal members and E for external members.

NOTES

- 1 For higher nominal cover, the values given in above table may be increased by 10 percent for every 10 mm increase in nominal cover.
- 2 For structural members prone to ingress of moisture through seepage from roofs or pipes, propagation period should be taken as 5 years (maximum).

9.5.4.4 Chloride Induced Corrosion

Presence of chlorides in concrete in a quantity exceeding a certain limit, accelerates the corrosion of the steel in the concrete and results in early deterioration of the structure. The higher the chloride content, or if subsequently exposed to warm moist conditions, the greater the risk of corrosion. Such chlorides may enter the concrete from an external environment or supplied to the concrete from materials such as cement, aggregate, chemical admixtures, mixing water and any other concrete making constituents. In order to protect steel against corrosion, it is very important to control and keep the total quantity of chloride ions supplied to the concrete from harmful chemical salts. The levels of such harmful salts in concrete coming from concrete materials, that is, cement, aggregates, water and admixtures, as well as by diffusion from the environment should be within the range that does not cause or accelerates steel corrosion.

Chlorides may enter the concrete from the environment or constituent materials such as cement, aggregates, chemical admixtures, mixing water, etc which can result in corrosion of reinforcement and early deterioration of structures.

Even where structures are not exposed to chloride and is exposed to air and moisture, the total quantity of chloride ions be controlled in terms of the total acid soluble chloride ion content of concrete during mixing. The total acid soluble chloride ion content of concrete during mixing is the total of the quantities of chloride ions that are thought to be supplied from the materials used in concrete calculated on the basis of mix proportions. Wherever possible, the total chloride content of the concrete should be determined.

For determination of total chloride content in concrete, total acid soluble chloride ion content for cementitious material and water-soluble chloride for all other constituents shall be considered. The chloride content so determined in concrete shall be within limits specified in Table 77.

The chlorides more than these prescribed limits can accelerate corrosion rate caused by carbonation or can initiate corrosion even in absence of carbonation, if chloride content is very high.

Table 77 Limits of Chloride Content of Concrete

(Clause 9.5.4.4)

Sl No.	Type or Use of Concrete	Maximum Total Acid Soluble Chloride Content (expressed as kg/m ³ of concrete)
(1)	(2)	(3)
i)	Concrete containing metal and steam cured at elevated temperature and prestressed concrete	0.4
ii)	Reinforced concrete or plain concrete containing embedded metal	0.6 (or 0.15 % by mass of cement, whichever is lower) // to be reviewed
iii)	Concrete not containing embedded metal or any material requiring protection from chloride	3.0

NOTES

- 1 For hardened concrete, whenever required total acid soluble chloride in reinforced concrete shall be determined as per IS 14959 and the same shall not exceed 0.2 percent by mass of cement.
- 2 Where cement content is not known, the total acid soluble chloride shall not exceed the value given in the above table.

9.5.4.4.1 Concrete in Sea water

Seawater contains dissolved salts that are potentially aggressive to concrete. The most significant aggressive chemicals present in seawater include chlorides, sulphates and magnesium. The mitigation of seawater exposure requires the use of mixture proportions that minimize the permeability. Lower permeability of concrete made with appropriate amounts of micro silica, slag or pozzolana cement and with their combination can be achieved.

Concrete in sea-water or exposed directly along the sea-coast shall be at least M 20 Grade in the case of plain concrete and M 35 in case of reinforced concrete. The use of silica fume, slag or pozzolana cement is advantageous under such conditions.

9.5.4.4.1.1 Special attention shall be given to the design of the mix to obtain the densest possible concrete; slag, broken brick, soft limestone, soft sandstone, or other porous or weak aggregates shall not be used.

9.5.4.4.1.2 As far as possible, preference shall be given to precast members unreinforced, well-cured and hardened, without sharp corners, and having trowel-smooth finished surfaces free from crazing, cracks or other defects; plastering should be avoided.

9.5.4.4.1.3 No construction joints shall be allowed within 600 mm below low water-level or within 600 mm of the upper and lower planes of wave action. Where unusually severe conditions or abrasion are anticipated, such parts of the work shall be protected by bituminous or silico-fluoride coatings or stone facing bedded with bitumen.

9.5.4.4.1.4 In reinforced concrete structures, care shall be taken to protect the reinforcement from exposure to saline atmosphere during storage, fabrication and use. It may be achieved by treating the surface of reinforcement with cement wash or by suitable methods

9.5.4.4.1.5 To ensure adequate resistance to chloride ingress, basic requirements with respect to minimum cover, maximum water/cement ratio or maximum water/cementitious ratio, minimum grade of concrete, minimum cementitious content, minimum nominal cover and durability test requirements for 50 years design service life for normal structures is given in Table 78. However, for other values of design service life up to 120 years, proper design for durability can be carried out as per **9.5.4.2**.

**Table 78 Requirements for Normal Structures Subjected to Chloride Induced Corrosion
for 50 Years Design Service Life for and $\gamma_m = 1.05^*$
(Clause 9.5.4.4.1.5)**

Class designation	Description of the environment	Type of Cement	Nominal depth of cover (in mm)	Maximum w/c or w/c _{cm}	Minimum Grade of Concrete	Minimum cementitious content (kg/m ³)	Durability Requirement	
							Electrical Resistivity (kohm cm) at 28 days	RCPT (Charge passed in coulombs)
SW1	a) Near shoreline (up to 0.1 km)	PPC or OPC+ fly ash	50*	0.35	M40	400	≥28	<600
		PSC or OPC+ GGBS	45	0.40	M35	380	≥48	<500
	b) 0.1 km to 0.5 km	PPC or OPC+ fly ash	45*	0.40	M35	380	≥24	601-800
		PSC or OPC+ GGBS	40	0.42	M35	360	≥46	<500
	c) 0.5 km to 1.0 km	OPC	45	0.42	M35	360	≥11	2001-2500
		PPC or OPC+ fly ash	30	0.5	M25	340	≥21	601-800
		PSC or OPC+ GGBS	30	0.50	M25	340	≥40	501-700
		OPC	35	0.50	M25	360	≥10	2001-2500

	d) 1.0 km to 10 km	PPC or OPC+ fly ash	30	0.50	M25	340	≥ 21	601-800
		PSC or OPC+ GGBS	30	0.50	M25	340	≥ 40	501-700
	e) 10 km to 50 km	OPC	30	0.50	M25	320	≥ 10	2501-3000
		PPC or OPC+ fly ash	25	0.50	M25	320	≥ 21	801-1000
		PSC or OPC+ GGBS	25	0.50	M25	320	≥ 40	501-700
	SW2, CH1	PPC or OPC+ fly ash	50*	0.35	M40	400	≥ 28	601-800
		PSC or OPC+ GGBS	45	0.40	M35	380	≥ 48	<500
SW3, CH2		PPC or OPC+ fly ash	55*	0.35	M40	400	≥ 28	<600
		PSC or OPC+ GGBS	50	0.41	M40	400	≥ 48	<500

* NOTES

- 1 In these specific 4 cases, cover can be reduced by 5mm by reducing w/c ratio by 0.05 and increasing min cementitious content by 40 kg. For other cases, cover cannot be reduced further.

- 2 In exposure classes SW1(b) and SW1(c), corrosion due to carbonation may also be a critical. Therefore, while designing RCC structures in this exposure class, requirements for corrosion due to carbonation shall also be checked.
- 3 For exposure classes where OPC is not mentioned, the same shall not to be used . (Because It is difficult to obtain required resistance to chloride ingress with reasonable concrete cover and w/c.)
- 4 For Y_m refer 9.2.6.1.
- 5 Composite cement conforming to IS 16415 can be used in exposure classes SW1(a), SW2 and CH1 in lieu of PPC. Resistivity values with use of composite cement, however need to be established for the specific cement proposed to be used.
- 6 Durability test required shall be done for exposure classes SW1(a), SW2 , SW3 and CH2.
- 7 In case durability tests are not performed for exposure classes SW1(a), SW2 , SW3 and CH2, then minimum grade of concrete shall be increased to next level and maximum w/c or w/c_{cm} to be reduced by 0.05. However for important projects, durability tests are recommended for all classes.

9.5.4.4.2 Limit state of chloride induced corrosion

Where concrete containing reinforcement or other embedded metal is subject to contact with chlorides from sea water or air carrying salt originating from sea water, the limit state of durability for deterioration due to chlorides shall be when a threshold value of chlorides induced corrosion is reached at the surface of steel reinforcement. To ensure this limit state is not reached during the design service life, the calculated service life (t_l) as per procedure given below shall be more than design service life (t_{sl}).

To calculate the service life, the actions and effects to be considered (which cause the chloride induced corrosion), the parameters of resistance to chloride ingress and procedure to verify performance are given below.

9.5.4.4.2.1 Actions and Effects**9.5.4.4.2.1.1 Surface chloride content (C_s)**

The characteristic value of surface chloride content C_{sck} shall be taken as specified in Table 79 corresponding to different exposure classes.

Table 79 Characteristic Value of Surface Chloride Content C_{sck}
(Clause 9.5.4.4.2.1.1)

Sl No.	Exposure Class			Characteristics surface chloride content C_{sck} (by weight of concrete , kg/m ³) for service life design of concrete structure
(1)	(2)			(3)
1.	SW1	a)	Near shoreline (up to 0.1 km)	9
			i) 0.1 km to 0.50 km	5.0
			ii) 0.50 km to 1 km	2.0
			iii) 1 km to 10 km	1.75
		b)	10 Km to 50 km	1
2.	SW2, CH1			9
3.	SW3, CH2			13

The design value of surface chloride content, $C_{sd} = C_{sck}\gamma_r$,
where

γ_r is partial factor of safety for action / effects and shall be taken as 1.05

Higher value of C_{sd} may be adopted for importance structures.

9.5.4.4.2.1.2 Threshold chloride content (C_{th})

Design value of threshold chloride content/ critical chloride content shall be taken as specified hereunder:

- a) For concrete made with OPC = 0.4% by mass of cement
- b) For concrete made with PPC or concrete containing fly ash $\geq 25\%$ by mass of cement = 0.3% by mass of cement
- c) For concrete made with PSC or concrete containing GGBS $\geq 50\%$ by mass of cement = 0.3% by mass of cement

9.5.4.4.2.2 Resistance to chloride ingress

Rate of ingress indicate the resistance of the concrete against the chloride ion penetration. Chloride diffusion coefficient D_a is considered as the rate determining parameter of chloride ingress in concrete, which is a time dependent function and decreases with time.

Based on w/c ratio, the initial diffusion coefficient $D_a(t_0)$ for normal structures can be determined using the following equations

$$D_a(t_0)(\text{in } mm^2/\text{yr}) = \begin{cases} 524\rho + 68 & \text{OPC} \\ 361\rho - 18.7 & \text{PPC} \\ 270\rho - 51 & \text{PSC} \end{cases}$$

where ρ is water-cement ratio or water cementitious ratio

NOTE

- 1 For concrete containing fly ash between 30 percent to 35 percent, D_a value as proposed for PPC to be used.
- 2 For concrete containing fly ash between 15 percent to 30 percent, for D_a value linear interpretation to be done between OPC and PPC.
- 3 For concrete containing fly ash < 15 percent, for D_a value as proposed for OPC shall be used.
- 4 For concrete containing GGBS between 25 percent to 50 percent, for D_a value linear interpretation to be done between OPC and PSC.
- 5 For concrete containing GGBS < 25 percent, for D_a value as proposed for OPC shall be used.

9.5.4.4.3 Characteristic $D_a(t_0)$ and design $D_a(t)$ values of chloride diffusion coefficient of concrete

The values of initial chloride diffusion coefficient as determined on the basis of w/c ratio shall be considered as characteristic values.

Design value of chloride diffusion coefficient for service life (exposure period) t shall be calculated as

$$D_d(t) = D_a(t_0)\gamma_m A_t$$

where,

$D_d(t)$ = Design value of chloride diffusion coefficient after exposure period, t

γ_m = Partial factor of safety of material, which shall vary from 1.0 to 1.15 depending upon quality control level selected and design service life as specified hereunder

Quality Assurance level	γ_m	
	$t_{sl} \leq 75 \text{ yr}$	$t_{sl} > 75 \text{ yr}$
1 (Normal)	1.11	1.15
2 (Medium)	1.05	1.1
3 (High)	1.0	1.05

A_t = time dependent multiplying factor and is given as $A_t = \left(\frac{t_0}{t_{eff}}\right)^\alpha$

t_0 = initial duration of hydration at the time of exposure in years and to be taken as 0.2466 yrs (for 90 days)

α = ageing factor and shall be taken as

Type of cement	α
OPC	0.30
PPC	0.50
PSC	0.45

t_{eff} = duration (in years) through which reduction in D_a will be there = $0.8 \times t_{sl}$, subject to maximum value of 60 yrs

9.5.4.4.4 Verification of chloride diffusion coefficient, $D_a(t_0)$

At design mix stage for a known w/c or w/cm ratio, performance based accelerated tests such as RCPT/electrical resistivity shall be conducted as applicable and value of $D_a(t_0)$ shall be determined based upon relationship as given in **Table 80**. The values given in Table 80 are valid for concrete made with Ordinary Portland cement, for concrete made with PPC or concrete containing fly ash between 30 percent to 35 percent by mass of cement and for concrete made with PSC or concrete containing GGBS upto 50 percent by mass of cement. For other percentages of SCM's, relationship should be drawn based upon the actual test values.

For important structures, $D_a(t_0)$ shall be determined through unidirectional diffusion test as per IS 516 Part..., which is generally a long duration test as compared to 7.3.5.3c i. Values determined using above test will be considered as mean value $D_{a,m}$. Characteristic value $D_a(t_0)$ shall be taken as:

$$D_a(t_0) = D_{a,m} * 1.1$$

NOTE: Durability tests as mentioned in shall also be conducted for determining the values to be considered for acceptance during quality control.

Table 80 Relationship between $D_a(t_0)$ and Electrical Resistivity / Charge passed (RCPT)
(Clause 9.5.4.4.4)

Sl No.	Chloride diffusion coefficient $D_a(t_0)$, mm^2/yr	Electrical Resistivity (kohm-cm) at 28 days	Charge passed (in coulombs) at 28 days	Remarks
(1)	(2)	(3)	(4)	(5)
i)	50	50	< 500	1. With OPC, it may not be possible to obtain $D_a(t_0) < 200$. For lower values, blended cements need to be used.
ii)	75	42	501-700	
iii)	100	29	< 600	
iv)	125	24	601-800	
v)	150	21	801-1000	
vi)	175	18	< 1500	2. With PPC, values of $D_a(t_0) < 100$ may not be possible. For values less than 100, PSC need to be used.
vii)	225	12	1501-2000	
viii)	250	12	2001-2500	
ix)	300	11	2501-3000	

9.5.4.4.5 Verification of performance against chloride ingress

Verification of performance against chloride ingress in designed exposure class shall be done by determining service life on the basis of critical surface chloride content and threshold chloride content value using modified Fick's second law of diffusion.

$$C(x,t) = C_s - (C_s - C_i) * \text{erf}(z)$$

where,

$C(x,t) =$	chloride content at cover depth x and exposure time t (in percentage by mass of concrete);
$C_s =$	chloride content at the exposed surface for different exposure classes as given in Table 77 (in percentage by mass of concrete);
$C_i =$	Initial chloride content to be determined as per IS 14959 Part 2 and should not be more than as specified in Table 77 clause 9.5.4.4 (in percentage by mass of concrete);
$\text{erf}(z) =$	error function (refer table in appendix .. for values of $\text{erf}(z)$ against diff values of z)

$D_d(t)$ is the design non-steady-state chloride diffusion coefficient, in m^2/sec for time t ;

$$z = \frac{x}{2\sqrt{(D_d(t)t)}}$$

x is the nominal cover in mm

t is the design service life in yrs,

$D_d(t)$ in mm^2/yr

In order to prevent chloride ingress it shall be ensured that $C(x,t) \leq C_{th}$ (C_{th} shall be considered as per 9.5.4.4.5)

9.5.4.4.6 Effect of crack width on service life due to corrosion of rebar (due to chloride ingress or carbonation)

Crack width can impact the durability, however effect of crack width at service loads on durability has not been established. The crack width control from serviceability requirements obtained through proper reinforcement detailing has been found to be sufficient in the absence of any established model for calculating its effect on durability. However for structures exposed to aggressive environment, lower crack widths are recommended. Table 81 provides general guidelines for crack width.

Table 81 Permissible crack width at service loads for different exposure classes
(Clause 9.5.4.4.6)

Sl No.	Exposure type	Permissible crack width at service loads
(1)	(2)	(3)
i)	Normal structures except as given under	0.3 mm
ii)	Structures or parts of structure exposed to high moisture continuously, alternate wetting and drying, parts buried under soil and structures within 0.1 km of coastline	0.2 mm
iii)	Structures or part of structures under sea water or in splash zone (exposure class, SW2 and SW3)	0.15 mm
iv)	Water retaining structures	Refer IS 3370

9.5.5 Sulphate Attack

Sulphate attack most often occurs in concrete exposed to internal or external sources which may result in expansion, cracking and eventual disintegration of the concrete.

Sulphate attack most often occurs in concrete exposed to external sources although internal sources of sulphate can also result in damage, particularly when concrete is exposed to excessive temperature at early ages. Sulphate ions attack the hydrates in the cement paste, leading to the formation of ettringite and gypsum. The formation of these compounds can cause expansion, cracking and eventual disintegration of the concrete.

Sulphate attack can either be 'external' or 'internal'. Internal sulphate attack generally occurs when a source of sulphate is incorporated into the concrete at the time of mixing. Internal source of sulphates mostly consists of cements, aggregates and water. The excessive amounts of water-soluble sulphate from these or other mix constituents can cause expansion and disruption of concrete. External sources of sulphate are more common and usually are a result of high-sulphate soils and ground waters, or can be the result of atmospheric or industrial water pollution.

To prevent this, the total water-soluble sulphate content of the concrete mix, expressed as SO_3 (where, $\text{SO}_3 = 0.833 \text{ SO}_4$), should not exceed 4 percent by mass of the cement in the mix. The sulphate content should be calculated as the total from the various constituents of the mix.

The 4 percent limit does not apply to concrete made with supersulphated cement complying with IS 6909.

NOTE - Delayed ettringite formation (DEF) is a special case of internal sulfate attack. It occurs in concrete which has been cured at elevated temperatures, for example, where steam curing has been used. It can also occur in large concrete pours where the heat of hydration has resulted in high temperatures within the concrete.

9.5.5.1 Design requirements against sulphate attack

Requirements for concrete to have adequate resistance to sulphate attack are given in Table 82.

Table 82 Requirements for Concrete against Sulphate Attack
(Clause 9.5.5.1)

Sl No.	Exposure class	Min. grade of concrete	Min. Cementitious Content (kg/m^3)	Max. w/cm	Min. clear cover (mm)	Type of cement
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	S0	M 25	320	0.50	30	Any cement allowed for RC
ii)	S1	M 35	340	0.45	40	SRC, PPC, PSC, OPC with slag or silica fume.
iii)	S2	M 40	400	0.40	50	SRC, PPC, PSC, OPC with slag or silica fume.
iv)	S3	M 50	400	0.40	50	SRC, PPC

NOTE

- 1 Cementitious content given above is irrespective of grades of cement. The cementitious content as given above are the minimum recommended. For SO_3 contents near the upper limit of any exposure class, cement contents above these minimum requirements are advised. For the purpose of calculation of SO_3 content (if the test results are in the form of SO_4), the relation $\text{SO}_3 = 0.833 \text{ SO}_4$ shall be used.
- 2 For severe conditions, such as thin sections under hydrostatic pressure on one side only and sections partly immersed, considerations should be given to a further reduction of water -cement ratio.
- 3 Where chlorides encountered along with the sulphates in soil or ground water, ordinary Portland cement with C_3A content from 5 to 8 percent shall be desirable to be used in concrete instead of sulphate resisting cement. Alternatively, Portland slag cement conforming to IS 455 having

more than 50 percent slag or a blend of ordinary Portland cement and slag may be used provided sufficient information is available on performance of such blended cements in these conditions.

- 4 Supersulphated cement is not readily available, and has issues of strength reduction when subjected to temperatures above 40°C. Thus, it has not been included in the above table.

9.5.6 Freezing and Thawing

Deterioration of concrete exposed to freezing generally occurs when there is sufficient amount of internal moisture. The transition of water to ice results an increase in volume. When the saturated pores in the concrete freezes, it can cause severe cracking and disruption.

Some of the factors that can affect the freezing and thawing behavior of concrete are:

- i. Moisture content
- ii. Curing regime and procedure
- iii. Severity of exposure condition (i.e. number of freeze cycle)
- iv. Compressive strength

Where freezing and thawing actions under wet conditions exist, enhanced durability can be obtained by the use of suitable air entraining admixtures. When concrete lower than grade M 50 is used under these conditions, the mean total air content by volume of the fresh concrete at the time of delivery into the construction shall be limited to the values specified as below:

<i>Nominal Maximum Size Aggregate (mm)</i>	<i>Entrained Air (In Percentage)</i>
(1)	(2)
10	7 ± 1
20	5 ± 1
40	4 ± 1

Since air entrainment reduces the strength, suitable adjustments may be made in the mix design for achieving required strength. It shall be noted that air entraining admixtures to be used shall comply with the requirements of IS 9103.

9.5.6.1 Design requirements against freezing and thawing

Requirements for concrete for adequate resistance against freezing and thawing are given in Table 83.

Table 83 Requirements for Concrete against Freezing and Thawing
(Clause 9.5.6.1)

Sl No .	Class designation	Type of Cement	Maximum w/c	Minimum grade of concrete	Minimum cementitious content (kg/m ³)	Additional requirements
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	F1	OPC or PPC or PSC	0.50	M25	300	Refer Table 17
2	F2	OPC or PPC or PSC	0.45	M30	320	
3	F3	PPC or PSC	0.45	M35	340	

NOTE - In case of exposure class F3, additional use of silica fume is advantages subjected to the permissible limits of 35 percent for fly ash and 50 percent for GGBS.

9.5.7 Alkali Aggregate Reactivity

Alkali aggregate reactivity have been recognized as Alkali silica reactivity (ASR) and Alkali carbonate reactivity (ACR). Alkali carbonate reaction is associated with certain dolomitic limestone and it involves a reaction between an alkali source and certain dolomites. Alkali silica reaction (ASR) results from the reaction between alkali hydroxide in the pore solution and certain forms of reactive silica present in some types of siliceous aggregates. ASR is far more widespread than ACR.

Some aggregates containing particular varieties of silica may be susceptible to attack by alkalis (Na₂O and K₂O) originating from cement or other sources, producing an expansive reaction which can cause cracking and disruption of concrete. Damage to concrete from this reaction will occur only when all the following are present together:

- A high moisture level, within the concrete;
- A cement with high alkali content, or another source of alkali; and
- Aggregate containing an alkali reactive constituent.

9.5.7.1 Tests for alkali silica reactivity

To determine the manifestation of alkali silica reactivity, accelerated mortar bar test as per IS 383 and/or mortar bar test as per IS 2386 Part VII shall be conducted. In case of calcite limestone or dolomitic limestone aggregates are encountered, concrete prism bar test shall be preferred over mortar bar test. Till this test is included in IS 2386 part VII, specialist literature may be referred for the test and applicable requirements.

Where the service records of particular cement/aggregate combination are well established and do not include any instances of cracking due to alkali-aggregate reaction, no further precautions should be necessary. When the materials are unfamiliar, one or more of the following precautions shall be taken:

- a) Use of non-reactive aggregate from alternate sources.
- b) Use of low alkali OPC having total alkali content not more than 0.6 percent (as Na_2O equivalent). Further advantage can be obtained by use of fly ash conforming to IS 3812 or granulated blast furnace slag conforming to IS 12089 as part replacement of ordinary Portland cement (having total alkali content as Na_2O equivalent not more than 0.6 percent), provided fly ash content is at least 25 percent or slag content is at least 50 percent.
Alternatively, PPC or PSC shall be used after determining the potential alkali-silica reactivity of combination of PPC or PSC proposed to be used at site for making concrete and aggregate using accelerated mortar bar method and expansion of aggregate shall be less than 0.10 percent at 16 days.
- c) Measures to reduce the degree of saturation of the concrete during service such as use of impermeable membranes; and
- d) Limiting the cement content in the concrete mix and thereby limiting total alkali content in the concrete mix. For more guidance, specialist literatures may be referred.

9.5.8 Abrasion

Abrasion of concrete surfaces results from friction, which may cause a grinding action; or by repetitive impact and overloading, which causes local crushing. Abrasion of concrete is broadly classified into following areas:

- i. Wear on floor and slab construction
- ii. Wear on concrete road surfaces
- iii. Erosion of hydraulic structure due to abrasive action of material carried by flowing liquid
- iv. Cavitation action on concrete in hydraulic structures due to high flow velocities and negative pressure

Abrasion resistant surfaces primarily involve consideration of surface finishing technique curing regime, aggregate characteristics, compressive strength and cement content. The use of controlled permeability formwork can help to reduce abrasion. Compressive strength of concrete may provide a relative indication of resistance.

Specialist literatures may be referred to for durability requirements of concrete surfaces exposed to abrasive action, for example, in case of machinery and metal tyres.

Abrasion resistance of concrete for industrial floors shall be determined as per IS 516 Part 9 through revolving disc method for the surfaces exposed to attrition, rubbing and friction and through under water abrasion test for hydraulic structures.

9.5.9 Protection Measures

In cases where normal protection measures are insufficient particularly in aggressive environment, special protection measures may be adopted such as:

- a) Applying surface coverings, using specific products for concrete protection (paints or coatings);
- b) Cathodic protection of reinforcements using sacrificial anodes or applied current; and
- c) Active corrosion inhibitors.

9.5.9.1 The special protection may have useful life shorter than that of the structural element and hence the design shall include schedules for the appropriate maintenance of these protection systems.

9.5.9.2 At sites where alkali concentrations are high or may become very high, the ground water should be lowered by drainage so that it cannot come into direct contact with the concrete. Additional protection may be obtained by the use of suitable impermeable barriers.

9.6 Robustness Criteria

This standard requires three effects to be considered as part of examining the robustness criteria of a structure, namely its resistance against fire, blast and impact.

9.6.1 Resistance against Fire

A structure or structural element shall be designed to possess adequate capacity in the materials exposed to fire to resist penetration of flames like integrity, appropriate resistance in all materials to heat transmission, and appropriate resistance against the reduction in load bearing capacity and excessive deformation. The resistance against fire of a structural element shall be expressed in terms of time (in hours) in accordance with IS 1641. And, essential requirements of fire safety of buildings with respect to details of construction are given in IS 1642.

These provisions are applicable only to normal strength concretes, and not to high-strength concretes (see **9.6.2.2**), concretes with low permeability (like self-compacting

concrete), and concretes with inadequate drying after curing. Such concrete members may be designed for fire resistance based on specialist literature.

Design of concrete structures for fire resistance shall be based on results from testing and from past performance of real structures. The component level testing in accordance with IS 1641, where mechanical and thermal boundary conditions of the specimen during testing closely reflect the boundary conditions of the component in the real structure, including the load eccentricity (if any), can be utilized to estimate the fire resistance of the component being designed. The test data from earlier tests can be interpolated to calculate the fire resistance of the component being designed.

In absence of the said test data, fire resistance in concrete structures shall be provided by:

- a) Prescriptive approach provided in 9.6.1.1 shall be adopted for normal buildings and structures, namely at least a minimum size of members and at least a minimum cover to reinforcement.
- b) Prescriptive approach involving empirical equations for buildings of intermediate importance, buildings with unusual framing systems (such as long spans, long cantilevers), or buildings with higher risk of spread of fire due to lack of compartmentalization. Approximate member level analysis to be performed to estimate the fire resistance of individual structural members.
- c) Performance-based approach for critical and important buildings and structures which involves detailed nonlinear method of analysis to be performed to study the transmission of heat and its consequences, and member design (like size, cover, reinforcement amount and detail).

The load combinations to be used for estimating the demands to check the sufficiency of fire resistance shall be as specified in Table 5; the owners may specify more load combinations than those specified in Table 5. Therein, the temperature load shall be considered for checking the strength capacity of the structure or its structural member to withstand the effects of fire. The temperature loads shall include the thermal expansion effects and the resulting change in the member force demands due to the restraint from neighbouring members. Also, the reinforcement detailing shall reflect the changing cross-sections of members, and ensure that both individual members and the structure as a whole contain adequate support, ties, bonds and anchorages for the required fire resistance.

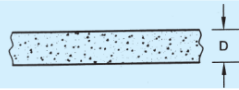
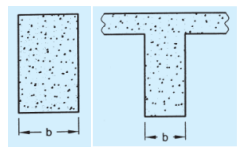
9.6.1.1 Prescriptive Approach

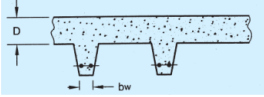
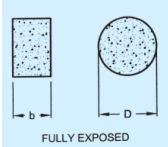
In the absence of more rigorous estimates of effects of temperature, the following prescriptive values shall be adopted in normal-weight aggregate concrete structures to ensure their fire resistance:

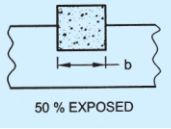
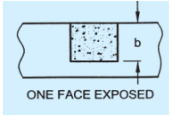
- (1) The minimum size of concrete members shall be as specified in Table 84; and
 (2) The minimum nominal cover to reinforcement bars shall be as specified in Table 85.

Additional measures (like application of fire resistant finishes, provision of fire resistant false ceilings and sacrificial steel in tensile zone) shall be adopted when nominal cover required exceeds 40 mm for beams and 35 mm for slabs, to give protection against spalling. Specialist literature may be referred to for prescriptive requirements to determine fire resistance of concrete structures that have not been covered under this clause.

Table 84 Minimum Sizes of Concrete Members to meet Prescriptive Requirements of Fire Resistance
 (Clause 9.6.1.1)

Sl No	Member	Condition of Member	Type of Concrete	Minimum Size of Members (mm)				
				Fire Resistance Rating (hours)				
				1.0	1.5	2.0	3.0	4.0
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Thickness D of slabs 	Simply Supported	Normal Weight Concrete	125	125	125	160	185
			Light Weight Concrete	900	900	1000	1200	1400
		Continuous	Normal Weight Concrete	125	125	125	150	170
			Light Weight Concrete	900	900	1000	1100	1300
ii)	Width b of beams 	Simply Supported	Normal Weight Concrete	200	200	200	240	280
			Light Weight Concrete	140	140	150	170	200
		Continuous	Normal Weight Concrete	200	200	200	235	255
			Light Weight Concrete	140	140	150	165	195
iii)	Prestressed beams	Simply Supported	Normal Weight Concrete					
			a) Quartzite, Granite, Basalt, Sandstone	225	225	225	260	300
			b) Limestone	200	200	200	235	270

			Light Weight Concrete	160	160	160	180	210
		Continuou s	Normal Weight Concrete					
			a) Quartzite, Granite, Basalt, Sandstone	225	225	225	255	275
			b) Limestone	200	200	200	230	245
			Light Weight Concrete	160	160	160	175	185
iv)	Width b_w of ribs of waffle slabs	Simply Supported	Normal Weight Concrete					
			a) Quartzite, Granite, Basalt, Sandstone	125	125	125	150	175
			b) Limestone	100	100	120	135	160
			Light Weight Concrete	90	90	100	115	125
		Continuou s	Normal Weight Concrete					
			a) Quartzite, Granite, Basalt, Sandstone	125	125	125	140	155
			b) Limestone	100	100	120	120	145
			Light Weight Concrete	90	90	100	100	115
v)	Columns	Fully exposed to fire	Normal Weight Concrete					
			a) Quartzite, Granite, Basalt, Sandstone	200	250	300	400	450
			b) Limestone	180	225	270	360	400

	 		Light Weight Concrete	14 0	17 5	21 0	28 0	31 5
		50 percent exposed to fire	Normal Weight Concrete					
			a) Quartzite , Granite, Basalt, Sandstone	16 0	20 0	25 0	30 0	35 0
			b) Limestone	14 5	18 0	22 5	27 0	31 5
			Light Weight Concrete	11 0	14 0	17 5	21 0	25 0
		One face exposed to fire	Normal Weight Concrete					
			a) Quartzite , Granite, Basalt, Sandstone	12 0	14 0	16 0	20 0	24 0
			b) Limestone	11 0	12 5	14 5	18 0	21 5
			Light Weight Concrete	90	10 0	11 5	14 0	17 0
vi)	Walls	Two faces exposed to fire	Normal Weight Concrete					
			a) Quartzite , Granite, Basalt, Sandstone	15 0	17 5	20 0	25 0	30 0
			b) Limestone	12 5	14 0	16 0	20 0	24 0
			Light Weight Concrete	12 5	12 5	12 5	15 0	18 0
		One face exposed to fire	Normal Weight Concrete					
			a) Quartzite , Granite, Basalt, Sandstone	14 0	15 0	16 0	20 0	24 0
			b) Limestone	12 5	13 5	14 5	18 0	21 5

			Light Weight Concrete	10 0	10 5	11 5	14 0	17 0

Table 85 Minimum Nominal Cover to meet Prescriptive Requirements of Fire Resistance
(Clause 9.6.1.1)

Sl No.	Member	Condition of Member	Type of Concrete	Minimum Nominal Cover (mm)				
				Fire Resistance Rating (hours)				
				1.0	1.5	2.0	3.0	4.0
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
i)	Slabs	Simply Supported	Normal Weight Concrete	20	25	35	45	55
			Light Weight Concrete	20	20	25	35	45
		Continuous	Siliceous	20	20	25	35	45
			Carbonaceous	20	20	20	30	40
			Light Weight	20	20	20	25	35
ii)	Beams	Simply Supported	Siliceous	30	40	60	70	80
			Carbonaceous	20	20	35	55	65
			Light Weight	20	20	30	50	60
		Continuous	Siliceous	20	35	50	60	70
			Carbonaceous	20	20	25	35	45
			Light Weight	20	20	20	30	40
iii)	Prestressed Beams	Simply Supported	Siliceous	40	55	70	80	90
			Carbonaceous	40	50	65	75	85
			Light Weight	40	45	60	70	80
		Continuous	Siliceous	30	40	55	70	80
			Carbonaceous	30	35	50	65	75
			Light Weight	30	30	45	60	70
iv)	Ribbs of Waffle Slabs	Simply Supported	Siliceous	25	35	45	55	65
			Carbonaceous	20	30	40	50	60
			Light Weight	20	25	35	45	55
		Continuous	Siliceous	20	20	35	45	55
			Carbonaceous	20	20	30	40	50
			Light Weight	20	20	25	35	45
v)	Columns	Fully exposed to fire	Siliceous	40	45	50	60	70
			Carbonaceous	35	40	45	55	65
			Light Weight	30	35	40	50	60
			Siliceous	40	45	50	55	60

		50% exposed to fire	Carbonaceous	35	40	45	50	55
			Light Weight	30	35	40	45	45
		One face exposed to fire	Siliceous	30	35	40	45	50
			Carbonaceous	30	35	40	40	45
			Light Weight	30	35	40	40	40
vi)	Walls	Two faces exposed to fire	Siliceous	40	45	50	60	70
			Carbonaceous	35	40	45	55	65
			Light Weight	30	35	40	50	60
		One face exposed to fire	Siliceous	40	45	50	55	60
			Carbonaceous	35	40	45	50	55
			Light Weight	30	35	40	45	45

9.6.1.2 Prescriptive Approach involving Empirical Equations

The requirements of member sizes and reinforcement cover as calculated in 9.6.1.1 shall be applicable in addition to the provisions of this section.

9.6.1.2.1 Columns

The fire resistance of RC columns can be calculated by either of the following two methods:

a) Critical Temperature Method

This method can be used for short columns when:

- 1) the column axial load (calculated as per load combination 24 in Table 5) is less than 50 percent of the design axial load capacity in compression at ambient temperatures (calculated as per 9.3), and
- 2) the combined stiffness of the axial restraint of all the stories above the column against the expansion of the column is less than 10 percent of the axial stiffness of the column.

Fire resistance of an RC column shall be estimated as the minimum of the time taken for the temperature in the outer most layer of lateral reinforcement to reach 500°C, and 50 percent area of the column cross-sectional to reach 500°C. Temperatures inside the column cross-section can be determined with the help of the isotherm graphs given in Fig. 82 and 83, or by conducting a two-dimensional transient heat transfer analysis of the column cross-section as specified hereunder:

The procedure to perform two-dimensional transient heat transfer analysis of the column.

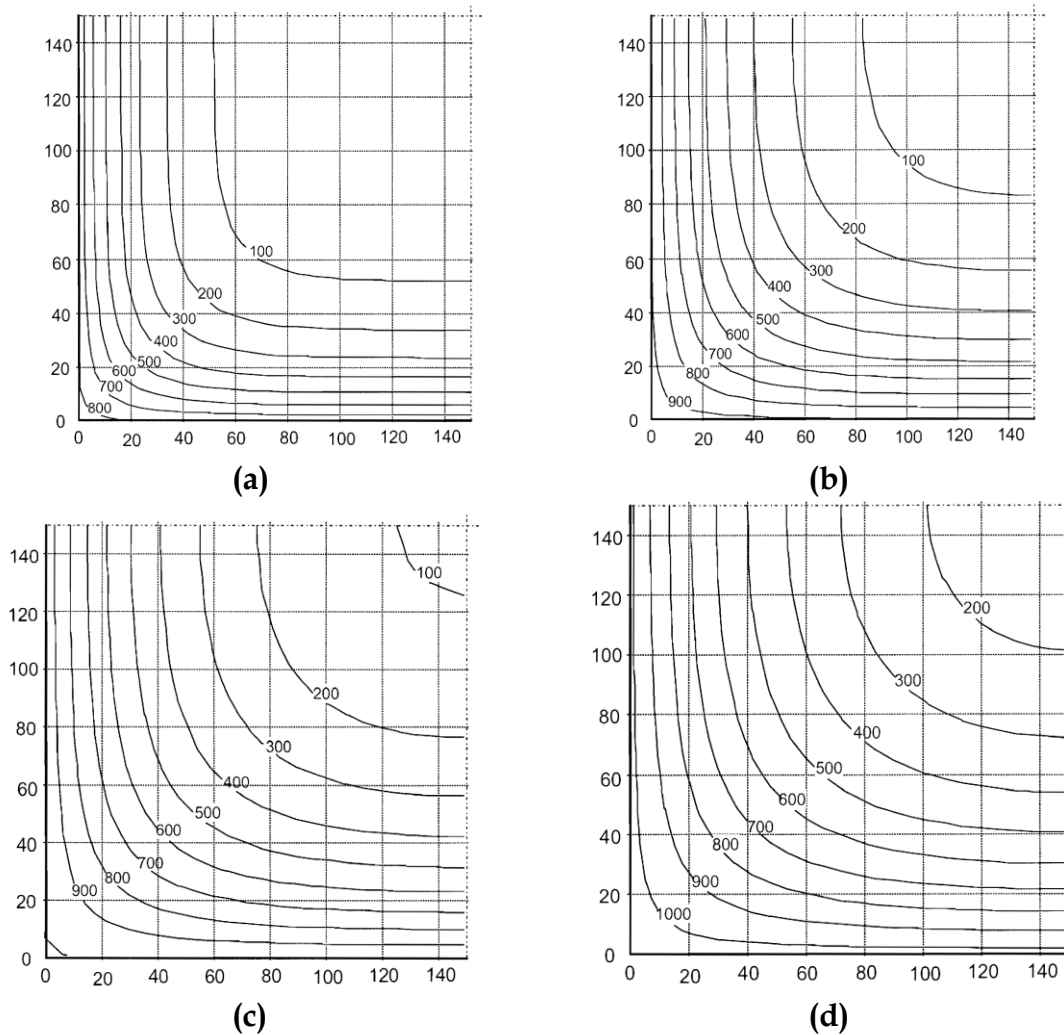


FIG. 82 TEMPERATURE PROFILES ACROSS THE DEPTH OF SQUARE COLUMNS OF 300MM $\times$ 300MM DIAMETER: (A) R = 30 MINUTES, (B) R = 60 MINUTES, (C) R = 90 MINUTES, AND (D) R = 120 MINUTES

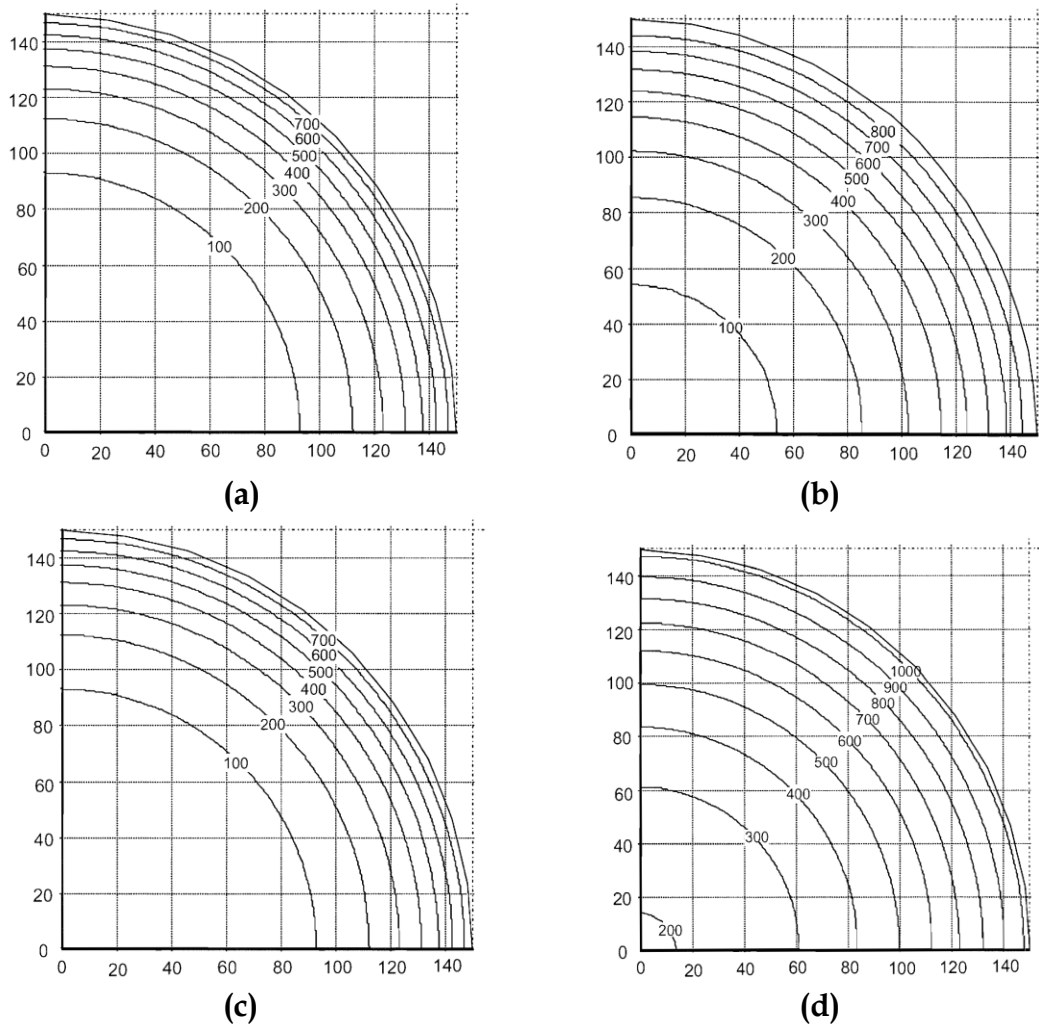


FIG. 83 TEMPERATURE PROFILES ACROSS THE DEPTH OF CIRCULAR COLUMNS OF 300MM DIAMETER: (A) $R = 30$ MINUTES, (B) $R = 60$ MINUTES, (C) $R = 90$ MINUTES, AND (D) $R = 120$ MINUTES

b) *Load Capacity Method*

The fire resistance R (minutes) of an RC column shall be calculated as:

$$R = 0.98C_t [8k(SR + 5)(0.2 - LR) + 30],$$

where LR is the load ratio of the column given by the maximum axial load on the column (as per combination 24 in Table 5) divided by the design load capacity of the column, and SR the slenderness ratio of the column

In the estimation of load ratio, the increase in the axial force demand in the column owing to thermal expansion of a column restrained with very stiff beams or wall systems, shall be estimated as per specialist literature. Here, the structural analysis shall include the thermal expansion effect due to fire loads. Average temperature of the column cross-section can be calculated from the

temperature field in the column cross-section. The temperature field in the column cross-section can be determined with the help of the isotherm graphs given in Fig. 82 and 83 or by conducting a 2-dimensional transient heat transfer analysis of the column cross-section specified in **9.6.1.2.1(a)**. The thermal properties (conductivity, specific heat, and density of concrete and steel) required for this analysis shall be taken as per **6**.

9.6.1.2.2 RC Slabs and Walls

The...(to be discussed)

9.6.1.2.3 RC Beams

The...(to be discussed)

9.6.1.3 Performance-based Approach

The requirements specified hereunder shall be complied with to ensure their fire resistance:

- a) The requirements specified in the prescriptive approach in **9.6.1.1** shall be complied with.
- b) Nonlinear analysis shall be performed as per **8.1.2** and shown that the concrete structure meets the following requirements:
 - 1) The design strength capacities of members re-estimated (in lines with strength criteria calculations using material properties corresponding to the higher temperatures but without the partial safety factors) shall be such that they resist the combined effects of stress resultant demands obtained from nonlinear analysis of the structure under the load combinations given in Table 3.
 - 2) The inelastic deformations at critical locations of the structure and in members shall be less than the limits specified in Table 86.

Table 86 Permissible deformations in members and the structure under actions associated with Temperature Load
[Clause 9.6.1.3(b)(2)]

Sl No.	Action	Permissible value
(1)	(2)	(3)
i)	Inelastic Rotation	
	(a) Beams	0.020 radians
	(b) Columns	0.005 radians

	(c) Walls in out-of-plane directions	0.005 radians
ii)	Inelastic Transverse Deflection Ratio	
	(a) Beams	0.04
iii)	Inelastic Lateral Drift Ratio	
	(a) Columns	0.03
	(b) Walls in out-of-plane directions	0.03

9.6.1.3.1 Time –Temperature Curves

The standard time-temperature curve specified in IS 3809 shall be used for estimating air temperatures of the fire loading. In case a more accurate Specific Time-Temperature Curve is available for the air temperatures of the fire loading based on the expected fire conditions in the structure, the same may be used, provided that the Specific Time-Temperature Curve is more stringent than the standard curve.

9.6.2 Mechanical Properties of Materials at Elevated Temperatures

The properties of concretes and reinforcing steel shall be taken as specified hereunder for the purposes of estimation of fire resistance.

9.6.2.1 Normal weight concrete

The variation of salient parameters of stress-strain concrete of uniaxially stressed normal weight concrete with siliceous or calcareous aggregates (containing at least 80 percent calcareous aggregate by weight) at elevated temperatures, namely characteristic compressive strength $f_{ck}(T)$, strain $\varepsilon_{cp}(T)$ at peak stress $f_{ck}(T)$, and ultimate strain $\varepsilon_{cu}(T)$, shall be taken as per Table 87. Possible strength gain of concrete in the cooling phase shall not be considered.

Table 87 Variation in Characteristic Compressive Strength $f_{ck}(T)$, Strain $\varepsilon_{cp}(T)$ at peak stress $f_{ck}(T)$, and Ultimate Strain $\varepsilon_{cu}(T)$ of Normal Weight Concrete
(Clause 9.6.2.1)

Sl No.	Concrete Temperature (°C)	Siliceous Aggregates			Calcareous Aggregates		
		$f_{ck}(T)/f_{ck}$	$\varepsilon_{cp}(T)$	$\varepsilon_{cu}(T)$	$f_{ck}(T)/f_{ck}$	$\varepsilon_{cp}(T)$	$\varepsilon_{cu}(T)$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	20	1.00	0.0025	0.0200	1.00	0.0025	0.0200
ii)	100	1.00	0.0040	0.0225	1.00	0.0040	0.0225
iii)	200	0.95	0.0055	0.0250	0.97	0.0055	0.0250
iv)	300	0.85	0.0070	0.0275	0.91	0.0070	0.0275
v)	400	0.75	0.0100	0.0300	0.85	0.0100	0.0300
vi)	500	0.60	0.0150	0.0325	0.74	0.0150	0.0325
vii)	600	0.45	0.0250	0.0350	0.60	0.0250	0.0350

viii)	700	0.30	0.0250	0.0375	0.43	0.0250	0.0375
ix)	800	0.15	0.0250	0.0400	0.27	0.0250	0.0400
x)	900	0.08	0.0250	0.0425	0.16	0.0250	0.0425
xi)	1,000	0.04	0.0250	0.0450	0.06	0.0250	0.0450
xii)	1,100	0.01	0.0250	0.0475	0.02	0.0250	0.0475
xiii)	1,200	0.00	-	-	0.00	-	-

9.6.2.2 High strength concrete

High strength concretes shall be grouped into three classes for the purposes of design for resistance against fire, namely:

- Class 1 Concretes with f_{ck} in the range 60–75 MPa;
- Class 2 Concretes with f_{ck} in the range 75–90 MPa; and
- Class 3 Concretes of grades M95 to M100.

When the actual characteristic strength of concrete used is likely to be of a higher class than that specified in design, the relative reduction in strength for the higher class shall be used for design for resistance against fire.

9.6.2.2.1 Compressive Strength

The compressive strength of concrete under elevated temperature shall be taken as per Table 88.

Table 88 Influence of Temperature on Characteristic Compressive Strength of Concrete

(Clause 9.6.2.2.1)

Sl No.	Concrete Temperature (°C)	$f_{ck}(T) / f_{ck}$		
		Class 1	Class 2	Class 3
(1)	(2)	(3)	(4)	(5)
i)	20	1.00	1.00	1.00
ii)	50	1.00	1.00	1.00
iii)	100	0.90	0.75	0.75
iv)	200	??	??	0.70
v)	250	0.90	??	??
vi)	300	0.85	??	0.65
vii)	400	0.75	0.75	0.45
viii)	500	??	??	0.30
ix)	600	??	??	0.25
x)	700	??	??	??
xi)	800	0.15	0.15	0.15
xii)	900	0.08	??	0.08

xiii)	1,000	0.04	??	0.04
xiv)	1,100	0.01	??	0.01
xv)	1,200	0.00	0.00	0.00

9.6.2.2.2 Tensile strength

The tensile strength of concrete shall be ignored (conservative). When tensile strength of concrete is required to be considered, the influence of temperature on tensile strength shall be taken as per the tensile strength reduction factor $k_t(T)$ given by:

$$k_t(T) = \begin{cases} 1.0 & 0 < T < 100^\circ\text{C} \\ 1 - \left(\frac{T - 600}{500} \right) & 100^\circ\text{C} < T < 600^\circ\text{C} \end{cases}$$

9.6.2.2.3 Stress-strain curve

The...(to be discussed)

9.6.2.2.4 Modulus of elasticity

The influence of temperature on modulus of elasticity E_c shall be taken as per the modulus reduction factor $k_E(T)$ given by:

$$k_E(T) = \begin{cases} 1.0 & 0 < T < 150^\circ\text{C} \\ 1 - \left(\frac{T - 700}{550} \right) & 150^\circ\text{C} < T < 700^\circ\text{C} \end{cases}$$

9.6.2.2.5 Thermal elongation

The elongation of high strength concrete at elevated temperature ($T^\circ\text{C}$) shall be taken as the value corresponding to that at 20°C multiplied by strain reduction factor $k_\varepsilon(T)$ given by:

a) For concretes with siliceous aggregates:

$$k_\varepsilon(T) = \begin{cases} -1.8 \times 10^{-4} + 9 \times 10^{-6}T + 2.3 \times 10^{-11}T^3 & 20^\circ\text{C} < T < 700^\circ\text{C} \\ +140 \times 10^{-4} & 700^\circ\text{C} < T < 1,200^\circ\text{C} \end{cases}$$

b) For concretes with calcareous aggregates:

$$k_\varepsilon(T) = \begin{cases} -1.2 \times 10^{-4} + 6 \times 10^{-6}T + 1.4 \times 10^{-11}T^3 & 20^\circ\text{C} < T < 850^\circ\text{C} \\ +120 \times 10^{-4} & 850^\circ\text{C} < T < 1,200^\circ\text{C} \end{cases}$$

9.6.2.2.6 Specific heat

The specific heat $C_p(T)$ (J/kg $^\circ\text{K}$) of dry concrete (with percentage moisture content $\mu=0$ by weight of concrete) shall be taken for concretes with siliceous aggregates as:

$$C_p(T) = \begin{cases} 900 & 20^\circ\text{C} < T < 100^\circ\text{C} \\ 900 + (T - 100) & 100^\circ\text{C} < T < 200^\circ\text{C} \\ 1000 + \left(\frac{T - 200}{2}\right) & 200^\circ\text{C} < T < 400^\circ\text{C} \\ 1100 & 400^\circ\text{C} < T < 1,200^\circ\text{C} \end{cases}$$

Where moisture is present in concretes with siliceous aggregates, the peak values of $C_{p,\max}$ (J/kg°K) for different moisture contents shall be taken as:

$$C_{p,\max} = \begin{cases} 900 & \mu = 0.0\% \\ 1470 & \mu = 1.5\% \\ 2020 & \mu = 3.0\% \end{cases}$$

$C_{p,\max}$ occurs at a temperature of 115°C. $C_p(T)$ shall be considered to vary linearly in the ranges (110°C-115°C) and (115°C-200°C).

9.6.2.2.7 Density

The variation in density of concrete with temperature shall be taken as:

$$\rho_c(T) = \rho_c(20^\circ\text{C}) \times \begin{cases} 1.00 & 20^\circ\text{C} < T < 115^\circ\text{C} \\ 1.00 - 0.02\left(\frac{T - 115}{85}\right) & 115^\circ\text{C} < T < 200^\circ\text{C} \\ 0.98 - 0.03\left(\frac{T - 200}{200}\right) & 200^\circ\text{C} < T < 400^\circ\text{C} \\ 0.95 - 0.07\left(\frac{T - 400}{800}\right) & 400^\circ\text{C} < T < 1,200^\circ\text{C} \end{cases}$$

9.6.2.2.8 Thermal conductivity

The variation of upper limit of thermal conductivity $\lambda_c(T)$ (W/m°K) of normal concretes with temperature (in the range 20°C - 1,200°C) shall be taken as:

$$\lambda_c(T) = 2.00 - 0.2451\left(\frac{T}{100}\right) + 0.0107\left(\frac{T}{100}\right)^2,$$

and the variation of lower limit of thermal conductivity $\lambda_c(T)$ (W/m°K) of normal concretes with temperature (in the range 20°C - 1,200°C) shall be taken as:

$$\lambda_c(T) = 1.36 - 0.136\left(\frac{T}{100}\right) + 0.0057\left(\frac{T}{100}\right)^2.$$

9.6.2.3 Reinforcing steel

The strength and deformation properties of reinforcing steel at elevated temperatures shall be defined by an idealised elasto-plastic stress-strain curve (Fig. 84), which is

characterised by three salient properties, namely initial elastic modulus $E_s(T)$, proportional limit $f_{s,p}(T)$, and yield stress $f_{s,y}(T)$.

The variation with temperature of these salient properties shall be taken as given in Table 89. Linear interpolation may be employed for obtaining the properties at intermediate temperatures. The strength of reinforcing steel at elevated temperatures shall be taken the values given in Table 89 multiplied by strength reduction factor $k_s(T)$, when the steel is in:

- a) Compression in columns,
- b) Compression zone in beams and slabs, and
- c) Tension when strain in steel is less than 0.02,

where

$$k_s(T) = \begin{cases} 1.00 & 20^\circ\text{C} < T < 100^\circ\text{C} \\ 0.70 - 0.30\left(\frac{T-400}{300}\right) & 100^\circ\text{C} < T < 400^\circ\text{C} \\ 0.57 - 0.13\left(\frac{T-500}{100}\right) & 400^\circ\text{C} < T < 500^\circ\text{C} \\ 0.10 - 0.47\left(\frac{T-700}{200}\right) & 500^\circ\text{C} < T < 700^\circ\text{C} \\ 0.10\left(\frac{1,200-T}{500}\right) & 700^\circ\text{C} < T < 1,200^\circ\text{C} \end{cases}$$

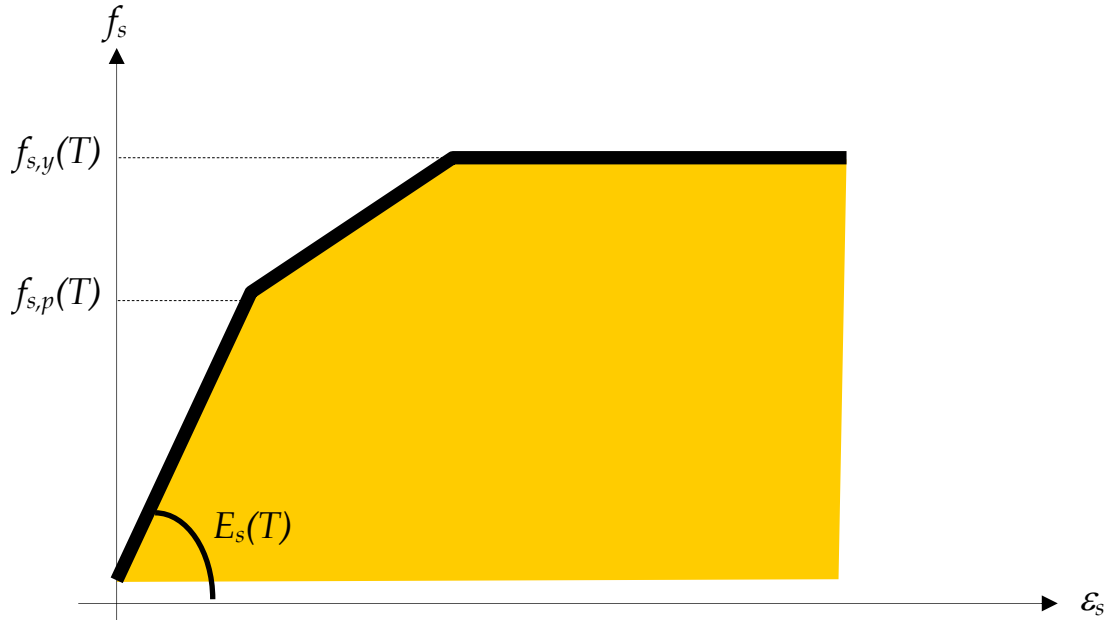


FIG. 84 SALIENT PARAMETERS OF THE STRESS-STRAIN CURVE OF REINFORCING STEEL

Table 89 Variation with Temperature of salient properties characterizing the idealised Stress-Strain Curve of Reinforcing Steel
(Clause 9.6.2.3)

Sl No.	Steel Temperature (°C)	$f_{s,y}(T)/f_y$		$f_{s,p}(T)/f_y$		$E_s(T)/E_s$	
		$f_{s,y}(T)/f_y$	$f_{s,y}(T)/f_y$	$f_{s,p}(T)/f_y$	$f_{s,p}(T)/f_y$	$E_s(T)/E_s$	$E_s(T)/E_s$
		Hot Rolled	Cold Worked	Hot Rolled	Cold Worked	Hot Rolled	Cold Worked
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	20	1.00	1.00	1.00	1.00	1.00	1.00
ii)	100	1.00	1.00	1.00	0.96	1.00	1.00
iii)	200	1.00	1.00	0.81	0.92	0.90	0.87
iv)	300	1.00	1.00	0.61	0.81	0.80	0.72
v)	400	1.00	0.94	0.42	0.63	0.70	0.56
vi)	500	0.78	0.67	0.36	0.44	0.06	0.40
vii)	600	0.47	0.40	0.18	0.26	0.31	0.24
viii)	700	0.23	0.12	0.07	0.08	0.13	0.08
ix)	800	0.11	0.11	0.05	0.06	0.09	0.06
x)	900	0.06	0.08	0.04	0.05	0.07	0.05
xi)	1,000	0.04	0.05	0.02	0.03	0.04	0.03
xii)	1,100	0.02	0.03	0.01	0.02	0.02	0.02
xiii)	1,200	0.00	0.00	0.00	0.00	0.00	0.00

The elongation of reinforcing steel at elevated temperatures shall be taken the values given in the above table multiplied by strain reduction factor $k_\varepsilon(T)$ given by:

$$k_\varepsilon(T) = \begin{cases} -2.416 \times 10^{-4} + 0.12 \times 10^{-6}T + 0.4 \times 10^{-8}T^2 & 20^\circ\text{C} < T < 750^\circ\text{C} \\ 110 \times 10^{-4} & 750^\circ\text{C} < T < 860^\circ\text{C} \\ -62.0 \times 10^{-4} + 0.20 \times 10^{-4}T & 860^\circ\text{C} < T < 1,200^\circ\text{C} \end{cases}$$

$$k_\varepsilon(T) = \begin{cases} -2.416 \times 10^{-4} + 0.12 \times 10^{-6}T + 0.4 \times 10^{-8}T^2 & 20^\circ\text{C} < T < 750^\circ\text{C} \\ 110 \times 10^{-4} & 750^\circ\text{C} < T < 860^\circ\text{C} \\ -62.0 \times 10^{-4} + 0.20 \times 10^{-4}T & 860^\circ\text{C} < T < 1,200^\circ\text{C} \end{cases}$$

9.6.2.4 Prestressing steel

The strength and deformation properties of prestressing steel at elevated temperatures shall be defined by an idealised elasto-plastic stress-strain curve (Fig. 85), which is characterised by three salient properties, namely:

- Initial elastic modulus $E_{sp}(T)$,
- Proportional limit $f_{sp,p}(T)$,
- Yield stress $f_{sp,y}(T)$,

- d) Strain $\varepsilon_{sp,y}(T)$ at yield, and
- e) Ultimate strain $\varepsilon_{sp,u}(T)$.
- f) Ultimate strain $\varepsilon_{sp,u}(T)$.

The variation with temperature of these salient properties cold worked (wires & strands) and quenched & tempered (bars) prestressing steel shall be taken as given in Table 90. Linear interpolation may be employed for obtaining the properties at intermediate temperatures.

$$\beta = \begin{cases} \left[\left(\frac{\varepsilon_{uD} - \frac{f_{sp,p}}{E_{sp}}}{\varepsilon_u - \frac{f_{sp,p}}{E_{sp}}} \right) \left(\frac{f_{sp} - f_{sp,p}}{f_{sp}} \right) + \left(\frac{f_{sp,p}}{f_{sp}} \right) \right] & \text{Class A} \\ 0.9 & \text{Class B} \end{cases}$$

$$\beta = \begin{cases} \left[\left(\frac{\varepsilon_{uD} - \frac{f_{sp,p}}{E_{sp}}}{\varepsilon_u - \frac{f_{sp,p}}{E_{sp}}} \right) \left(\frac{f_{sp} - f_{sp,p}}{f_{sp}} \right) + \left(\frac{f_{sp,p}}{f_{sp}} \right) \right] & \text{Class A} \\ 0.9 & \text{Class B} \end{cases}$$

The elongation of prestressing steel at elevated temperatures shall be taken the values given in Table 90 multiplied by strain reduction factor given by:

$$k_\varepsilon(T) = -2.016 \times 10^{-4} + 10.0 \times 10^{-6} + 0.4 \times 10^{-8} \quad \text{for } 20^\circ\text{C} < T < 1,200^\circ\text{C}$$

$$k_\varepsilon(T) = -2.016 \times 10^{-4} + 10.0 \times 10^{-6} + 0.4 \times 10^{-8} \quad \text{for } 20^\circ\text{C} < T < 1,200^\circ\text{C}$$

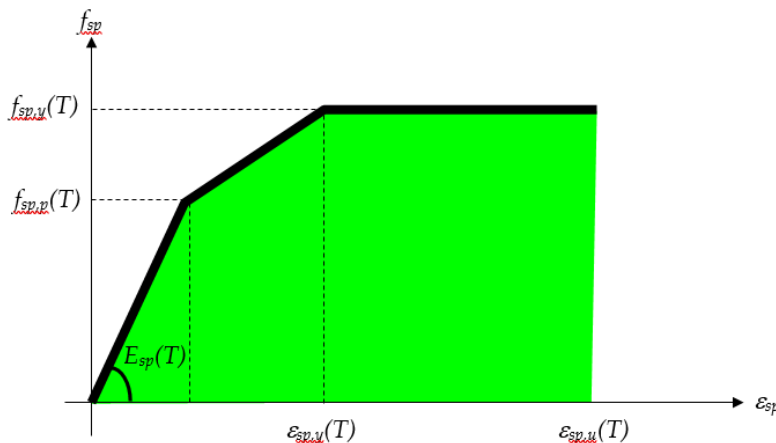


FIG. 85 SALIENT PARAMETERS OF THE STRESS-STRAIN CURVE OF PRESTRESSING STEEL

Table 90 Variation with Temperature of salient properties characterizing the idealised Stress-Strain curve of Prestressing Steel
(Clause 9.6.2.4)

Sl No.	Steel Temperature (°C)	$f_{sp,y}(T)/(\beta f_{yp})$			$f_{sp,p}(T)/(\beta f_{yp})$		$E_{sp}(T)/E_{sp}$		$\varepsilon_{sp,p}$	$\varepsilon_{sp,u}$
		CW		QT	CW	QT	CW	QT	CW & QT	CW & QT
		Class A	Class B							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
i)	20	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.050	0.100
ii)	100	1.00	0.99	0.98	0.68	0.77	0.98	0.76	0.050	0.100
iii)	200	0.87	0.87	0.92	0.51	0.62	0.95	0.61	0.050	0.100
iv)	300	0.70	0.72	0.86	0.32	0.58	0.88	0.52	0.055	0.105
v)	400	0.50	0.46	0.69	0.13	0.52	0.81	0.41	0.060	0.110
vi)	500	0.30	0.22	0.26	0.07	0.14	0.54	0.20	0.065	0.115
vii)	600	0.14	0.10	0.21	0.05	0.11	0.41	0.15	0.070	0.120
viii)	700	0.06	0.08	0.15	0.03	0.09	0.10	0.10	0.075	0.125
ix)	800	0.04	0.05	0.09	0.02	0.06	0.07	0.06	0.080	0.130
x)	900	0.20	0.03	0.04	0.01	0.03	0.03	0.03	0.085	0.135
xi)	1,000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.090	0.140
xii)	1,100	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.095	0.145
xiii)	1,200	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.100	0.150
NOTE										
CW : Cold Worked										
QT : Quenched & Tempered										

9.6.2 Resistance against Blast

A blast imposes two hazards, namely blast loading and missile impact. Protecting the asset from both of these hazards disintegrating and collapsing the structure shall be the central theme of design. But, the provisions of this standard address only the effects of the former and not of the latter. It is the intent of this standard to mitigate the possible damage in a structure, which depends on its proximity to the blast. The following possible effects shall be addressed depending on the proximity of the structure to the blast:

a) *Close proximity:*

- 1) Collapse of a part of the building,
- 2) Progressive collapse of a part of the building, or
- 3) Progressive collapse of the whole building;

b) *Medium proximity:* Failure (that is collapse or deterioration) of:

- 1) Structural members of the exterior bay, like beams, columns, walls and slabs; and
- 2) Non-structural elements, along the façade of the building closest to the

blast; and

c) *Distant proximity:*

Damage to non-structural elements, along the façade of the building closest to the blast (like window breakage, falling of light and other fixtures, and flying debris).

9.6.2.1 *Blast resistant design*

Protection of a concrete structure against blast, shall provide resistance in structural members; and protect non-structural elements. This standard provides specifications related to blast resistant design of structural members only.

In the absence of more rigorous estimates of effects of blast, the following requirements shall be complied with by concrete structures to ensure their blast resistance:

- a) The combined strength capacities of members estimated as per limit state method of design shall be more than the combinations of stress resultants obtained from nonlinear analysis of the structure under the load combinations considered. The minimum design load combinations on concrete structures specified in Table 5 with blast load shall be considered for checking the strength capacity of the structures and its members and the non-structural elements to withstand the effects of blast.
- b) The deformations at critical locations of the structure and in members shall be less than the limits specified in **Table 85.9**.

9.6.2.2 *Blast resistance in structural members*

A blast resistant structure shall possess five features, namely:

- a) Large mass & stiffness,
- b) Large redundancy,
- c) Member strengths proportioned as per ductile design provisions specified in IS 13920,
- d) Resistance to reversed loading through ductile detail specified in IS 13920, and
- e) Strong connections specified in IS 13920.

9.6.3 Resistance against impact

Under the action of impact load specified in IS 875 (Part 5), the damage to concrete structures shall be kept to a minimum so that the functionality of the structure is not jeopardised. The stiffness of the concrete structure is determined by serviceability considerations. Hence, the required toughness of a structure to provide the needed impact resistance shall be determined by tuning the design lateral force and the overall inelastic deformability of the structure.

9.6.3.1 Impact analysis

Structural analysis of a structure subjected to impact loading shall be nonlinear time-history analysis; the time increment of the structural analysis shall be in milli-seconds. The method of analysis shall account for material and geometric nonlinearities. Also, it shall modify dynamically the stiffness and strength characteristics of the material as per the nonlinear stress-strain curve of the constituent materials.

The detailed analysis shall consider the following facets:

- a) Three-dimensional analytical model of the full structure;
- b) Description of impact loading as a function of time specified in IS 875 (Part 5);
- c) At least the minimum combinations of loads specified in Table 5 that include impact load; and
- d) Nonlinear analysis considering both material and geometric nonlinearity.

9.7 Integrity Criteria

Progressive collapse analyses shall be performed to ensure that these two aspects of the structure are provided for, to resist extreme loading arising from blast, impact and earthquake ground shaking:

Redundancy in the structure, that is multiple alternate load paths for transferring loads when unforeseen situations result in the failure of a few members; and
Redistribution Capacity in members, that is inelastic strength and deformation capacity in members along the alternate load paths for accepting the unforeseen loads.

When these analyses are required to be performed, the critical combinations of loads shall be taken as specified in 6.

9.7.1 Available Design Methods

The following three methods given hereunder shall be employed to mitigate damage due to progressive collapse when subjected to fire load, blast load, impact load and earthquake shaking load.

9.7.1.1 Prescriptive method

The design shall be deemed to meet the requirements of integrity criteria due to redundancy and deformability, if the structure complies with the following prescriptive requirements:

- a) Choose a structural system to resist localized loads arising from fire, blast and impact, through provision of alternate load paths;
- b) Select a regular structural system, as defined in IS 1893 (Parts 1 to 11) relevant to the structure being designed;
- c) Design the joints and connections to remain elastic;
- d) Perform linear static analysis of the structure under the action of the loads prescribed as per integrity load combinations specified in Table 6;
- e) Design the Members to meet the requirements of strength, serviceability and durability criteria as prescribed in this standard, and
- f) Detail the members for ductility as per the requirements of IS 13920 (Parts 1 to 11) relevant to the structure being designed.

This method shall be sufficient for low-rise buildings up to 5 storeys of plan area less than 450 m² and normal structures to ensure the desired level of protection.

9.7.1.2 *Key members method*

9.7.1.2.1 The structure shall be designed for the stress-resultants obtained from:

- a) Analysis of the structure under all applicable load combinations specified in Table 6, and
- b) Results of progressive collapse analysis specified in **8.2.3.3(a)**.

9.7.1.2.2 These design strength demands shall be used to design all members and their connections in the structure. When employing progressive collapse analysis, if the applicable loads,

- a) exclude blast load, each of the key elements, which is likely to be lost along the load path during the action of applicable loads, shall be removed one at a time, and the remaining structure examined for safety under the applicable loads; and
- b) include blast load, each combination of the key elements, which are likely to be lost during the action of applicable loads and the blast load, shall be removed one combination at a time, and the remaining structure examined for safety under the applicable loads.

9.7.1.2.3 The design shall be deemed to meet the requirements of:

- a) The integrity criteria due to redundancy, if the key elements are designed and detailed to comply with the requirements of IS 13920 (Parts 1 to 11) and shown

to withstand the design loads without the failure of any other member. The method of analysis here can be linear analysis.

- b) The integrity criteria due to deformability, if all members of the structure are designed and detailed to comply with the requirements of IS 13920 (Parts 1 to 11) and shown to possess large lateral deformability. The method of analysis here shall be nonlinear analysis, including both geometric and material nonlinearities.

This method shall be employed for important and critical & lifeline structures.

9.7.1.2.4 When designing important and critical & lifeline structures for the Integrity Criteria, Progressive Collapse Analysis shall be performed to estimate the design strength and deformation demands on members, especially the key members of the primary (vertical and lateral) load resisting system, to prevent:

- a) Local collapse involving formation of plastic hinges of a few adjoining members resulting in that local portion of the structure to become unstable under the inelastic condition and unrestricted deformation under the applied loads,
- b) Global collapse involving formation of plastic hinges of all members of the structure (in general) resulting in the whole structure to become unstable under the inelastic condition and unrestricted overall deformation under the applied loads.

9.7.1.3 *Nonlinear analysis method*

Nonlinear response analysis of the structure shall be performed to capture the nonlinear dynamic structural response. The locations of blast load and impact load are critical in this analysis.

Based on the nonlinear behaviour of the members, the design shall be considered sufficient to meet the requirements of the:

- a) Integrity criteria due to redundancy, if in the event of the failure of a member, the structure withstands the loads without the failure of any other member, and
- b) Integrity criteria due to deformability, the inelastic rotations at the ends of the members are within 0.004 rad.

This method shall be adopted necessarily for critical & lifeline structures and special to perform more detailed investigations and demonstrate the robustness of the design chosen.

9.8 Restorability Criteria

Structures damaged under the action of the design loads, especially under earthquake ground shaking, shall meet the following requirements:

- a) The damage shall be limited only to the spalling of cover concrete, and in extreme cases, to yielding of longitudinal bars.
- b) The alterations and modifications required to stiffen and/or strengthen the structure shall be such that the locations are accessible and the existing structure does not disintegrate.

10 CONSTRUCTION

While the construction sequence of a project is a detailed process and has many activities performed in parallel, the provisions given hereunder shall be followed with regard to the said activities related to each material.

When new technology materials, product or process is introduced in construction, the same shall be subject to rigorous study through:

- a) detailed experimental work,
- b) appropriate analytical work (if relevant),
- c) rigorous independent third party peer review panel consisting of subject specialists, each with at least 30 years experience in the subject matter in focus. The review panel shall be duly appointed by the owner.
- d) specific QA and QC requirements.

10.1 Storage Of Materials

Stacking and storage of materials (cement, aggregate, steel and other construction materials) at site shall be as per the requirements stated in IS 4082.

10.1.1 Cement

10.1.1.1 Cement shall be protected during transportation and handling to avoid any deterioration or contamination. Cement bags shall be stored on site in perfectly dry and waterproof sheds, and on platforms that are raised about 0.2 m above ground level, and about 0.6 m clear off the walls in a manner to avoid deterioration or contamination. The bags shall be stacked with not more than 8 bags along the height, and in such a manner as to facilitate their removal and use in the order in which the batches of cement are received.

10.1.1.2 When cement is stored in bulk storage containers (like silos), the following shall be complied with:

- a) It should be waterproof and uncracked.
- b) The capacity of the containers shall be sufficient to cater to the site requirements.
- c) The containers shall be cleaned at least once in 3 months.
- d) The cement shall be used in the sequence in which it is received at site (assuming that their manufacture is in the same sequence).
- e) Each consignment shall be stored separately to readily identify and inspect, when required.
- f) Any consignment of cement (or a part of it) that has deteriorated in any way during storage shall not be used in works, and shall be removed immediately from site.
- g) Cement shall be used within 3 months or date of expiry specified by the manufacturer, whichever is earlier.

10.1.2 Aggregates

Aggregates (fine and coarse) shall be stacked separately on hard well-drained surfaces or platforms, such that the stockpiles are sufficiently away from each other. When aggregates are placed directly on hard ground, only that material which is 0.3m above ground level shall be initially taken out and used. The remaining material shall be used only after it has been completely cleaned.

10.1.3 Reinforcing Steel

Reinforcement bars shall be:

- a) Stored in dry covered sheds and on platforms, skids or other supports, which are raised about 0.15 m above ground level, and about 0.3 m clear off the walls;
- b) Stored separately according to their classification, size and lengths; and
- c) Given a coat of protective cement wash, preferably at all sites, but necessarily in coastal areas or when stored for long.

10.1.4 Prestressing Material

10.1.4.1 All prestressing material (for example steel tendons, sheathing, anchorages, sleeves or couplers) shall be protected during transportation, handling and storage to avoid any damage or deterioration.

10.1.4.2 Prestressing steel shall be absolutely clean and without any signs of rust. It shall be stored:

- a) In dry covered sheds with instruments capable of measuring air humidity;
- b) On platforms, skids or other supports, raised about 0.15 m above ground level, and about 0.3 m clear off the walls;
- c) Wrapped with protective covering of tar paper or polythene or any other approved material; and
- d) Separately according to their classification, size and lengths.

10.1.4.3 Tendons shall

- a) be protected from splashes from any other material, cutting operation of oxy-acetylene torch or arc welding processes in the vicinity, or on-site heat treatment or welding operation, or metallic coating such as galvanising;
- b) not be kinked or notched during transportation and storage;
- c) be stored in large diameter coils, which enable tendons to be laid out straight. As a guide wires above 5mm diameter, coils of about 3m diameter shall be obtained from manufacturer;
- d) be wrapped with protective sheeting that is chemically neutral; and
- e) be provided with temporary protection during storage.

10.1.5 Anchorage Components

All components shall be:

- a) Handled and stored to prevent mechanical damage and detrimental corrosion.
- b) Prevented from any type of corrosion to gripping and securing systems.
- c) Used with such formulated oils and greases, which are guaranteed by the manufacturer to be non-aggressive and non-degrading.

10.1.6 Environmental Considerations at the Site

Suitable measures, including the following, shall be employed:

- a) Store cement and other fine powders in closed rooms and bins, and carry them in closed containers during transportation.
- b) Sprinkle water on fine aggregates.
- c) Batch and mix concrete in closed containers.
- d) Regularly monitor the quality of run-off water from the batching plants, mixing chambers and construction sites.

10.2 Concrete

Concrete shall be handled, held in position, placed, finished and cured as per provisions given hereunder.

10.2.1 Formwork

Falsework, in the widest meaning is the total system of support for freshly placed concrete including the term formwork, mould or shuttering which contacts the concrete as well as supporting members, hardware and necessary bracing, etc

10.2.1.1 General

The falsework shall be designed and constructed so as to remain sufficiently rigid during placing and compaction of concrete, and shall be such as to prevent loss of slurry from the concrete. For further details regarding design, detailing, etc. reference shall be made to IS 14687. The tolerances on the shapes, lines and dimensions shown in the drawing shall be within the limits given below:

Sl No.	Component of Concrete Structure	Level of Quality Control (in mm)		
		QA3	QA2	QA1
(1)	(2)	(3)	(4)	(5)
i)	Wall			
	a) Verticality up to 2m	± 4	± 10	± 15
	b) Verticality up to 3m	± 6	± 12	± 20
	c) Verticality up to 7m	± 8	± 15	± 25
	d) Verticality up to 10m	± 10	± 20	± 25
	e) Straightness in 5m length	± 3	± 6	± 10
ii)	Column			
	a) Verticality up to 3m	± 5	± 10	± 15
	b) Verticality up to 7m	± 7	± 12	± 20
	c) Verticality up to 20m	± 10	± 20	± 25
	d) Cross section up to 300 mm	± 2	± 3	± 5
	e) Cross section 300 to 600 mm	± 3	± 5	± 10
	f) Cross section 600 to 1.5 m	± 4	± 10	± 15
	g) Squareness	3	6	10
iii)	Beam			
	a) Depth up to 300 mm	± 3	± 3	± 8
	b) Depth up to 600 mm	± 4	± 8	± 15
	c) Depth over 600 mm	± 5	± 10	± 20
	d) Variation in datum	± 5	± 10	± 10
	e) Straightness in 5m length	± 3	± 8	± 12
iv)	Soffits			
	a) Flatness on 2.5m grid	± 9	± 15	± 20
	b) Variation in datum	± 8	± 10	± 15
v)	Footings			
		Deviation from dimensions		
	a) Plan Dimensions	+50 mm and -10 mm		
	b) Eccentricity	0.02 times the width of the footing in the direction of deviation, but not more than 50 mm		
	c) Thickness	+50 mm and -10 mm, or ± 0.05 times the specified thickness, whichever is less		

Variations shall be measured from 0 level. These tolerances apply only to external dimensions of concrete members and not to the positioning of reinforcing steel within them or dowels protruding from them.

10.2.1.2 Formwork Systems

Standard/patented formwork systems using different materials like plywood, steel, aluminium and plastic etc, conforming to the requirements given in IS 14687 and this standard may be specified/used for normal or specialized construction requirements.

Where the shape of the element is such that the formwork has re-entrant angles, the formwork shall be removed as soon as possible after the concrete has set, to avoid shrinkage cracking occurring due to the restraint imposed.

10.2.1.3 Cleaning and Treatment of Formwork

All rubbish, particularly, chippings, shavings and sawdust shall be removed from the interior of the forms before the concrete is placed. The face of formwork in contact with the concrete shall be cleaned and treated with form release agent. Release agents shall be applied so as to provide a thin uniform coating to the forms without coating the reinforcement.

10.2.1.4 In prestressed concrete structures, moulds for pre-tension members shall be sufficiently strong and stiff to withstand (without distortion) the effects of placing and compacting concrete as well as those of prestressing in the case of manufacture by the individual mould process, where the prestressing tendon is supported by the mould before transfer.

10.2.1.5 While using self compacting concrete, extra care need to be taken in formwork design as lateral pressure of fresh concrete on formwork is more in case of SCC as compared to normal concrete.

10.2.1.6 Stripping time

Forms shall not be released until the concrete has achieved strength of at least twice the stress to which the concrete may be subjected at the time of removal of formwork. The strength referred to shall be that of concrete using the same cement and aggregates and admixture, if any, with the same proportions and cured under conditions of temperature and moisture similar to those existing on the work. However, minimum strength of concrete at the time of removal of formwork shall be as given below:

		<i>Minimum strength required</i>

<i>Sl No.</i>	<i>Age of concrete cube at the time of testing (in days)</i>	<i>Up to M40 (percent of specified strength)</i>	<i>More than M40 (N/mm²)</i>
(1)	(2)	(3)	(4)
i)	3	45	18
ii)	7	60	24
iii)	14	85	34

10.2.1.6.1 While the above criteria of strength shall be the guiding factor for removal of formwork, in normal circumstances, where ambient temperature does not fall below 15°C and adequate curing is done, the striking period specified below may deem to satisfy the guideline given in **92.1.6**.

<i>Sl No.</i>	<i>Type of Formwork</i>	<i>Minimum Period before striking Formwork</i>	
		<i>OPC</i>	<i>OPC with cementitious materials like fly ash & slag</i>
(1)	(2)	(3)	(4)
i)	Vertical formwork to columns, walls and beams	*16-24 hours	*24 - 48 hours
ii)	Soffit formwork to slabs (props to be re-fixed immediately after removal of formwork)	3 days	7 days
iii)	Soffit formwork to beams (props to be re-fixed immediately after removal of formwork)	7 days	10 days
iv)	Props to slabs: (a) Spanning up to 4.5 m (b) Spanning over 4.5 m	7 days 14 days	10 days 14 days
v)	Props to beams & arches: (a) Spanning up to 6 m (b) Spanning over 6 m	14 days 21 days	14 days 21 days
*wherever retarders are used, the stripping time may be increased appropriately. In case high grades of concrete (M40 and above) are used, the stripping times may be reduced appropriately subject to meeting minimum strength requirements at the time of stripping.			

10.2.1.6.2 The number of props left under, their sizes and disposition shall be such as to safely carry the full dead load of the slab, beam or arch, as the case may be, together with any imposed load likely to occur during curing or further construction.

10.2.1.6.3 Where the shape of the element is such that the formwork has re-entrant angles, the formwork shall be removed as soon as possible after the concrete has set, to avoid shrinkage cracking occurring due to the restraint imposed.

After stripping, utmost care shall be taken to provide props. These props shall be provided immediately after stripping each shuttering panel and not after stripping all the panels of the entire slab.

10.2.2 *Production of Concrete*

The provisions hereunder shall be complied with during batching, mixing and measuring of concrete.

10.2.2.1 *Batching*

In batching concrete, the quantity of all the ingredients of concrete shall be determined only by weigh-batching.

10.2.2.1.1 Volume batching of cement and aggregates may be allowed for PCC below foundations and other non-structural concrete works, only where weigh-batching is not practical and provided accurate bulk densities of materials have been pre-established to be used in concrete. For volume batching accurately measurable wooden/metal/plastic boxes will only be used. But, in such cases, allowance for bulking shall be made in accordance with IS 2386 (Part 3). The mass volume relationship should be checked as frequently as possible (preferably everyday), the frequency for the given job being determined by engineer-in-charge to ensure that the specified grading is maintained.

10.2.2.1.2 For large and medium project sites, concrete shall be sourced from ready-mixed concrete plants or captive on-site or off-site automatic batching & mixing plants. The quality control aspects of concrete produced and supplied by either of these plants shall be in accordance with IS 4926. Further:

- a) The practical number of different concrete mixes shall be kept small on any site or in any one plant.
- b) Grading of aggregate should be controlled by obtaining coarse aggregate in different sizes and blending them in the right proportions. Aggregates of different sizes shall be stocked in separate stock-piles. Material shall be stock-piled for several hours of production, preferably a day before use. Grading of coarse and fine aggregate shall be checked as frequently as possible, the frequency for a project being determined by the engineer-in-charge to ensure that the specified grading is maintained.
- c) The accuracy of the measuring devices shall fall within the following limits of the quantity of material measured in each batch:
 - 1) Cement : ± 1 percent

- 2) Water : ± 1 percent
- 3) Aggregate : ± 2 percent
- 4) Admixture : ± 3 percent
- d) In a batching plant, the concrete producing equipment shall be calibrated initially at the time of installation or re-conditioning of the equipment, and subsequently at the following intervals:
 - 1) Mechanical/knife edge systems: At least once every 2 months
 - 2) Electrical/load cell systems: At least once every 3 months
 - 3) Probes for moisture content: At least once every 2 months
- e) Proportion/Type and grading of aggregates shall be made by trial in such a way so as to obtain densest possible concrete.
- f) Water-cement ratio specified shall be maintained constant at its design value. To this end, moisture content in both fine and coarse aggregates shall be determined as frequently as possible, the frequency for a project being determined by the engineer-in-charge according to weather conditions. The amount of the added water shall be adjusted to compensate for any observed variations in the moisture contents. Moisture content in aggregates shall be determined as per IS 2386 (Part 3). Also, suitable adjustments shall be made in the masses of aggregates to allow for variation due to moisture content. In the absence of exact data, only in the case of nominal mixes, the amount of surface water may be estimated from the values given in Table 91.
- g) The mechanical mixer shall be fitted with a water measuring (metering) device.
- h) No substitutions in materials used on the project or alterations made in the established proportions shall be made without additional tests to show that the quality and strength of concrete are satisfactory except as permitted in 10.2.2.1.1 and 10.2.2.1.2(f).

Table 91 Surface Water Carried by Aggregate
[Clause 10.2.2.1.2(f)]

Sl No.	Aggregate	Approximate Quantity of Surface Water	
		percentage by mass	Liters/m ³
(1)	(2)	(3)	(4)
i)	Wet Sand	7.5	120
ii)	Moderately Wet Sand	5.0	80
iii)	Moist Sand	2.5	40
iv)	Moist Gravel or Crushed Rock	2.5 – 1.25	40 – 20

10.2.2.2 Mixing

Concrete shall be mixed only by mechanical methods (and not by manual mixing), either in a mechanical mixer or in a batching and mixing plant complying with IS 1791.

It shall be ensured that stationary, central and truck mixers comply with the performance criteria of mixing efficiency laid down in IS 12119. Mixing efficiency test shall be performed at least once in a year for each mixer. The mixing shall be continued until there is a uniform distribution of the materials, the entire mass of concrete is uniform in colour and consistency; and each individual coarse aggregate is coated completely with mortar containing its proportionate amount of cement. If there is segregation after unloading from the mixer, the concrete should be remixed immediately in case there is sufficient time to allow for placing and compacting before the initial set time, else it shall be discarded. Further:

- a) The mixing time shall be at least 2 min for conventional free fall (drum) batch type concrete mixers. For more efficient mixers, manufacturers' recommendations shall be followed.
- b) Workability should be checked at frequent intervals (see IS 1199).
- c) Dosages of retarders, plasticizers and superplasticizers shall be arrived based on requirement of workability, setting time and early strength. The maximum limit shall be as per manufacturer's recommendation. Dosages of retarders, plasticizers and superplasticizers shall be restricted to 0.5, 1.0 and 2.0 percent respectively by mass of cementitious materials, unless a higher value is agreed upon between the manufacturer and the constructor based on performance test.
- d) The mixer and plant shall be approved by engineer-in-charge. The mixer or plant shall be at a location approved by the engineer-in-charge, considering the properties of mixes and the transportation arrangements available with the contractor.
- e) Mixers that have been out of use for more than 30 minutes shall be thoroughly cleaned before putting in a new batch.
- f) Mixing plant shall be thoroughly cleaned before changing from one type of cement to other.

10.2.2.3 *Transporting and Handling*

After mixing, concrete shall be transported to the formwork as rapidly as possible by methods which will prevent the segregation or loss of any of the ingredients or ingress of foreign matter or water and maintaining the required workability. In general, concrete shall be discharged from the truck-mixer within 2 h of the time of loading. Longer duration of discharge of concrete may be permitted as per agreement between the supplier and purchaser, if retarding admixtures are used or in cool humid weather or when chilled concrete is produced. The time of loading shall be measured from mixing water to dry mix of cement & aggregate, or from adding cement to wet aggregate, whichever is applicable.

10.2.2.3.1 During hot or cold weather, concrete shall be transported in deep containers. Other suitable methods to reduce the loss of water by evaporation in hot weather and heat loss in cold weather may also be adopted.

10.2.2.3.2 When ready mix concrete is used, an additional dose of admixtures may be added to the concrete, such that total dosage of the admixture does not exceed the maximum recommended for that admixture; and sufficient trials have been carried out to ensure the performance of the concrete after such redosing. Also, sufficient number of rotations of the transit mixer must be carried out at mixing RPM so as to ensure a homogenous mixing of the redosing admixture.

10.2.2.3.3 Unless otherwise specified, the equipment used for transportation and compaction of concrete shall include:

- a) Concrete dumper of at least 2 tonne capacity,
- b) Powered hoist of at least 0.5 tonne capacity,
- c) Chutes,
- d) Buckets handled by cranes,
- e) Transit truck mixer,
- f) Concrete pumps,
- g) Concrete distributor booms,
- h) Belt conveyors,
- i) Crane with skips, and
- j) Tremies.

10.2.2.4 *Placing*

Concrete shall be deposited as close as practicable to the member being concreted to avoid rehandling. The concrete shall be placed and compacted before initial setting of concrete commences, and should not be subsequently disturbed. Methods of placing shall be such as to preclude segregation. Care should be taken to avoid displacement of reinforcement or movement of formwork. In general, concrete shall be placed gently, and the maximum permissible free fall of concrete shall be limited to 1.5 m.

10.2.2.5 *Compacting*

Concrete shall be thoroughly compacted, and fully worked around the reinforcement, around embedded fixtures and into corners of the formwork. The equipment used for compaction shall be compatible with the workability of concrete and type of concrete element.

10.2.2.5.1 Concrete shall be compacted using mechanical vibrators complying with IS 2505, IS 2506, IS 2514 and IS 4656. Over-vibration and under-vibration of concrete are harmful and shall be avoided. Also, vibration of very wet mixes shall be avoided. Whenever vibration has to be applied externally, the design of formwork and disposition of vibrators shall receive special consideration to ensure efficient compaction and to avoid surface blemishes.

10.2.2.5.2 The concrete used shall have adequate workability that can be properly compacted by means of internal vibration. Adequate compaction shall be ensured by providing suitable workability and by employing appropriate placing and compacting equipment and procedures. Full compaction shall be ensured in the vicinity of: (a) construction and movement joints, (b) embedded water bars and reinforcement, (c) tandems or duct formers, (d) embedded fixtures, and (e) corners of formwork, to produce a dense homogeneous void-free mass having the required surface finish.

10.2.2.5.3 When vibrators are used, vibration shall be done continuously during the placing of each batch of concrete until the expulsion of air has practically ceased and in a manner that does not promote segregation.

10.2.2.5.4 When external vibrators are used, the design of formwork and access of vibrator shall be such as not to be applied through reinforcement and all inserts (like ducts). Internal vibrators shall be inserted in an orderly manner and the distance between insertions vibrators shall be about one and half times the radius of the area visibly affected by vibration. Additional vibrators in serviceable condition shall be kept at site so that they can be used immediately in the event of break-down.

10.2.2.6 *Finishing*

After compaction, the operations of screeding, floating and first troweling shall be performed to obtain a flat and durable surface. The process of finishing operation from floating to troweling shall be delayed as long as possible, while still working toward the desired grade and surface smoothness. Over-manipulation of the concrete shall be avoided to control the deposition of excessive fines and water at the surface.

During or after finishing operations, free water will not accumulate at the surface of concrete, if proper consistency and design mix proportions are used. If it does, the same shall be removed by blotting with absorbing mats, draining or pulling off with a loop of hose so that the surface loses its water sheen before the next finishing operation is performed. Under no circumstance, any finishing tool shall be used in an area with accumulated water, before removing the same. Vacuum dewatering can be used where required. Surface shall not be overworked on. Also, addition of water or cement to aid in finishing shall be avoided.

10.2.2.7 *Curing*

The curing regime shall ensure suitable temperature and humidity in the concrete for an adequate duration that allows the development of desired properties.

Concreting operations shall commence only after adequate arrangements for curing had been made. Usually, active curing measures are taken after the setting of the concrete. However, in hot and/or dry/windy conditions, early measures may be required to prevent the evaporation of water before setting.

10.2.2.7.1 *Normal curing*

Curing shall be performed either by moist curing or membrane curing, in both normal and high strength concrete.

10.2.2.7.2 *Moist curing*

Exposed surfaces of concrete shall be kept continuously in a damp or wet condition by ponding or by covering with a layer of sacking, canvas, hessian or similar materials, and kept constantly wet for at least:

- a) 7 days from the date of placing concrete, when ordinary Portland cement is used,
- b) 10 days from the date of placing concrete when exposed to dry and hot weather conditions, and
- c) 14 days from the date of placing, when cement with mineral admixtures or blended cements are used.

10.2.2.7.3 *Membrane curing*

Approved curing compounds shall be used in lieu of moist curing with the permission of the engineer-in-charge. Efficacy of the curing compounds to be used shall be ascertained at the mix design stage. Such compounds shall be applied to all exposed surfaces of concrete as soon as possible after the concrete has set. Impermeable membrane such as polyethene sheeting covering closely the concrete surface may also be used to provide effective barriers against evaporation. When adopting membrane curing for the concretes containing Portland pozzolana cement, Portland slag cement or cement with mineral admixture, the duration of curing shall be increased.

10.2.2.7.4 *Steam Curing*

It shall be ensured that steam curing is done in a suitable enclosure to contain the live steam to minimize moisture and heat losses and applied at 100 percent relative humidity. Steam shall not be applied directly on the concrete surface. The following schedule shall be adopted for steam curing:

- a) Steam curing shall be started only after about 4 hours of placement of concrete to allow the initial setting of concrete. Where retarders are used, this waiting period shall be increased to about 6 hours.
- b) The rate of increase of temperature of ambient air inside the curing chamber shall not exceed 5°C per hour.
- c) The maximum temperature applied shall be 60°C to 70°C.

- d) The maximum temperature shall be maintained until concrete has reached the desired strength.
- e) When steam curing is being discontinued, the rate of decrease of temperature of ambient air inside the curing chamber shall not exceed 5°C per hour.
- f) The temperature of ambient air inside the curing chamber shall be lowered to about 10°C above the temperature of the air to which the concrete will be exposed, and kept steady for ____ **hours** and discontinued thereafter.
- g) Concrete shall not be exposed to temperatures below 0°C for at least 6 days after curing.

10.2.2.8 Supervising

It is exceedingly difficult and costly to redo concrete once placed. Hence, constant and strict supervision shall be ensured of:

- a) All members being concreted during the progress of the construction work, including the proportioning and mixing of the concrete, and
- b) Reinforcement and its placing before concreting.

Before any important operation, such as concreting or stripping off the formwork is started, adequate notice shall be given to the authorized construction supervisor.

10.3 Concreting Under Special Conditions

The provisions given hereunder shall be complied with when concreting under hot weather, cold weather, under water, mass concrete and aggressive soil conditions.

10.3.1 Hot Weather Concreting

Concreting in hot weather conditions, that is when ambient air temperature is above 40°C or when the temperature of concrete at time of its placement is expected to be beyond 40°C other than during steam curing, shall be done as specified hereunder. The provisions hereunder shall be applicable to minimize the harmful effects of hot weather on concreting and concrete.

- a) *Temperature Control of Ingredients of Concrete:*

All available means shall be used for maintaining the ingredient materials of concrete at as low temperatures as practicable.

- 1) *Aggregates*

Any one of the following procedures or a combination of the same may be used for lowering the temperature or preventing excessive heating of coarse aggregates:

- i) Shading stockpiles from direct rays of the sun.
- ii) Sprinkling with water the stockpiles of coarse aggregate and keeping them moist. When coarse aggregates are stockpiled, successive layers should be sprinkled as the stockpile is built up. This procedure results in cooling by evaporation, and it is effective especially when relative humidity is low. Such sprinkling should be done uniformly, because it leads to uniformity in surface moisture and thereby gives uniform workability.
- iii) Heavy spraying of cold water (if available) on coarse aggregate immediately before use may be done to cool them directly. Alternately, they can be inundated in cold water.
- iv) Circulating refrigerated air through pipes in the stock piles of coarse aggregates.

2) *Water* – Any one of the following procedures or a combination of the same may be used for lowering the temperature of water:

- i) Use cold water; keep it cold by insulating the pipes, water storage tanks, etc.
- ii) Insulate tanks or trucks used for transporting water, or colour them white or yellow.
- iii) Use mechanical refrigerator to cool water.
- iv) Use crushed ice (in place of water) when mixing concrete.

Under certain circumstances, reduction in water temperature may be most economically accomplished by methods (iii) and (iv) above. But, on the contrary, in method (iv) above, if all ice does not melt completely by the time mixing is completed, and if a portion of ice melts after consolidation of concrete, hollow pockets are left in concrete, which has detrimental effects.

3) *Cement* – Any one of the following procedures or a combination of the same may be used for lowering the temperature of cement:

- i) Storing cement in shaded areas.
- ii) Ensuring minimal heat gain into the room in which the cement bags are placed, during the day from direct sunlight on its walls and roof.

b) *Proportioning of concrete mix materials and concrete mix design*

Concrete mix shall be designed with:

- 1) Minimum cement content consistent with other functional requirements; and
- 2) Cements having low heat of hydration, instead of those having high fineness and high heat of hydration.

If the temperature is such as to increase mixing water demand or reduce workability, then water reducing and set-retarding admixtures may be used to offset the accelerating effects of high temperatures and high mix water. Any such admixture shall be used only on the recommendation of competent technical advice, or on the basis of advance testing (when practicable) with the cement and other materials involved.

c) *Temperature of Concrete as placed*

The temperature of concrete at the time of placing shall be maintained below 40°C. Towards this end, the temperature T (°C) of the freshly mixed concrete at the time of leaving the mixer or batching plant shall be estimated (using the temperature of all its constituents) as:

- 1) Cold water as mix water (without ice)

$$T = \frac{S(T_a W_a + T_c W_c) + T_w W_w + T_{wa} W_{wa}}{S(W_a + W_c) + W_w + W_{wa}}; \text{ and}$$

- 2) Ice added to the mix water

$$T = \frac{S(T_a W_a + T_c W_c)}{S(W_a + W_c) + W_w + W_{wa} + W_i} + \frac{(W_w - W_{wa})T_w + W_{wa}T_{wa} - 79.6W_i}{S(W_a + W_c) + W_w + W_{wa} + W_i},$$

where T_a , T_c , T_w and T_{wa} are temperatures (°C) of aggregate, cement, mix water and free water on aggregate, respectively, W_a , W_c , W_w , W_{wa} and W_i are masses (kg) of aggregate, cement, mix water, free water on aggregate, and ice, respectively; and S , the ratio of specific heat of cement & aggregate to that of water. In practice, S shall be taken as 0.22 and $T_{wa} = T_a$.

The increase in temperature may be taken as 2°C during transportation, in the absence of specific information.

d) *Production and Delivery*

Temperatures of aggregate, water and cement shall be maintained at the lowest practical levels so that the temperature of the concrete is below 40°C at the time of placement. Mixing time shall be kept to a minimum, to ensure adequate quality and uniformity, because the concrete is warmed from the work of

mixing, from the air and from the sun. Attention shall be given to coordinating the delivery of concrete with the rate of placement to avoid delays in delivery.

e) Placement, Protection and Curing

1) Placement and Finishing

Forms, reinforcement, and sub-grade shall be sprinkled with cool water just prior to placement of concrete. The area around the work shall be kept wet to the extent possible to cool the surrounding air and increase its humidity, thereby reducing temperature rise and evaporation from the concrete. When temperature conditions are critical, concrete placement may be restricted to the evenings or night when temperature are low and evaporation is less.

Ample personnel shall be employed to handle and place concrete immediately on delivery. On flat work, all steps in finishing shall be carried out promptly.

Concrete shall be placed in layers thin enough and in area small enough so that the time interval between consecutive placements is reduced and vibration or other working of the concrete will ensure complete union of adjacent portions. If cold joints tend to form or if surfaces set and dry too rapidly, or if plastic shrinkage cracks tend to appear, the concrete shall be kept moist by means of fog sprays, wet burlap, cotton mats, or other means.

All placement procedures shall be directed to keep the concrete as cool as practicable and to ensure its setting and hardening, under temperature conditions which are reasonably uniform and, under moisture conditions, which will minimize drying. Concrete, whether delivered by a truck or otherwise, shall reach the forms at a temperature not higher than 40°C, and whatever is practicable shall be done to minimize temperature increase during placing, consolidation, finishing, and curing operations.

2) Protection and Curing

Attention shall be paid to ensure that all surfaces are prevented from drying. Immediately after consolidation and surface finish, concrete shall be protected from evaporation of moisture, without letting ingress or external water, by means of wet (not dripping) gunny bags, Hessian cloth, *etc.* Moist curing shall commence once the concrete has attained some degree of hardening sufficient to withstand surface damage (approximately 12 hours after mixing). The duration of curing shall not be less than 10 days.

Reliance shall not be placed on the protection afforded by forms for curing in hot weather. If possible, water shall be applied to formed surface while forms are still in place and unformed surfaces shall be kept moist by wet curing. The covering material shall be kept soaked by

spraying. Steeply sloping and vertical formed surfaces shall be kept completely and continuously moist prior to and during form removal by applying water to top surfaces so that it will pass down between the form and the concrete.

On exposed unformed concrete surfaces, wind breakers shall be provided as far as possible.

On hardened concrete (especially on flat surfaces), curing, water shall not be much cooler than the concrete, because of the possibilities of thermal stresses and resultant cracking. At the termination of curing with water, effort shall be made to reduce the rate of drying by avoiding air circulation.

f) Inspection and Temperature Records

Competent inspection personnel shall be available to anticipate the need for:

- 1) Requirements during hot weather concreting (such as spraying of forms and sub-grade),
- 2) Ice as a portion of the mixing water, providing shades, wind screens, *etc.*, and
- 3) Minimizing delays in placement and curing.

The supervisor shall record at frequent intervals air temperature, general weather condition (calm, windy, clear & cloudy), and relative humidity. The record shall include information on frequent checks on temperature of concrete as delivered and after placing in the forms, and the protection and curing time as provided.

All such data shall be gathered with the work in progress such that the conditions surrounding the construction of any part of the structure can be determined (if necessary) at a later date. A copy of all the observations shall be included in the permanent records of the job.

g) Concrete Testing

Extra care shall be exercised to maintain temperature and moisture conditions for strength test specimens as required in standard test methods. Proper temperature shall be maintained by avoiding exposure to the sun and by utilizing the cooling effect of evaporating water, either from damp burlap or wet sand covering the specimen. Specimens used as the basis for acceptance of concrete as delivered to the structure shall be transferred at the age of one day to a location (usually the laboratory) where they will receive continuous standard moist curing until test age of specimen. Specimens cured on the job shall never be substituted for laboratory cured specimens as a check of the

proportioning and mixing of the concrete. Moulded specimens used for these purposes shall be cured at the same place and as nearly as possible under the same conditions as those around the structure.

10.3.2 Cold Weather Concreting

Concreting in cold weather conditions, that is when ambient air temperature is around 5°C or less, shall be done as specified hereunder. The provisions hereunder shall be applicable to minimize the harmful effects of cold weather on concrete and concreting. The degree to which these provisions are applicable depends on circumstances and shall be determined appropriately. For this purpose, cold weather shall be classified into three categories, namely:

- a) When the temperature is below 5°C, but does not fall below freezing point,
- b) When frost occurs at night only and is not very severe, and
- c) When there is severe frost day and night.

Table 92 summarizes the precautions to be taken in each case.

Table 92 Precautionary Measures to be taken during Cold Weather Concreting
(Clause 10.3.2)

Sl No.	Cold Weather Category	Mitigation Measures
(1)	(2)	(3)
i)	When the temperature is below 5°C, but does not fall below freezing point	(1) Keep formwork in position for longer duration, or use rapid hardening cement; (2) Cover the top of the concrete with insulating material; (3) Insulate steel formwork; and (4) Make sure that concrete is delivered to the point of placing at not less than 5°C.
ii)	When frost occurs at night only and is not very severe	(1) Keep formwork in position for longer duration, or use rapid hardening cement; (2) Cover the top of the concrete with insulating material; (3) Insulate all formwork; (4) Make sure that concrete is delivered to the point of placing at not less than 5°C; (5) Make sure that concrete is not placed against a frozen sub-grade or against reinforcement or forms covered with snow or ice; and

- (6) Place concrete quickly and insulate.
- iii) When there is severe frost day and night
- (1) Keep formwork in position for longer duration, or use rapid hardening cement;
 - (2) Cover the top of the concrete with insulating material;
 - (3) Insulate all formwork;
 - (4) Ensure that concrete is delivered to the point of placing at not less than 5°C;
 - (5) Ensure that concrete is not placed against a frozen sub-grade or against reinforcement or forms covered with snow or ice;
 - (6) Place concrete quickly and insulate;
 - (7) Heat the water, and, the aggregate, if necessary; and
 - (8) Ensure that concrete is delivered to the point of placing at not less than 10°C, place quickly and insulate; or make sure concrete is delivered to the point of placing at not less than 5°C, place quickly and provide continuous heating to the concrete.

a) *Temperature Control of Ingredients of Concrete*

All available means shall be used for maintaining the ingredient materials of concrete at as high temperatures as practicable.

- 1) *Aggregates* – Heating of aggregate shall be such that frozen lumps, ice and snow are eliminated and at the same time over-heating is avoided. At no point shall the aggregate temperature exceed 100°C; the average temperature of aggregate for an individual batch shall not exceed 65°C.
- 2) *Water* – Mixing water shall be heated under such a control and in sufficient quantity as to avoid appreciable fluctuations in temperature from batch to batch.
- 3) *Cement* – It is preferable to use rapid hardening portland cement conforming to IS 8041.

b) *Proportioning of Concrete Mix Materials and Concrete Mix Design*

The cement content in the mix shall be not less than 300 kg/m³. Air-entraining admixtures and accelerators, when proposed to, shall conform to the requirements of IS 9103, and shall be used only after proper evaluation with the cement and aggregates proposed to be used in the works. If a combination of

different admixture is used, preliminary tests shall be carried out with such combination prior to use in the actual construction.

c) Accelerating admixtures

Accelerating admixtures when used in reinforced concrete should be chloride free admixtures. Calcium chloride may be used as an accelerating admixture, only in case of plain concrete not containing any metal reinforcement. Admixtures containing very small amount of chlorides may be used subject to the condition that total calcium chloride in concrete from all ingredients together shall be limited to the value specified in this standard.

d) Air-Entraining Agents

The compressive strength at 28 days of air-entrained concrete shall not be less than 90 percent of a reference mix without air entrainment. Air-entraining agents shall be permitted in prestressed concrete, only if it does not contain chlorides.

e) Temperature of Concrete as placed

The temperature of concrete at the time of placing shall be maintained above 5°C. Towards this end, the temperature T (°C) of the freshly mixed concrete at the time of leaving the mixer or batching plant shall be estimated (using the temperature of all its constituents) as:

$$T = \frac{S(T_a W_a + T_e W_e) + T_w W_w + T_{wa} W_{wa}}{S(W_a + W_e) + W_w + W_{wa}},$$

where T_a , T_e , T_w and T_{wa} are temperatures (°C) of aggregate, cement, mix water and free water on aggregate, respectively, W_a , W_e , W_w and W_{wa} are masses (kg) of aggregate, cement, mix water, free water on aggregate, and ice, respectively; and S , the ratio of specific heat of cement & aggregate to that of water. In practice, S shall be taken as 0.22 and $T_{wa} = T_a$.

The drop in temperature may be taken as 2°C during transportation, in the absence of specific information.

f) Production and Delivery

Preparation for concreting in cold weather shall be completed well in advance of setting in of severe conditions. Wind breakers shall be erected to shield the mixing and batching plants. Also, steam generating plant or other necessary equipment

shall be installed and checked for correct functioning. Further, suitable protecting clothing shall be available to the site staff. The operators shall be instructed on the procedure for concreting during cold weather. The concrete shall be delivered to the point of placing at not less than 5°C. It is necessary to place the concrete quickly and cover the top or the concrete with an insulating material.

g) Placement

1) Placement and Finishing

Before any concrete is placed, all ice, snow and frost shall be removed completely and the temperature of all surfaces to be in contact with new concrete shall be raised as close to the temperature of fresh concrete to be placed thereon as practicable. No concrete shall be placed on a frozen sub-grade or on one that contains frozen materials.

Arrangements for covering and insulating of newly placed concrete shall be made in advance of placement and shall be adequate to maintain in all parts of concrete, the temperature and moisture conditions that are referred to herein for winter curing temperature and methods.

2) Protection

During cold weather, all concrete surfaces shall be covered as soon as the concrete has been placed to keep the heat in and to help prevent freezing.

3) Curing

For concrete cast in insulated formwork, the member shall be covered completely to retain sufficient water for the hydration of the cement. On removal of the formwork and insulation, the member shall be immediately covered with plastic sheet or tarpaulins, properly lapped and made wind-tight. On no account shall such concrete, just released from insulated formwork, be saturated with cold water. When protective measures are to be discontinued, the surface temperature or the concrete shall be adjusted gradually to the air temperature.

4) Removal of Forms

The time limit indicated for removal of formwork to the sides of beams, columns and wall shall, in no case, be less than the pre-hardening period as given in Table 93.

Table 93 Minimum Pre-hardening Periods
[Clause 10.3.2(g)(4)]

Sl No.	Grade of Concrete	Requisite Pre-Hardening Period (in hours) at stated curing temperature of concrete in house			
		20°C	15°C	10°C	5°C
i)	M 20	24	32	46	71
ii)	M 25	22	30	42	65
iii)	M 30	20	27	38	59
iv)	M 40	17	23	33	50

h) Inspection, Temperature and Humidity Records

During cold weather inspection, personnel shall keep a record of the date, time, outside air temperature, temperature of concrete at the time of placing and general weather (calm, windy, clear, cloudy, *etc.*). The record shall include the temperature at several points within the enclosure and on the concrete surface, corners and edges in sufficient number to show the *highest* and the *lowest* temperatures of the concrete.

When using an accelerator in approved dosage, the minimum pre-hardening period can be reduced by 25 percent. But, whatever the measure taken to accelerate the hardening, the minimum pre-hardening period shall not be less than the values given below.

<i>Curing Temperature (°C)</i>	<i>Minimum pre-hardening Period (in hours)</i>
12	20
16	15
24	10
36	5

j) Concrete Testing

The testing of the properties of the concrete constituents shall be carried out in accordance with IS 1199. Test results shall be used for fixing the time of removal of insulations, forms, etc, or be the basis for further precautions at the building site. In addition to the regular quality control, the following shall be considered:

- 1) Determine suitability of concrete making materials for winter concreting and control of the properties of fresh concrete;
- 2) Record air and concrete temperatures at placing and during concreting; and
- 3) Control strength development in the structure by testing similarly cured specimens.
- 4) The specimens shall not be tested in frozen state.

10.3.3 Under-Water Concreting

10.3.3.1 When it is necessary to deposit concrete under water, the methods, equipment, materials and proportions of the mix to be used shall be submitted to and approved by the *Engineer-in-Charge* before the work is started.

10.3.3.2 Under-water concrete should have a slump specified in 7.1.4.7. The water-cement ratio shall not exceed 0.6 for PCC work and 0.45 for reinforced concrete, depending on the grade of concrete or the type of chemical attack. For aggregates of 10 mm, 20 mm and 40 mm nominal maximum sizes, the cement content shall be at least 420 kg/m³, 380 kg/m³ and 360 kg/m³ of concrete, respectively.

10.3.3.3 Cofferdams or forms shall be sufficiently tight to ensure still water, if practicable, and in any case to reduce the flow of water to less than 3m/min through the space into which concrete is to be deposited. Cofferdams or forms in still water shall be sufficiently tight to prevent loss of mortar through the walls. De-watering by pumping shall not be done, while concrete is being placed or until 24 hours thereafter.

10.3.3.4 Concrete shall be deposited, continuously until it is brought to the required height. While depositing, the top surface shall be kept as nearly level as possible and the formation of seams avoided. The methods to be used for depositing concrete under water shall be one of the following:

- a) Tremie Concreting,
- b) Direct placement with pumps,
- c) Drop bottom bucket,
- d) Bags, and
- e) Grouting.

10.3.3.5 To minimize the formulation of laitance, great care shall be exercised not to disturb the concrete as far as possible while it is being deposited.

10.3.3.6 Ingredient materials of concrete to be placed under water shall meet all relevant specifications. In addition, the following shall apply:

- a) The maximum size of aggregate shall be 20mm for reinforced concrete members, but can be 40mm for non-reinforced concrete members.
- b) The high-range water reducing admixtures for placement of mass concrete (like Polycarboxylate or base superplasticizers) shall not be used, unless slump loss testing has shown no detrimental results. They may be used for placement of small masses of concrete.
- c) Fine aggregate shall be 40 to 55 percent of total aggregate. The slump shall be maintained in the range of 150mm to 230mm.

10.3.4 *Concreting in Aggressive Soils*

Requirements for concrete exposed to sulphate attack are provided in **8.2.2.4**. At sites where alkali concentrations are high or may become very high, the ground water shall be lowered by drainage so that it will not come into direct contact with the concrete. Additional protection shall be obtained by the use of impermeable barriers. In addition to the grade of concrete specified, the minimum cement content and maximum water-cement ratio shall be controlled. Also, Portland Slag Cement shall be used in consultation with subject specialists.

10.4 *Mass Concreting*

The general construction practices to be followed during mass concreting for plain and reinforced concrete structures shall be as per IS 457.

10.5 Reinforcing Steel Bars

Reinforcing steel shall be cut, fabricated and assembled, as per provisions indicated hereunder.

10.5.1 *Cutting*

Reinforcing bars shall be cut to length by shearing when the diameter of the bar is 12 mm or smaller, and sawing when diameter of the bar is 16 mm or larger. Cutting by burning with oxyacetylene equipment, and heating of reinforcing bars for bending or straightening shall not be permitted.

10.5.2 *Fabricating*

Bars should be bent in accordance with IS 2502. All bars shall be bent in ambient temperate conditions (called as cold condition). Proper bending tools shall be used, thus guarding against notching of the bars. Bent bars are then checked to ensure that lengths, depths, and radii are correctly reproduced, as per the bending details shown on the construction drawings.

The use of hand bending tools shall be limited to making adjustments during placing. The minimum diameter Φ of mandrel over which a bar of diameter ϕ shall be bent shall be taken as specified in IS 1786.

Steel bars shall not be re-bent. In particular, no dowels shall be left (by bending it into the formwork to stay on the surface of the member) for continuing the construction to stay within the formwork and straightened after the forms are removed.

10.5.3 *Assembling*

Assembling of reinforcing steel shall be as per the IS 2502. The spacing of ties in concrete structures shall be sufficient to prevent shifting of bars during placing of concrete. The main and stirrup reinforcing steel shall be assembled (preferably) outside the formwork, and then the assembled grill placed lowered into the position inside the formwork.

10.5.3.1 Reinforcement Detailing

10.5.3.1.1 Reinforcement detailing and placing shall be such that:

- a) it permits compaction by insertion of the needle vibrator;
- b) no two steels, which are manufactured by different methods, are required to be in contact with each other;
- c) they shall not be bent aside at construction joints and bent back afterwards into their intended positions; and
- d) they shall be placed and tied in such a way that concrete placement be possible without segregation of the mix.

10.5.3.1.2 When the reinforcing detailing scheme shown on the construction drawings does not meet the above two requirements, the said drawings shall be referred back to the structural designer for providing a revised detailing scheme meeting the above requirements.

10.5.3.1.3 Bar bending schedules prepared for all reinforcement work (in keeping with the procedure specified in IS 2502) shall comply with the following:

- a) All reinforcement shall be placed and maintained in the position shown in the drawings by providing proper cover blocks, spacers, supporting bars, etc.
- b) Crossing bars should not be tack-welded for assembly of reinforcement.
- c) The high strength deformed steel bars shall not be re-bent.

10.5.3.2 Handling of Reinforcement

Rough handling, shock loading (prior to embedment) and the dropping of reinforcement from a height shall not be permitted.

10.5.3.3 Tolerances on Placing of Reinforcement

10.5.3.3.1 Reinforcement bars shall be secured against displacement outside the specified limits. Spacers, chairs and other supports detailed on drawings, together with such other supports as may be necessary, shall be:

- a) used to maintain the specified nominal cover to the steel reinforcement,
- b) placed at a maximum spacing of 1 m or closer, and
- c) made of concrete of same or higher strength of PVC.

10.5.3.3.2 In any case, spacers, chairs and other supports shall be placed in the formwork within the following tolerance on clear cover:

- (1) +5, -0 mm, when specified clear cover is 30 mm or less, and
- (2) ± 5 mm, when specified clear cover is more than 30 mm.

10.5.4 *Welded Joints or Mechanical Connections*

Reinforcement bars shall not be welded. Splicing of bars can be done by:

- a) lapping, when diameter of bar is 12mm or less, and
- b) only by use of mechanical couplers (complying with IS 16172), when diameter of bar is 16mm or more; Tests shall be conducted on such samples of bars coupled mechanically to prove that the splice is able to provide full strength in connected bars.

10.6 Prestressing Steel

Prestressing steel (wires, strands, tendons or cables) shall be cut, fabricated and assembled, as per provisions indicated hereunder.

10.6.1 *Cutting and Placing*

All cutting to length and trimming of the ends of wires or strands shall be done by suitable mechanical cutters.

10.6.1.1 *Straightness*

- a) The wire or strand, as supplied, shall be self-straightening when uncoiled.
- b) In high tensile steel bars, any straightening if required, shall be carried out by mechanical means. Bars shall not be straightened when their temperature is less than 10°C
- c) In no case, heat shall be applied to facilitate straightening of prestressing steel.

10.6.1.2 *Arrangement and Positioning*

- a) All prestressing steel shall be carefully and accurately located in the exact positions shown in the design drawings. The permissible tolerance in the location of the prestressing tendon shall be ± 5 mm. Curves or bends in prestressing cables required by the designer shall be gradual and the prestressing tendon shall not be forced around sharp bends exceeding 1 in 6 both in plan and elevation or be formed in any manner that is likely to set up undesirable secondary stresses. The minimum radius of curvature for curved cables shall be specified to ensure that bursting of side cover both perpendicular to the plane of curvature and in the plane of the ducts do not take place.

- b) The relative position of prestressing steel in a cable shall be maintained by suitable means such as sufficiently rigid and adequately distributed spacers.
- c) The internal cross-sectional area of duct shall not be less than 3 times the nominal area of prestressing tendons to ensure free flow of grout.
- d) The method of fixing and supporting the wires, cables, or tendons in the mould or the formwork shall be such that it is not displaced during the placing or compaction of the concrete or during tensioning of the steel.
- e) The type of fixtures used for positioning the steel shall be such that it does not give rise to friction greater than that assumed in the design.

10.6.1.3 *Jointing*

- a) Coupling units and other similar fixtures used in conjunction with the prestressing steel shall have an ultimate tensile strength of not less than the individual strengths of the wires or bars being joined.
- b) Hard-drawn wires and strands used in prestressed concrete work shall be continuous over the entire length of the tendon.
- c) High tensile steel bars may be joined together by means of couplings, provided the strength of the coupling is such that in a test of destruction, the bar shall fail before the coupling.
- d) Welding shall not be permitted in prestressing steel.

10.6.2 *Protection of Prestressing Steel and Anchorages*

In all constructions of the post-tensioned type, whether prestressing is subsequently bonded or not bonded with concrete, the anchorages, prestressing metallic sheathing material forming ducts in concrete and anchorages shall be given adequate protection against corrosion at all stages of construction and during service life.

10.6.2.1 *Construction with Unbonded Steel embedded in Concrete*

For protection of metallic sheathing material forming ducts, the durability requirements shall be followed. Furthermore, the internal surface of duct, prestressing steel and anchorage shall be protected by following methods in suitable combination depending upon severity of environment and extent of layers of protection considered desirable:

- a) Coating of steel and duct by reactive metals, like zinc (Zn) or zinc plus aluminium (Zn + Al), preferably applied by hot dipping method.
- b) Use of non-corroding sheath material, like high density polyethylene and fibre reinforced plastic.
- c) Coating of steel by non-reactive materials, like epoxy (factory applied).
- d) Covering individual uncoated steel or coated steel strands/wires in sheaths formed by extrusion process in factory using non-corroding material, like

HDPE or similar. The space between sheathing and duct can be filled with corrosion inhibiting materials, like grease and wax.

- e) In case of large size prestressing units, use of protective and replaceable corrosion inhibition material for grouting the ducts, such as grease, wax polyurethane foam, wherein any corrosive trace elements, shall be controlled to remain within acceptable limits. Systems for checking the efficacy of the method during service life and provisions for replacing/ making up loss of filler material shall be incorporated.
- f) External parts of anchorages and projecting cables shall be covered by suitable casing and protected by suitable material and method.

For selection of appropriate method, material specifications, installation process, etc, specialist literature should be referred to.

10.6.2.2 *Construction with Unbonded Steel and Anchorages not embedded in Concrete (External Prestressing Elements)*

The methods given in **10.6.2.1** shall be applicable for this type of construction also. In addition, the material, strength and environmental resistance of the ducts forming external enclosure shall provide adequate mechanical strength and stability against environmental attack, including solar radiation. The ducts shall be strong enough to resist pressure from the grouting operations.

10.6.2.3 *Construction with Post-tensioned Bonded Prestressing Steel*

Internal prestressing sheathing material as well as prestressing steel is protected by dense alkaline concrete cover, which shall be of quality and thickness as specified. The steel inside ducts is best protected in long terms by cement grouts, which are properly executed as per **10.6.3**. The protection by grouting can be further augmented by use of methods given below:

- a) Use of coated steel using Zn, Zn+, Al or epoxy, and
- b) Use of non-corroding impervious sheathing material, like HDPE. But, bond between sheathing and concrete and sheathing and grout shall be adequate as needed by design. This method is particularly suitable in presence of migrating water or variable drying/wetting condition of concrete of severe external penetration of corrosive agents.

External parts of anchorages and projecting cables should be covered by suitable casing and protected by suitable material and method.

10.6.2.4 *Protection during construction period*

- a) The prestressing steel, sheathing and anchorages shall be manufactured, transported and stored at site in such a way as to provide them with adequate corrosion protection. Use of proper packaging and provision of externally applied passivating agents, oil films, etc, which is carried over in factory itself is recommended.
- b) If prestressing steel or metallic sheathing material is left unprotected at site or embedded in concrete during construction period longer than two weeks, it shall be protected by flushing with oil-water mixture (such as cutting oil) with periodic reflushing at not more than three months interval.
- c) After stressing, the sheaths or steel shall be provided with permanent protection as soon as possible, preferably within one week. While providing protection by pressure grouting of cement, care shall be taken that the neighbouring cables are not penetrated by grout. In case of doubt, such cables shall be grouted in immediate sequence before the earlier grout could have set. If the earlier stressed cables are not grouted to avoid this problem, they shall be protected by periodic flushing by oil-water mixture as given in (b).

10.6.3 Prestressing

The provisions hereunder shall be applicable to prestressing processes.

10.6.3.1 Prestressing Equipment

10.6.3.1.1 Tensioning apparatus

- a) These requirements shall apply to both the pre-tensioned and the post-tensioned methods of prestressing concrete except where specifically mentioned otherwise.
- b) Prestressing steel may be tensioned by means of hydraulic or mechanical jacks. The tension apparatus shall be such that it facilitates measurement and control of application of force. The tensioning apparatus shall be independently supported so that it does not induce any undesirable secondary stresses.
- c) The anchorage provided for the temporary gripping of tendons on the tensioning apparatus shall be secured such as not to damage the tendon.
- d) Devices attached to the tensioning apparatus for measuring the applied force shall be such that they do not introduce errors exceeding 5 percent.

10.6.3.1.2 Temporary gripping device

Prestressing tendons may be gripped by wedges, yokes, double cones or any other approved type of gripping devices. The prestressing wires may be gripped singly or in groups. Gripping devices shall be such that in a tensile test, the wire or wires fixed by them would break before failure of the grip itself.

10.6.3.1.3 *Releasing device*

The releasing device shall be so designed that during the period between the tensioning and release, the tension in the prestressing elements is fully maintained by positive means, such as external anchorages. The device shall enable the transfer of prestress to be carried out gradually so as to avoid large difference of tension between wires in a tendon, severe eccentricities of prestress or the sudden application of stress to the concrete.

10.6.3.1.4 *Anchorage device*

The anchorage device may be patented or otherwise, which complies with the requirements given below:

- a) The anchoring device shall be strong enough to resist in all respects a force equal to at least the breaking strength of the prestressing tendon it anchors.
- b) The anchorage shall transfer effectively and distribute, as evenly as possible, the entire force from the prestressing tendon to the concrete without inducing undesirable secondary or local stresses.
- c) The anchorage shall be safe and secure against both dynamic and static loads as well as against impact (see Annex C).
- d) The anchorage shall have provision for the introduction of a suitable protective medium, such as cement grout, for the protection of the prestressing steel unless alternative arrangements are made.

10.6.3.2 *Procedure for Tensioning and Transfer***10.6.3.2.1** *Stressing*

The tensioning of prestressing tendons shall be carried out in a manner that will induce a smooth and even rate of increase of stress in the tendons. The prestressing shall be controlled by measuring both the force applied to the tendon and the elongation of the tendon. Stressing schedule shall be incorporated in the design, which may include the following:

- a) Details of equipment and procedure for applying prestress, specification of one end or simultaneous stressing from both ends of tendons, stressing in single stage or multiple stages, re-stressing for adjustment of force or for monitoring of force, if required, etc.
- b) Calculated amount of tensile force and corresponding expected elongation of each tendon, when methods described in (a) are followed.
- c) Minimum requirements of strength of concrete at transfer, stressing sequence with respect to age of concrete or stage of construction, requirements with respect to grouting operations of neighbouring tendons.
- d) Method(s) of measuring force and elongation.

- e) Tolerances of variation from design requirements stated in (i) and (ii), beyond which reference should be made to the designer (In the absence of any specified provision provided by the designer, method described in 10.6.3.2.3 may be adopted for acceptance).
- f) Assumptions made in calculation of force elongation relationship; these include effective E_{sp} value of tendon, cross sectional area of tendon, coefficient of friction and wobble factor between tendon and duct, assumed slip in anchoring devices (if any), efficiency of prestressing jacks where force is measured indirectly by pressure gauges on jacks, corrections to elongation due to changes in gripping length beyond anchorages during stressing, and any other relevant information.

10.6.3.2.2 Slack in prestressing

Any slack in the prestressing tendon shall be overcome first by applying a small initial tension. The initial tension required to remove slackness shall be taken as the starting point for measuring elongation. Further increase of tension shall be carried out in suitable steps and corresponding elongations noted. The 'force-elongation' relation shall be recorded in tabular and/or graphical form. The magnitude of initial effective elongation corresponding to initial tension applied to remove slackness shall be obtained from the recorded and linearized portion of measured tension elongation relationship and added to the measured elongation to give the total elongation. Alternatively, the same correction can be carried out graphically as shown in Fig. 86.

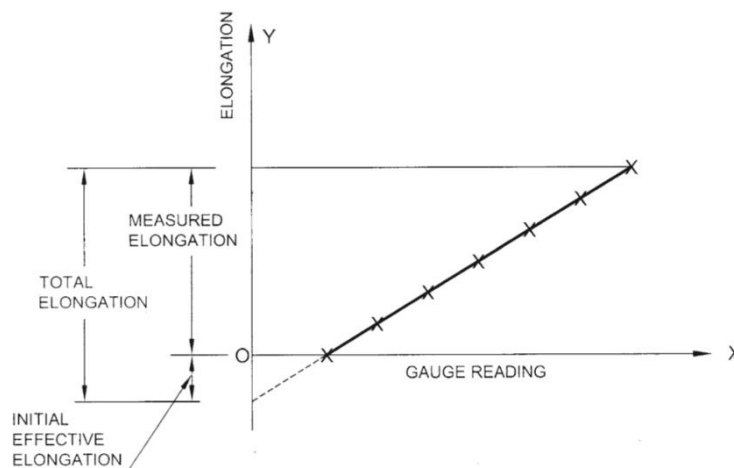


FIG. 86 DETERMINATION OF ACTUAL ELONGATION

10.6.3.2.3 In practice, the force and elongation of tendon may not exactly match with the expected values given in stressing schedule. In such cases either the force (or the elongation) will be achieved first and the other value lags behind. In such cases, the force (or elongation) shall be further increased, but not exceeding 5 percent of the design value till the elongation (or force), which had lagged behind reaches the design value. If, even after reaching 5 percent extra value of the force (or elongation), the

other lagged quantity does not reach the design value, reference shall be made to the designer for review and corrective action.

When two or more prestressing wires or strands are to be tensioned simultaneously by the same tensioning apparatus, care shall be taken to ensure that all such tendons are of the same length from grip to grip.

The placement of tendons and the order of stressing and grouting shall be so arranged that the prestressing steel, when tensioned and grouted, does not adversely affect the adjoining ducts.

10.6.3.2.4 *Measurement of prestressing force*

10.6.3.2.4.1 The force induced in the prestressing tendon shall be determined by means of gauges attached to the tensioning apparatus as well as by measuring the extension of the steel. It is essential that both methods are used jointly so that the inaccuracies to which each is singly susceptible are minimized. Due allowance shall be made for the frictional losses in the tensioning apparatus.

10.6.3.2.4.2 All dynamometers and pressure gauges including master gauge shall be calibrated by an approved laboratory immediately prior to use and then at intervals not exceeding 3 months and true force determined from the calibration curve. Further, whenever there is a breakage of tendons, the calibration shall be carried out before reuse of the gauges. The pressure gauges shall be accurate within 2 percent of their full capacity.

10.6.3.2.4.3 In measuring the extension of prestressing steel, any slip which may occur in the gripping device shall be taken into consideration.

10.6.3.2.5 *Breakage of wires*

The breakage of wires in any one prestressed concrete member shall not exceed 2.5 percent during tensioning. If the breakages are more than 2.5 percent, it shall be examined and corrective actions taken. Wire breakages after anchorage, irrespective of percentage, shall not be condoned without special investigations.

10.6.3.2.6 *Transfer of prestressing force*

10.6.3.2.6.1 The transfer of the prestress shall be carried out gradually so as to avoid large differences of tension between wires in a tendon, severe eccentricities of prestressing force and the sudden application of stress to the concrete.

10.6.3.2.6.2 Where the total prestressing force in a member is built up by successive transfers to the force of a number of individual tendons on to the concrete, account shall be taken of the effect of the successive prestressing.

10.6.3.2.6.3 In the long line and similar methods of prestressing, when the transfer is made on several moulds at a time, care shall be taken to ensure that the prestressing force is evenly applied on all the moulds, and that the transfer of prestress to the concrete is uniform along the entire length of the tension line.

10.6.3.3 *Grouting*

The equipment required for grouting and the process of grouting shall be as specified hereunder.

10.6.3.3.1 *Equipment*

10.6.3.3.1.1 *Grout colloidal mixer*

Grout shall be continuously mixed in a colloidal mixer with a minimum speed of 1,000 rpm and travel of discharge not exceeding 15 m/s.

10.6.3.3.1.2 *Grout pump*

The pump shall be of positive displacement type, and should be capable of injecting the grout in a continuous operation and not by way of pulses. The grout pump shall be fitted with a pressure gauge to enable pressure of injection to be controlled. The minimum pressure at which grout should be pumped shall be 0.3 MPa, and the grout pump shall have a relief arrangement for bypass of the grout in case of built up of pressure beyond 1.0 MPa. The capacity of the grout pump shall be such as to achieve a forward speed of grout of around 5 to 10m/ min.

Use of hand pumps and compressed air operated equipment for injection of grout shall not be permitted.

10.6.3.3.1.3 *Water pump*

Before commencement of grouting, a stand-by direct feed high pressure water pump should be available at site for an emergency.

In case of any problem in grouting the ducts, such pump shall immediately be connected to the duct and all grout flushed by use of high pressure water flushing. Therefore, it is necessary to have adequate storage of clean potable water for operation of the water pump for such emergencies.

10.6.3.3.1.4 *Grout screen*

The grouting equipment shall contain a screen having a mesh size of IS Sieve No. 106 (IS Sieve No. 150, if sand is used). Prior to introduction into the grout pump, the grout shall be passed through such screen. This screen shall be easily accessible for inspection and cleaning.

10.6.3.3.1.5 *Connections and air vents*

Standard details of fixing inlets, outlets, and air vents to the sheathing and/or anchorage shall be followed as recommended by specialist supplier of the system of prestressing. In general, all connections shall be of the 'Quick Couple' type and suitable reducers provided at change of diameters.

10.6.3.3.2 *Mixing of grout*

Proportions of materials shall be based on field trials made on the grout before commencement of grouting, but subject to the limits specified above. The materials shall be measured by mass.

10.6.3.3.3 *Preparations before grouting*

The following step-wise procedure shall be adopted:

- a) Water shall be added to the mixer, following by cement, and sand, if used. Admixture, if any, may be added as recommended by the manufacturer.
- b) Mixing time depends upon the type of the mixer but will normally be between 2 and 3 min. But, mixing shall be for such duration as to obtain uniform and thoroughly blended grout, without excessive temperature increase or loss of expansive properties of the admixtures. The grout shall be continuously agitated until it is injected. Once mixed, no water shall be added to the grout to increase its fluidity.
- c) Grouting shall be carried immediately after stressing the tendon, but within 2 weeks. Whenever this stipulation cannot be complied with for unavoidable reasons, adequate temporary protection of the steel against corrosion shall be taken till grouting by methods or products which will not impair the ultimate adherence of the injected grout. For structures in aggressive environment, anchorage ends shall be sealed immediately.
- d) Any trace of oil, if applied to steel for preventing corrosion, shall be removed before grouting operation.
- e) Ducts shall be flushed with water for cleaning as well as for wetting the surfaces of the duct walls. Water used for flushing should be of same quality as used for

grouting. It may, but, contain about 1 percent of slaked lime or quick lime. All water shall be drained through the lowest drain pipe or by blowing compressed air through the duct.

- f) The water in the duct shall be blown out with oil free compressed air. As blowing out water from duct for cables longer than 50m draped up at both ends by compressed air is not effective, outlet/vent provided at or near the lowest point shall be used to drain out water from duct.
- g) The connection between the nozzle of the injection pipe and duct shall be such that air cannot be sucked in.
- h) All outlet points including vent openings shall be kept open prior to commencement of injection grout.
- i) All air in the pump and hose shall be expelled before grouting. The suction circuit of the pump shall be air-tight.

10.6.3.3.4 *Injection of grout*

The following step-wise procedure shall be adopted:

- a) After mixing, the grout shall be kept in continuous movement.
- b) Injection of grout shall be continuous and uninterrupted.
- c) Injection shall be effected from the lowest anchorage or vent of the duct, for vertical cable or cables inclined more than 60° to the horizontal.
- d) The method of injection shall ensure complete filling of the ducts. To verify this, it is advisable to compare the volume of the space to be filled by the injected grout with the quantity of grout actually injected.
- e) Grouting shall be commenced initially with a low pressure of injection of up to 0.3MPa, and then increased until the grout comes out at the other end. The grout shall be allowed to flow freely from the other end until the consistency of the grout at that end is the same as that of the grout at the injection end. When the grout flows at the other end, the duct should be closed and pressure built up in it. Full injection pressure at about 0.5MPa shall be maintained for at least 1 min before closing the injection pipe. It is a recommended practice to provide a stand pipe at the highest point of the tendon profile to exit all water displaced by sedimentation or bleeding. If there is a build up of pressure in excess of 1.0 MPa without flow of grout coming at the other end, the grouting operation shall be discontinued and the entire duct flushed with high pressure water. Also, the bypass system indicated in **10.6.3.3.1.2** is essential for further safety.
- f) In the case of cables draped downwards for example, in cantilever construction simultaneous injection from both ends shall be adopted (*see* Fig. 87).
- g) Grout shall be used within 30 min after mixing or the time limit for which the manufacturer has confirmed the properties meet all requirements to be satisfied at the temperature. Grout not used within above specified time after mixing shall be rejected.

- h) Disconnection shall be facilitated by a short length of flexible tube connects the duct and injection pipe. This may be squeezed and cut off after the grout has hardened.

10.6.3.3.5 *Precautions and recommendations for effective grouting*

The following shall be ensured:

- a) In cold and frosty weather, injection of grout shall be postponed unless special precautions are taken. If frost is likely to occur within 48 h after injection, heat shall be applied to the member and maintained for at least 48 h after injection, so that the temperature of the grout does not fall below 5°C. Prior to commencement of grout, care shall be taken to ensure that the duct is completely free of frost/ice by flushing with warm water but not with steam.
- b) When the ambient temperature during the day is likely to exceed 35°C, grouting shall be done in the early morning or late evening hours.
- c) When the cables are threaded after concreting, the duct shall be temporarily protected during concreting by inserting a stiff rod or a rigid PVC pipe or any other suitable method.
- d) During concreting, care shall be taken to ensure that the sheathing is not damaged. Needle vibrators shall be used with extreme care by well experienced staff only, to ensure the above requirements.
- e) Prestressing cables shall be moved in both directions during the concreting operations. This can be done by light hammering the ends of the wires/strands during concreting. Also, 3 to 4 h after concreting, the cable should be moved both ways through a distance of about 200 mm. With such movement, any leakage of mortar, which has taken place in spite of all precautions loses bond with the cables, thus reducing the chance of blockages. This operation can be done also by fixing prestressing jacks at one end pulling the entire cable and then repeating the operation by fixing the jack at the other end.
- f) The cables to be grouted shall be separated by as much distance as possible.
- g) In case of stage prestressing, cables tensioned in the first stage shall not remain ungrouted till all cables are stressed. It is a good practice, while grouting any duct in stage prestressing, to keep all the remaining ducts filled up with water containing 1 percent lime or by running water through such ducts till the grout has set. After grouting the particular cable, the water in the other cables shall be drained and removed with compressed air to prevent corrosion.
- h) Care shall be taken to avoid leaks from one duct to another at joints of precast members.
- j) End faces where anchorages are located are vulnerable points of entry of water. They have to be necessarily protected with an effective barrier. Recesses shall be packed with mortar concrete and preferably be painted with water proof paint.
- k) After grouting is completed, the projecting portion of the vents shall be cut off and the face protected to prevent corrosion.

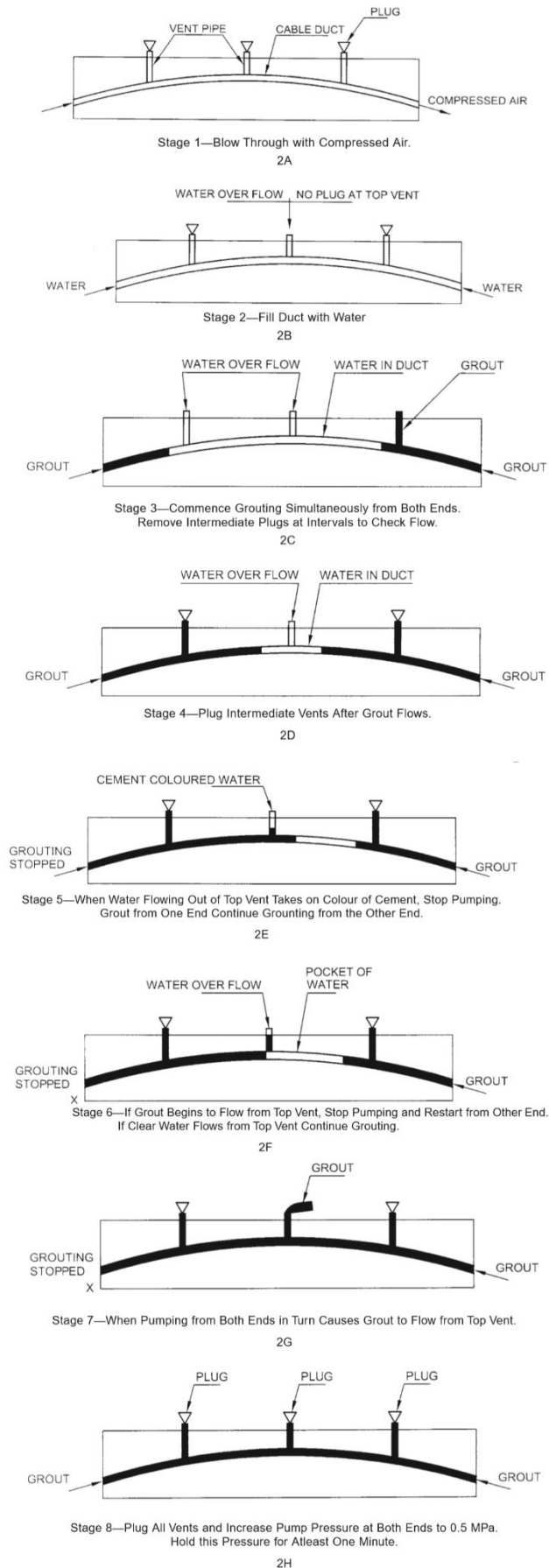


FIG. 87 PROCEDURE FOR GROUTING OF CABLES DRAPED DOWNWARDS

10.7 Untensioned Steel Bars In Prestressed Concrete Structures

All requirements of steel reinforcement bars in RC structures are applicable for untensioned steel in PSC structures.

10.8 Special Considerations

10.8.1 *Construction Joints*

Construction joints (also called cold joints) shall be avoided to the extent possible. If this is not possible, their number shall be minimized. Concreting shall be carried out continuously up to construction joints, the position and arrangement of which shall be indicated by the designer. Construction joints should comply with IS 11817.

10.8.1.1 Construction joints shall not be located on the portion subjected to large shear force. The direction of construction joints is crossed at right angles to the direction of compressive force in member. When construction joints are located on the portion subjected to shear force:

- a) If shear resistance to be mobilized is moderate at the construction joint, notches shall be provided and the construction joint shall be reinforced appropriately; and
- b) If shear resistance to be mobilized is large at the construction joint, shear keys may be provided.

10.8.1.2 Construction joints shall be placed at accessible locations to permit cleaning out of laitance, cement slurry and unsound concrete, to create rough or uneven surface. Laitance and cement slurry shall be cleaned out using wire brush on the surface of joint immediately after initial setting of concrete and to clean out the same immediately thereafter. The prepared surface shall be in a clean saturated surface dry condition, when fresh concrete is placed against it.

10.8.1.3 In the case of construction joints at locations where the previous pour has been cast against shuttering, a rough surface for the previously poured concrete shall be obtained to expose the aggregate with a high pressure water jet or any other appropriate means. Fresh concrete shall be thoroughly vibrated near construction joints so that mortar from the new concrete flows between large aggregates and develops proper bond with old concrete.

10.8.1.4 When leaving dowels open in structural members at a construction joint for some time before continuing the construction, precautions shall be taken. Cement slurry shall be applied on the bars during the exposed period. When steel bars are showing the signs of rusting, they shall be cleaned properly before continuing the construction. Anti-corrosive treatment may be done, if necessary.

10.8.1.5 When continuing construction at a construction joint, it shall be prepared to assure bond between the old and new concretes, by:

- a) removing thoroughly the sprayed curing membranes and release agents from joint surfaces; and
- b) applying an appropriate bond coat at the joint prior to placement of new concrete, which can be SBR latex-based or epoxy-based.

10.8.2 *Jointing of Butted Assemblies in Prestressed Concrete Construction*

10.8.2.1 The joints of butted assemblies shall be made of either cement grout or cement mortar or concrete. Cement grouting shall be used for joints up to 12 mm thick. For joints thicker than 12 mm and up to 75 mm, cement mortar shall be used. The mortar, which may be made of one part cement and one-and-a-half parts sand shall be of a dry consistency and shall be packed hard in layers so that it rings true. Where joints exceeding 75 mm are encountered, the joint shall be made up of concrete. Use of epoxy may be permitted with the approval of engineer-in-charge.

10.8.2.2 The stressing operations may be carried out in case of mortar joints immediately after placing the mortar but the stress in the mortar shall not exceed 7 MPa. In grouted joints and concrete joints, the allowable stress in the first 24 h after placing of the grout or concrete in the joint shall approximate as closely as possible to the strength of the grout or concrete used.

10.8.2.3 The holes for the prestressing tendons shall be accurately located and shall be in true alignment when the units are put together.

10.8.2.4 Full tensioning shall not be carried out until the strength of the concrete or mortar in the joint has reached twice the stress at transfer.

10.8.3 *Instability during Erection of Precast members*

In evaluating the erection procedure for lifting of slender members, the following factors require consideration:

- a) Member geometry, location of lifting points, method of lifting, and tolerances in construction shall be considered;
- b) All beams, which are lifted on vertical or inclined slings, shall be checked for lateral stability and lateral moment on account of tilting of beam due to inaccuracies in location of lifting points, and due to the lateral bow.
- c) For calculating the factor of safety against lateral instability, reference may be made to specialist literature; γ_i shall not be less than 2.
- d) For determining the lateral moment due to tilting, realistic values which are not likely to be exceeded in practice shall be assumed for the eccentricity of lifting

points and the lateral bow. The maximum tensile stress for $\gamma_i/(\gamma_i - 1)$ times the lateral moment due to tilting shall not exceed 1.5 MPa.

11 QUALITY ASSURANCE

Quality of concrete structures depends upon total quality management system which include quality policy, quality assurance plan for designs, drawings and detailing, quality control during construction, acceptance criteria, requirements of deployment of trained and skilled personnel, corrective action and documented systems, etc.

For the properties of the completed structure to be consistent with the requirements and the assumptions made during the planning and the design phase, adequate quality assurance measures shall be taken. The construction shall result in satisfactory strength, serviceability and long-term durability so as to keep the overall life-cycle cost to a minimum.

11.1 Levels of Quality Assurance (QA)

The level of quality assurance required will be different in different projects, depending on the importance of project, organisational setup, etc. For the purpose of this standard, the minimum quality assurance requirements shall be categorised under three levels, namely:

- a) QA1: Normal Quality Assurance Level,
- b) QA2: Medium Quality Assurance Level, and
- c) QA3: High Quality Assurance Level.

The levels of QA to be adopted in a project shall be decided by the owner or project authorities. But, as guidance, minimum level of QA required for different types of structures shall be as per Table 94.

Table 94 Applicability of the three levels of QA in Construction
(Clause 11.1)

Level	Applicable Constructions
(1)	(2)
QA1	Structures up to 4 storey with single floor plan area not more than 500 m ²
QA2	Structures up to 10 storey and single floor plan area not more than 5,000 m ² , or structures of lesser size but high importance or structures with large span roofs, underground structures, structures with deep foundations
QA3	Structures more than 10 storey and single floor plan area more than 5,000 m ² , or structures of lesser size but high importance

11.2 Requirements of Quality Assurance

The basic requirements of different levels of QA in construction of concrete structures shall be as per Table 95. For any item specified therein for any level of QA, requirements higher than those specified may be adopted depending upon the type, size and importance of the project.

Table 95 Minimum Measures required in QA levels during Construction
(Clause 11.2)

Sl No.	QA1	QA2	QA3
(1)	(2)	(3)	(4)
1) Design and Drawings			
i)	Design office with internal QA procedures	As QA1, but with checking and reviewing by in-house or external consultants or organisations	As in QA2, but with checking and reviewing by external experts or organisations
ii)	Drawings can be type-designs with necessary notes to cover variations	Project specific drawings	Project specific drawings
iii)	As built drawings are required	As in QA1, but all changes incorporated and revised drawings issued	As in QA2
2) QA Manual and Quality Assurance Plan (QAP)			
	Requirements can be covered in the Tender, or in a standard QAP of owner or consultant	Specific QA Manual or Standard Quality Manual with project specific QAP, including necessary inspection, sampling and testing formats (as per IS 15883)	As in QA2, but with roles and responsibilities defined of all controlling authorities mentioned in the QA Manual (as per IS 15883)
3) Constructor			
	Complying with pre-fixed requirements of experience and ability	Same as in QA1	As in QA2, but having accredited and documented QA system in place
4) Supervision			
i)	Full-time supervision by constructor	Full-time supervision by constructor	As in QA2, but with pre-approved site organisation chart, both by constructor and for secondary supervision
ii)	Secondary periodical supervision by owner, PMC or consultant	Full time supervision by owner/PMC/consultant	
5) Quality Surveillance and Audit			

i)	By constructors qualified engineer and owners representative	As in QA1 with separate QA cell or staff of constructor and owner's representative	As in QA2, but additional third party quality surveillance and audit is essential.
ii)	Third party quality surveillance optional	Regular surveillance and audit is essential. Third party quality audit recommended.	
6) Concrete Making Material Testing			
	As per 11.5		
7. Concrete Testing			
	As per 11.4		
8. Concrete Mix Design and Concrete Production			
	As per 11.6		
9. Durability Testing			
	As per 11.4.5		
10. Non-Destructive Testing			
	Not essential	Recommended, but not essential	Rebound hammer and UPV Tests to be conducted at predetermined frequency to assess in situ strength. (rebound hammer to be more frequently used).
11. Concrete Core Test			
	Only if required.	Only if required	A few locations to verify in-situ strength and to keep record as reference for use in future monitoring*
12. Chemical testing of hardened concrete for chlorides and sulphates			
	Optional for site mixed concrete Certificate by RMC supplier	Once at start of construction and subsequently in case of change of source of material or significant change in water test results	As in QA2 plus one test every six months.
12. Steel Reinforcement			
	As per 11.7		
13. Prestressing Materials and Grouts			
	As per 11.8		
14. Any Other Item Involved in Concrete Work			

	To be decided by owner or engineer-in-charge to ensure that material conforms to the specifications

***NOTE**

1 Core extraction and testing to be done by experienced personnel at non-critical locations to be decided in consultation with the structural designer.

2 For Non-destructive testing and core tests, relevant parts/sections of IS 516 shall be followed.

11.3 Quality Inspections

Inspections shall examine whether or not the construction complies with the design. The inspection shall be in accordance with the procedure laid down for materials, records, workmanship and other construction activities.

11.3.1 Inspections before start of construction shall ensure that:

- a) Design and drawings/details are capable of being executed to a suitable standard, with due allowance for dimensional tolerances;
- b) Clear instructions should be there for engineers on inspection standards and permissible deviations;
- c) Elements critical to workmanship, structural performance, durability and appearance are identified; and
- d) A system should be there to verify that quality in individual parts of the structure is satisfactory, especially the critical ones.

11.3.2 Quality inspections during construction (with testing wherever required) shall be assured at every stage for achieving desired quality in construction. Inspections shall be carried out and appropriately documented for:

- a) Materials to be used in concrete making,
- b) Formwork,
- c) Reinforcement work,
- d) Concrete production delivery and placing, and
- e) Post-concreting inspections.

11.3.3 Immediately after stripping the formwork, all concrete shall be inspected for defective work (if any) or small defects (if any) either removed or made good as applicable after approval of Competent Authority before concrete has thoroughly hardened. The defects found and remedial measures taken shall be documented appropriately.

11.4 Sampling and Testing of Concrete

Concrete shall be tested for performance parameters of fresh and hardened concrete, namely workability, strength and durability as per requirements of this standard. Tests for additional performance parameters, if any, shall be conducted as per project specific requirements. The frequency of sampling, testing and acceptance criteria for different tests shall be considered as given hereunder.

11.4.1 Test for Workability

Workability test shall be conducted as per IS 1199. Frequency of these tests shall be 1 sample for every 6 m³ of concrete or part thereof. A random sampling procedure shall be adopted to ensure that each concrete batch shall have a reasonable chance of being tested that is, the sampling should be spread over the entire period of concreting and cover all mixing units. Also, when concrete is supplied by a RMC plant, workability of concrete in each transit vehicle shall be checked at site immediately before use.

a) Acceptance Criteria

Measured values of slump shall be within the tolerances given in the table below. When high range superplasticizers like Polycarboxylate (PC) based superplasticizers are used (which tend to give higher slump), higher tolerance on upper side may be permitted, if strength tests are satisfactory, and higher variation in slumps were observed during mix design stage also.

For other workability tests, measured slump shall be within the range specified below:

<i>Slump Specified in Design (mm)</i>	<i>Permitted Tolerance in Slump (mm)</i>
< 60	-10, +10
75 - 125	-15, +25
> 125	-20, +40

11.4.2 Tests for Strength of Concrete**11.4.2.1 Specimen**

- a) Tests shall be performed on cubes for assessing compressive strength and on beam specimen for flexural strength. 3 test specimens shall be made for each sample for testing at 28 days. Additional specimens may be required for various purposes, such as to: (a) determine the strength of concrete at 7 days or at the time of striking the formwork, (b) determine the duration of curing, or (c) check the testing error. Additional specimens may also be required for testing specimens cured by accelerated methods as described in IS 9013. The specimens shall be tested as described in IS 516.

Testing using 100 mm cubes of high strength concrete, having maximum size of aggregates of 20mm may be permitted by engineer-in-charge, if felt necessary for QC purposes and provided with a relationship established at mix design stage. But, intermittent tests shall be carried out regularly for confirmation with 150 mm cubes.

- b) For compressive strength, samples from fresh concrete shall be taken as per IS 1199, and cubes made, cured and tested at 28 days in accordance with IS 516. To get a relatively quicker assessment of the quality of concrete, optional tests may be performed (beam specimen) by assessing modulus of rupture at $72h \pm 2h$ or at 7 days, or compressive strength tests at 7 days may be carried out in addition to 28 days. For this purpose, the values shall be arrived at based on actual testing. In all cases, the 28 days compressive strength shall alone be the criterion for acceptance or rejection of the concrete.

11.4.2.2 Frequency of sampling

A random procedure shall be adopted for sampling to ensure that each batch of concrete has reasonable chance of being tested that is, the sampling is spread over the entire duration of concreting and covers all mixing units. The minimum frequency of sampling of concrete of each grade shall be in accordance with the table given below. Also, at least 1 sample shall be taken from each shift. Where concrete is produced at continuous production unit, such as ready-mixed concrete plant, frequency of sampling shall be 1 sample for every 50 m³ of concrete.

Quantity of Concrete in Work (m ³)	Number of Samples
1-5	1
6-15	2
16-30	3
31-50	4
51 and above	4 +
	1 additional sample for each additional 50 m ³ or part thereof

11.4.2.3 Test results

The test results of the sample shall be the average of the strengths of 3 specimens. The individual variation shall not be more than ± 15 percent of the average strength. If it is more, the test result of the sample shall be deemed to be invalid.

11.4.2.4 Acceptance criteria

11.4.2.4.1 Compressive strength

The concrete shall be deemed to comply with the compressive strength requirements when both of the following conditions are met with:

- a) The mean strength determined from any group of four non-overlapping consecutive test results complies with the appropriate limits of Table 96.
- b) Any individual test result complies with the appropriate limits of Table 96.

11.4.2.4.2 Flexural strength

The concrete shall be deemed to comply with the flexural strength requirements when both of the following conditions are met with:

- a) The mean strength determined from any group of four non-overlapping consecutive test results exceeds the specified characteristic strength by at least 0.3 MPa; and
- b) The strength determined from any individual test result is not less than the specified characteristic strength by 0.3 MPa.

11.4.2.4.3 For concrete of quantity up to 30 m³ in a shift (where the number of samples to be taken is less than 4 as per frequency of sampling given in **11.4.2.2**), the mean of test results of 4 consecutive non-overlapping samples shall be calculated (including the samples of preceding and succeeding days). Where it is not possible for small works (where total quantity involved is less than 30 m³, at least 2 samples shall be taken and no sample shall have strength less than f_{ck} and mean of 2 samples not less than $f_{ck} + 2$.

11.4.2.4.4 The quantity of concrete represented by a group of four consecutive test results shall include the batches from which the first and last samples were taken together with all intervening batches. For the individual test result requirements given in Table 96 or in above, only the particular batch from which the sample was taken shall be at risk. Where the mean rate of sampling is not specified, the maximum quantity of concrete that four consecutive test results represent shall be limited to 60 m³.

11.4.2.4.5 If the concrete is deemed not to comply pursuant to **11.4.2.4.4**, the structural adequacy of the parts affected shall be investigated, and any consequential action shall be taken, as needed.

Table 96 Characteristic Compressive Strength Compliance Requirement
(Clauses 11.4.2.4.1 and 11.4.2.4.4)

Specified Grade of concrete	Minimum value of Individual Test Results (in MPa)	Minimum Mean of Group of 4 Non-Overlapping Consecutive Test Results (MPa)
M10 to M55	$f_{ck} - 2$	$Max[f_{ck} + 0.825\sigma; f_{ck} + 0.5X]$

M60 to M100	$f_{ck} - 0.5X$	where σ = Established standard deviation (rounded off to nearest 0.5 MPa), and X = Assumed standard deviation as per Table 11, when σ is not available based on 30 samples.

NOTE

In the absence of established value of standard deviation, the values given in Table 6.6 may be assumed. Attempt should be made to obtain results of 30 samples as early as possible to establish the value of standard deviation.

11.4.2.4.6 Concrete of each grade shall be assessed separately.

11.4.2.4.7 Cast concrete is liable to be rejected, if:

- it is porous or honey-combed, its placing has been interrupted without providing a proper construction joint;
- the reinforcement has been displaced beyond the tolerances specified; or
- construction tolerances have not been met.

However, the hardened concrete may be accepted after carrying out suitable remedial measures to the satisfaction of the engineer-in-charge.

11.4.3 *Non-destructive Tests during construction*

Non-destructive tests, wherever specified, shall be carried out as part of quality assurance measures based on given frequency. Frequency of NDTs for QA3 shall be decided and specified depending on type of structure, approximate daily concrete quantity involved, etc. The minimum frequency recommended shall be one set of tests for every 100–150 m³ of concrete for rebound hammer and ultrasonic pulse velocity tests.

- For rebound hammer, one set shall comprise of test at 3 locations of 3 members selected randomly. At each location, 6 readings shall be taken as per IS 516 (Part 5/Sec 4).
- For ultrasonic pulse velocity test, one set shall consist of three members and on each member about 1 m² of concrete surface shall be tested by cross probing as per IS 516 (Part 5/Sec 1).

11.4.3.1 *Acceptance Criteria*

When non-destructive tests are to be conducted to confirm in-situ compressive strength, a relation shall be established between NDT results and cube compressive strength results. During routine tests:

- a) The average of 3 test sets shall not be less than the established value by more than 10 percent and,
- b) The result of individual set of results shall not deviate by more than 20 percent.

11.4.4 Concrete Core Tests

Core tests are generally conducted when quality of concrete is in doubt and being examined. But, when specified, like in Quality level QA 3, limited cores (preferably of smaller diameter) can be taken from few larger size members at such locations which do not impact the load carrying capacity of member or structure. Such procedure shall be decided and documented in project-specific quality plan to be prepared before the start of the construction activity.

Frequency of concrete core tests for QA3 shall be decided and specified depending on type of structure, approximate daily concrete quantity involved, etc. The minimum frequency recommended shall be:

For core test, one sample (3 cores of at least 60 mm diameter and min. length of 100 mm) for every 750 m³ of concrete or part thereof for each grade of concrete. In case of concrete grade up to M50, if all cube results and rebound hammer are more satisfactory (more than established value as per prior relation established between rebound hammer and cube test results) and ultrasonic pulse velocity results are excellent, then core testing may be exempted at the discretion of engineer-in-charge.

Acceptance criteria for core tests will be as per **11.4.2.4**.

11.4.5 Durability Tests

Tests for durability such as electrical resistivity (as given in Annex D), RCPT, air permeability and oxygen permeability shall be conducted at site for quality control stage whichever is applicable in the mix design as per **9.5**.

11.4.5.1 Frequency of sampling

Frequency of tests shall be as per Table 97. At least one sample shall be taken from each shift. Where concrete is produced at continuous production unit, such as ready-mixed concrete plant, frequency of sampling shall be one sample for every 75 m³ of concrete.

For QA3 level, few samples shall be extracted from structure (or from the specific extra cast mock member or specimen kept at site under similar exposure conditions) and strength tests (rebound hammer and core test) to be conducted on them in addition to durability tests and results preserved for use as reference in future periodical investigations on durability and service-life assessment of the structure.

11.4.5.2 Specimen

For each test sample, three test specimens shall be prepared, and average result of the three specimens shall form one result.

11.4.5.3 Acceptance Criteria

Value obtained of each result shall not deviate by more than 10 percent (on lower durability side) of specified value, or the value obtained at mix design stage, whichever is on higher side of durability. Where there is no specified value, results shall be compared with values obtained at mix design stage only.

Table 97 Frequency of Test for Durability
(Clause 11.4.5.1)

<i>Quantity of Concrete in Work (m³)</i>	<i>Number of Samples</i>
Upto 100	1
> 100	1 additional sample for each additional 500 m ³ or part thereof

11.5 Tests on Concrete making Materials

All materials to be used for concrete making shall be tested periodically to ensure that they shall conform to the relevant specifications, and there is no significant deviation in critical parameters with respect to the values obtained at the time of mix design. When the deviation is significant, appropriate adjustments shall be made in concrete mix proportions. When the deviation is large, concrete mix shall be redesigned. The minimum frequency of various tests to be conducted on concrete making materials shall be as per Table 98.

Minimum testing of material required to generate data for mix design shall be carried out invariably at mix design stage, even if a particular test is not mandatory otherwise for a particular QA level.

Table 98 Measures in Three Levels of QA for Concrete Making Materials
(Clause 11.5)

Sl No.	Type of Test	QA1	QA2	QA3
(1)	(2)	(3)	(4)	(5)
Cement				
i)	Physical Test	Not required, except when there is a doubt	Once for every brand and type	All lots to be tested as under;

ii)	Chemical Test	Not required, except when there is a doubt	Not required	Required
Aggregates				
i)	Petrography and ASR Tests	Previous records may be used, or as in QA2	Once for every source	Once for every project for each source and every 500 m ³
ii)	Grading Test	Every 40 m ³ or part thereof	Every 40 m ³ or part thereof	Every 40 m ³ or part thereof
iii)	Chemical Test		Once for every source	Once for every source and every 200 m ³
iv)	Physical Test, other than Grading	Once for every source	Once for every source	Once for every source and every 200 m ³
v)	Silt content in fine aggregates	Every 20 m ³ or part thereof, or every stack	Every 20 m ³ or part thereof, or every stack	Every 20 m ³ or part thereof, or every stack
Water				
i)	Physical and Chemical Tests	For each source of water, one before commencement, and every 3 month thereafter (For potable For Municipal water source, one before commencement and every 6 months thereafter.)	As in QA1	One tests for each source every one month for all sources
Mineral Admixtures				
i)	Physical and Chemical Tests	Not required, if pre-approved for well known brand admixtures used	Same as in QA1; Manufacturer certificate for same lot is valid	Every lot to be tested

11.6 Quality Assurance in Concrete Production

QA requirements for control during concrete production shall be as per Table 99.

Table 99 Measures in Three Levels of QA during Concrete Production
(Clause 11.6)

Measures or Requirements	QA1	QA2	QA3
(1)	(2)	(3)	(4)
General	Site having: a) Proper storage of cement; b) Weigh batching of all materials; c) Controlled addition of water; d) Regular checking of all materials, aggregate grading and moisture content; and e) Periodical checking of workability and strength	Same as in QA1, with automatic weigh batching plant	Same as in QA2, with a) Moisture probes fitted to plant, b) Batching plant with bins, c) More frequent checking of grading and moisture content, d) Statistical quality control charts used to monitor variation from mean
Concrete Mix Design	Previous mix design (with same materials) can be used. In-house concrete mix design under supervision of owner or consultant permitted.	Same as in QA1, but project-specific site trials needed. Concrete mix design or verification of concrete mix design to be done and approved by third party. Proper mix design report with record of materials and	Project-specific mix design with site trials. Concrete mix design or verification of concrete mix design to be done and approved by third party. Report and record as in QA2.

		concrete test results.	
Site trials before start (to verify mix design at site)		Compressive strength of average of 4 samples not less than $f_{ck} + 1.65(\sigma - 1)$	Compressive strength of average of 4 samples, not less than target strength.
Variations in input materials (Concrete making materials)	Fine aggregate of same type and in same zone as used in mix design. Cement of same brand and type.	Same as in QA1, with variation in grading limits in material properties restricted (Refer Table 10.7)	Same as in QA2
Review of mix design	If variations in aggregate grading exceeds above limits, adjustment or revision of mix design needed. If cement brand different or strength issues, mix design to be revised.	Same as in QA1	Same as in QA2 Also, revise or review mix design, when: a) Periodical mean strength less than target strength, or b) Standard deviation observed more than assumed Root cause analysis required for such variations

Table 100 Acceptable limits on Variations in Routine Tests
(Table 11.6)

Sl No.	Material	Property	Acceptable Limits
(1)	(2)	(3)	(4)
i)	Cement	28-day compressive strength	-5%, +5%
		7-day compressive strength	-7%, +7%
ii)	Coarse aggregate	Specific gravity	- 0.055, +0.055

iii)	Fine Aggregate	Same zone with 600 micron passing	-8%, +5%
iv)	Fly Ash	45 micron passing	33%
		Fineness (Blaine's)	7%
v)	Chemical Admixture (Superplasticiser)	Specific gravity	1.5%
		Water reduction	5%
vi)	Concrete	Initial setting time	10%
		Final setting time	8%

11.7 Quality Assurance in Reinforcement Steel

11.7.1 Samples

The samples shall be selected and examined for each lot separately for ascertaining their conformity to the requirements of the relevant specifications.

All test pieces shall be selected either from cuttings of bars; or if desired, from any bar after it has been cut to the required or specified size, and the test piece taken from any part of it. The preparation of specimen shall be done in accordance with the IS ____.

11.7.2 Frequency

Sampling of reinforcing steel shall be done as per Table 101.

11.7.3 Test to be Conducted

Physical and chemical test shall be performed as per requirements of IS 1786.

11.7.4 Acceptance Criteria

The test results of the reinforcing steel shall conform to IS 1786. Further, some provisions of retesting are mentioned hereunder in case sample fails or rejected.

- If any tensile property of any specimen is less than that specified, and any part of the fracture is outside the middle-third of the gauge length (as indicated by scribe scratches marked on the specimen before testing), a retest shall be performed.
- If the results of an original tension specimen fail to meet the minimum requirements and are within 95 percent of the required tensile strength and yield point, a re-test shall be performed on two random specimens for each original tension specimen failure from the lot. If all results of these re-test specimens meet the specified requirements, the lot shall be accepted.
- If a bend test fails for reasons other than mechanical reasons or flaws in the specimen as described below, a re-test shall be performed on two random

specimens from the same lot. If the results of both test specimens meet the specified requirements, the lot shall be accepted.

- d) If a test specimen fails because of mechanical reason (such as failure of testing equipment or improper specimen preparation), it shall be discarded and another specimen to be taken.

Table 101 Sampling in Three Levels of QA for Reinforcement Steel
(Clause 11.7.2)

Diameter of bars (in mm)	QA1	QA2	QA3	
(1)	(2)	(3)	(4)	
Bars of any diameter	-	At least once for all types, diameters and brands; Manufacturer's Test Certificate obtained and kept in record for all lots	Manufacturer's Test Certificate as in QA2	
			Testing of lots for both physical and chemical as under, for consignment of	
			≤ 100 Tonnes	> 100 Tonnes
< 10	-	-	1 sample for every 25 tonnes	1 sample for every 40 tonnes
10 – 16	-	-	1 sample for every 35 tonnes	1 sample for every 45 tonnes
> 16	-	-	1 sample for every 45 tonnes	1 sample for every 50 tonnes

11.8 Quality Assurance in Prestressing Steel and Grout Work

11.8.1 Tests and Frequency

The tests to be conducted along with their frequency shall be as per Table 102.

11.8.2 Acceptance Criteria

All prestressing steel and other items shall meet the relevant specifications.

Table 102 Sampling in Three Levels of QA in Prestressed Concrete
(Clause 11.8.1)

Sl No.	Prestressing Elements	Tests	QA1	QA2	QA3
(1)	(2)	(3)	(4)	(5)	(6)
i)	Prestressing Cables and Strands	Physical Test to obtain modulus of elasticity, and relaxation test Chemical Test	Manufacturer's Test Certificate	Manufacturer's Test Certificate as in QA1 with additional random testing of 1 sample in every 50 MT Relaxation Test with stress-strain curve can be at an approved independent laboratory or manufacturers laboratory with Project QA Team witnessing it	As in QA2, with Independent Third Party Agency witnessing the Tests
ii)	Anchorage Assembly	Anchorage Assembly Tests	To be taken from approved proven supplier with Manufacturer's Test Certificate confirming compliance to specified requirements w.r.t. following tests (1) Static load test with tendon-anchorage. (2) Dynamic load test with tendon-	As in QA1, with tests at approved independent laboratory or manufacturers laboratory with Project QA Team witnessing it.	As in QA2, with Independent Third Party Agency witnessing the Tests Details of testing to be part of QA Plan

			anchorage assembly. (3) Load transfer test		
iii)	Sheathing Ducts		Manufacturer's Test Certificate from approved established or BIS certified suppliers	Manufacturer's Test Certificate with additional random testing from approved laboratory	As in QA2, with Independent Third Party Agency witnessing the Tests
iv)	Grouts		Manufacturer's Test Certificate from approved established or BIS certified suppliers	Manufacturer's Test Certificate with additional random testing from approved laboratory	As in QA2, with Independent Third Party Agency witnessing the Tests

11.9 Testing of Structures / Structural elements

The tests specified hereunder can be performed on the structural elements to assess quality of concrete used in the structure and/or the load carrying capacity or response of structure to static or dynamic loads. Testing of structures can be done by partially destructive tests like concrete core tests or non-destructive tests.

In case of doubt regarding the grade of concrete used either due to poor workmanship or based on the results of cube strength, tests like concrete core tests or non-destructive tests can be carried out on the basis of and/or load test (see).

Non-destructive tests are used to estimate the properties of concrete in the structure. It provide alternatives to core tests for estimating the strength of concrete in a structure, or can supplement the data obtained from a limited number of cores. These methods are based on measuring a concrete property that bears some relationship to strength. The accuracy of these methods, in part, is determined by the degree of correlation between strength and the physical quality measured by them.

11.9.1 Non-destructive Tests (NDTs)

Non-destructive tests (NDTs) shall be carried out on the structure, either as part of QA measure (refer 11.4) or when the grade of concrete is suspected to be deficient due to poor workmanship observed visually or small cube strengths obtained from compressive strength tests of concrete. NDTs shall be used prior to performing core tests for estimating select properties of concrete structures, or for supplementing the data obtained from a limited number of core tests.

Results of NDTs are only indicative and not confirmatory. Also, where both core tests and non-destructive tests have been conducted, results of core tests alone shall be used for assessment of strength. Correlation between results of core tests and non-destructive tests developed for one project shall be not applicable for other projects.

11.9.1.1 *Tests for estimation of strength and in place quality of concrete*

The methods adopted include following tests. Any of these methods (or a suitable combination) may be adopted, in which case the acceptance criteria shall be agreed upon prior to testing.

- a) Ultrasonic Pulse Velocity Test as specified in IS 516 (Part 5/Sec 1)
- b) Rebound Hammer Test as specified in IS 516 (Part 5/Sec 4)
- c) Probe Penetration Test
- d) Pullout Test as specified in IS 2770 (Part 1)
- e) Maturity Test
- f) Cover or Rebar Locator Test

11.9.2 *Concrete Core Test*

Core tests shall be carried out on the structure, either as part of QA measure (refer 11.4) or when the grade of concrete is suspected to be deficient due to poor workmanship observed visually or small cube strengths obtained from compressive strength tests of concrete. Cores shall be extracted, prepared and tested as per IS 516 (Part 4) to determine compressive strength and in-situ quality of concrete.

Core tests shall be conducted on a limited number of samples; these shall be supported by NDT tests. In all cases, results of core tests alone shall be final and binding, notwithstanding those of the non-destructive tests. The locations from where cores are to be taken and the number of cores required shall be at the discretion of the engineer-in-charge, and shall be representative of the whole of the concrete work of concern. In any case, the number of cores taken shall not be less than three.

11.9.2.1 *Acceptance criteria of core test results*

11.9.2.1.1 Concrete in the structural member represented by core test samples shall be considered acceptable, if:

- a) the individual member strength is to be determined or where one set of cubes (set of 4 consecutive samples) has failed. The core test results shall be considered satisfactory if average of equivalent cube strength of a minimum of three cores is more than $0.85f_{ck}$ and no individual core has equivalent cube strength less than $0.75f_{ck}$.
- b) If a large number of cube sets (each set consisting of 4 consecutive samples) has failed for the same grade of concrete within a batch to be assessed, say 2 consecutive sets or 3 scattered sets have failed. The core test results shall be considered to be satisfactory if average of equivalent cube strength of a

minimum 10 cores is more than $0.85(f_{ck} + X/2)$ and no individual core has equivalent cube strength less than $0.75f_{ck}$, where average means the average equivalent cube strength of all cores (minimum nine cores after excluding outliers, if any). Out of every 10 cores tested, maximum one core test result can be outlier. Outlier can be detected by:

- 1) inspecting the load-machine displacement curves, or
- 2) using statistics of results of core tests, if individual values are beyond $f_{ck} \pm 2\sigma$.

11.9.2.1.2 Where the requirements of (a) and (b) above as the case may be have been met, the concrete can be said to comply with requirements of IS 456. But where 10 or more cores are taken but the results do not meet the criteria given in (b) above, but results of average of cores from all the individual members tested have strength more than $0.85f_{ck}$ and no individual core has strength less than $0.75f_{ck}$, the concrete in the particular batch or member may be accepted for structural adequacy and contractual acceptance/penalties/deduction may be decided as per contract provisions by the project authorities.

11.9.2.1.3 When the core test results do not satisfy the above requirements, or when such tests could not be performed, load test (as per **11.9.3**) may be performed on the structure to ascertain its load carrying capacity.

11.9.2.1.4 Where there is doubt on strength, core tests could not be performed and load test has been carried out, additional NDTs shall be carried out to assess the strength and durability of concrete.

11.9.2.1.5 Normally, though, there is a gain of strength beyond 28 days, the quantum of increase depends upon the type and grade of cement, method of curing and prevalent environmental condition. Therefore, while assessing the strength of concrete based on cores extracted at a later age, no increase in strength of concrete shall be considered, except where considered in design or included in the contract specifications.

11.9.2.1.6 Tests as specified in 7 and in sections above shall be used to establish the properties of reinforcement and concrete making materials.

11.9.3 *Load Tests*

11.9.3.1 In case of doubt regarding the grade of concrete used either due to poor workmanship or based on results of cube strength tests or core tests, load tests may be carried out as specified hereunder. Load Tests shall be performed on

- a) Flexural Members of concrete structures that are suspect of poor quality/workmanship. Other members shall be investigated by detailed structural analysis using nonlinear finite element method.
- b) Whole structure that is designed on the basis of being approved only after the whole structure is tested after construction.

11.9.3.2 The following procedure shall be followed for conducting the load test on a structure:

- a) The load test shall be performed only after 28 days from the time of placing of concrete.
- b) The structure shall be subjected to the following load combinations for a period of 24 hours equal to:
 - 1) *RC Structure*: $(\gamma_L)_{DL}DL + 1.25(\gamma_L)_{LL}IL$, and
 - 2) *PSC Structure*: $(\gamma_L)_{DL}DL + (\gamma_L)_{LL}IL$,

and then the IL shall be removed, where DL (namely Dead Load, which includes self-weight of the structural members plus weight of finishes, walls or partitions, if any, as considered in the design) and IL (namely Imposed Load) shall be taken as specified in 5.1.1 and 5.1.2, respectively, and $(\gamma_L)_{DL}$ and $(\gamma_L)_{LL}$ are the partial safety factor for loads specified in Table 3;

- c) The deflection due to imposed load only shall be recorded. Within 24 hours of removal of IL, if the structure does not recover at least:
 - 1) *RC Structure*: 75 percent of the deflection under imposed load, then the test may be repeated after a lapse of 72 hours. Even then, if the recovery is less than 80 percent of the deflection under imposed load, the structure shall be deemed to be unacceptable.
 - 2) *PSC Structure*:
 - i) *Type 1 and Type 2 Structures*: 85 percent of the deflection under IL, then the test may be repeated after a lapse of 72 hours. Even then, if the recovery is less than 90 percent of the deflection under IL, the structure shall be deemed to be unacceptable.
 - ii) *Type 3 Structures*: 75 percent of the deflection under IL, then the test may be repeated after a lapse of 72 hours. Even then, if the recovery is less than 80 percent of the deflection under IL, the structure shall be deemed to be unacceptable.
- d) If the maximum deflection (in mm) observed during 24 hours under load is less than $40l^2/D$, where l is the effective span (in m); and D the overall depth of the section (in mm), it is not necessary for the recovery to be measured and the recovery provisions mentioned in (c) above shall not apply.
- e) The structure shall satisfy deflection as well as crack width requirements. The crack width measured in member(s) shall be less than the limits specified in 9.4.1.3.

11.9.4 *Vibration Tests*

Vibration test may be performed to ascertain the dynamic characteristics of an existing concrete structure. When doing so, accelerations, velocity and/or displacement sensors of high sensitivity shall be placed at critical locations on the structure and oriented along the directions in which the shaking is experienced to be dominant. The critical locations and orientation of sensors shall be decided by the engineer-in-charge.

11.9.4.1 *Ambient vibration test*

No external oscillation or disturbance is employed to vibrate the structure. The inherent vibrations are sufficient for recording the low inherent oscillations.

11.9.4.2 *Forced vibration test*

Specifically designed electro-dynamic shakers or mechanical mass shakers shall be employed to induce vibrations in the structure. When doing so, the following precautions shall be taken:

- a) They shall be secured at any location in the structure, but preferably at the topmost surface of the structure.
- b) They shall be used with an electrical controller so that the frequency of shaking is maintained precisely at a desired value.

11.9.5 *Impact Tests*

This test is to be performed to ascertain the dynamic characteristics of the structure. These tests may cause damage either locally or globally and to be performed (preferably on prototypes) with due caution and only where required and decided by engineer-in-charge to be carried out.

In this test, a large mass suspended as a pendulum shall be dropped from a predetermined height to cause external impulse at a predetermined location of the structure. The resulting response of the structure shall be measured using digital sensors with time sampling of data with micro-second resolution. The mass of the pendulum to be dropped, and the height from which the mass is to be dropped such that the impact is minimal enough to cause a measurable impact shall be decided by engineer-in-charge.

12 SPECIAL CONCRETES

12.1 High Performance Concrete

High performance concrete is a concrete whose ingredients, proportions and production methods are specifically chosen to meet specific performance and uniformity requirements. The specific higher performance requirements may be in terms of high Durability requirements, high abrasion/erosion resistance, higher strength or higher resistance to shrinkage etc. High strength concrete generally improves performance of all above parameters. The concrete mix proportioning for high strength concrete may be carried out in accordance with the good practice.

However special measures are needed to further improve the specific performance requirements. For further details, expert literature may be referred.

12.2 Steel Fiber Reinforced Concrete (SFRC)

Steel fiber reinforced concrete is a composite material having fibers as the additional ingredients, dispersed uniformly at random in percentages between 0.3 percent and 2.0 percent by volume in plain concrete.

12.2.1 *Flexure and Shear*

Where SFRC is used to improve properties other than flexure and shear, design for flexure and shear shall generally be done ignoring effect of steel fibres and no reduction of flexure and shear reinforcement will be made.

Where SFRC is adopted specifically to take part of flexure and shear to reduce reinforcement congestion, specific approval of engineer in charge is to be obtained and design is to be carried out using expert literature and international practices for similar type of structure. Design will be based on strain softening SFRC only and strain hardening effect will not be considered for flexure and shear design. SFRC flexure and shear Design Provisions given in our National Building Code may also be referred in such cases.

12.2.2 *Properties of SFRC*

SFRC shall be classified in terms of both its characteristic compressive strength and its characteristic residual tensile strength as given in Annex E.

12.2.2.1 *Compressive strength*

The characteristic compressive strength of SFRC at 28 days shall be determined in accordance IS:516 (Part-I) Sec-1-2021.

12.2.2.2 *Tensile Properties*

a) Classification

SFRC shall be classified as either softening or hardening as shown in Fig. 88. A hardening SFRC is outside the scope of this code.

NOTES

1. A hardening material is defined as one with a tensile strength equal to or greater than 1.1 times the strength of the matrix without fibres and taken at a crack opening displacement (COD) of equal to or greater than 0.3 mm.

2. A hardening material is one where multiple cracks can occur prior to crack localization. For material hardening to occur, the fibre dosage is typically such that the strength of the SFRC is at least 10 percent greater than the strength of the same concrete mix without fibres. The value of COD of 0.3 is sufficiently high such that matrix contribution to the strength of the composite is minimal and the tension at this point is substantially taken by the fibres alone.

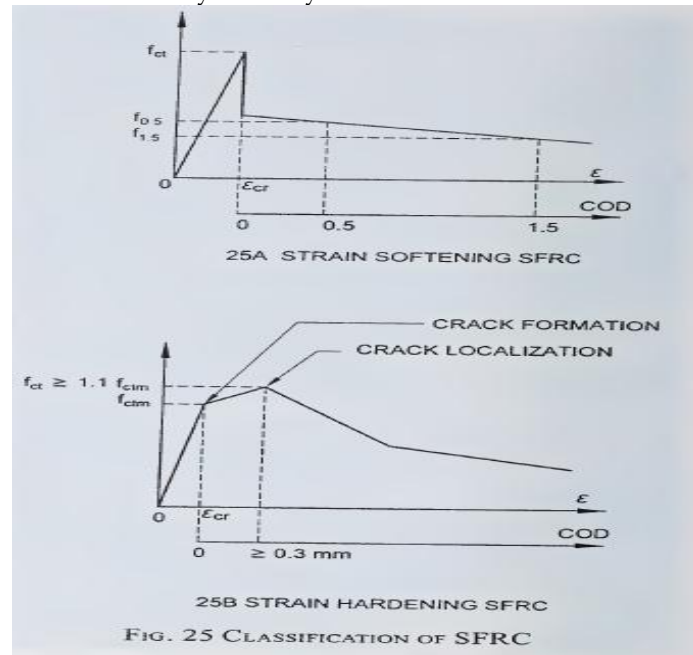


FIG. 88 Classification of SFRC

b) Flexural Tensile Strength

The flexural tensile strength of the softening SFRC shall be obtained as per IS 516 (Part 1/Sec 1). The characteristic flexural tensile strength of SFRC shall be calculated in accordance as follows:

$$f_{cr} = 0.7 \sqrt{f_{ck}}$$

where,

f_{cr} flexural tensile strength, in N/mm²; and

f_{ck} = characteristic compressive strength of concrete, in N/mm².

NOTE — For strain softening SFRC, the peak strength of the composite may be approximated as the strength of the concrete mix without fibres.

c) Residual Tensile Strength

The standard characteristic residual tensile strength grades are 0.4 MPa, 0.6 MPa, 0.8 MPa, 1.2 MPa, 1.6 MPa and 2.0 MPa. The characteristic residual tensile strengths of concrete at 28 days ($f'_{1.5}$) shall be determined statistically from tests carried out in accordance 12.1.3.3.4.

NOTE - The characteristic residual tensile strength depends on the fibre-matrix bond strength, which is usually a function of the compressive strength of the parent concrete, as well as the fibre content.

12.2.2.3 Determination of strength by direct testing

The characteristic residual tensile strength ($f'_{1.5}$) shall be obtained using direct tensile tests as specified in Annex E.

12.2.2.4 *Residual tensile strength-residual flexural strength relationship*

Alternatively, where matched direct and indirect testing (residual flexural tensile strength test) has been undertaken in accordance with Annex E for similar SFRC mixtures, the characteristic residual tensile strength may be determined as:

$$f'_{1.5} = k_{R4} \cdot f'_{R4}$$

Where f'_{R4} is determined in accordance with appendix... and calculated statistically, and the factor k_{R4} determined from Annex E.

For the purposes of this clause, similar SFRC mixtures are defined as having the same,

- a) fibre type and content,
- b) water to cementitious material ratio,
- c) maximum aggregate particle size, and
- d) compressive strength (f'_c).

The relationship between residual tensile strength and the residual flexural strength shall be obtained by matched tests using the same SFRC mixture. Residual tensile strength specimens shall be prepared and tested in accordance with Annex E. Residual flexural tensile tests shall be in accordance with the procedure given in Annex E.

NOTE — The residual tensile strength (as obtained from a direct tensile test) is a fundamental material property of SFRC. This clause allows the concrete producer and steel fibre manufacturer to correlate the direct tensile strength of their particular SFRC mix design with the flexural tensile strength.

12.2.2.5 *Determination of strength by indirect testing*

The characteristic residual tensile strength ($f'_{1.5}$) may also be obtained using indirect tests (flexural residual tensile strength test) and calculated as:

$$f'_{1.5} = 0.4f'_{R4} - 0.07f'_{R2}$$

12.2.2.6 *Modulus of elasticity*

The modulus of elasticity at the appropriate age (E_c) be determined in accordance with as per IS 516 (Part 1/Sec 1).

NOTE The addition of steel fibres to concrete in conventional dosages (100 kg/m³ and less) does not change the elastic modulus of the concrete.

12.2.2.7 Durability

The minimum concrete grade and cover for SFRC in respective exposure classification shall be as for concrete without fibres and shall apply to the steel reinforcement only. SFRC shall not be used in exposure classification C1 or C2. (needs to be correlated with durability exposure classes in durability chapter)

NOTES

- 1 Steel fibres do not require concrete cover as specified for steel reinforcement.
- 2 SFRC may not be suitable in some exposure classification of environments.
- 3 Fibres close to the surface are subject to corrosion and may result in surface staining. This will not impact durability.

12.2.3 Production of SFRC**12.2.3.1 Fibres**

Steel fibres shall comply with accepted standard. The 'Certificate of Conformity' shall be supplied on request to the relevant authority.

NOTE – Till the availability of a separate Indian Standard Specification for Steel Fibres, reference may be made international literature.

12.2.3.2 Mixing of Fibres

Fibres of the type and quantity specified shall be added in a controlled process ensuring that they are dispersed uniformly through the concrete mix. If added after the main mixing process, the concrete shall be remixed until the fibres have been completely dispersed throughout the batch.

NOTES

- 1 Balling of fibres should be avoided. Visual inspection of mixed concrete for balling of fibres should be done and if ball formation noticed the batch should be discarded.
- 2 A record of fibre content should be recorded for each batch.

12.2.3.3 Determining the steel fibre content

Steel fibre content shall be measured from samples taken from the production concrete, by volume of concrete. In addition,

- a) A sample shall be taken from the batch of concrete at unloading from the first third, middle third and final third of the batch;
- b) Each sample shall be a minimum of 10 litre;
- c) The sample container shall be filled in one continuous pour and where possible directly from the discharge chute; and
- d) Wash out or magnetic separation only shall be used.

The conformity of the correct steel fibre dosage is proven if the criteria in Table 103 are met.

Table 103 Criteria of Acceptance for Steel Fibre Dosage
(Clause 12.2.3.3)

CRITERIA OF ACCEPTANCE FOR STEEL FIBRE DOSAGE

Frequency	Test control	Criteria
Each sample	Each partial test	≥ 0.80 of the specified target dosage
Average of three samples from the batch	Each test	≥ 0.80 of the specified target dosage
Continuous control: average of >3 tests	Continuous control: average of >3 tests	≥ 0.90 of the specified target dosage

12.3 Structural Light Weight Concrete

12.3.1 General

Structural Light weight concrete shall be produced using light weight aggregates.

Lightweight aggregates are natural or manufactured with in-place density (unit weight) between 1440 to 1840 kg/m³. The lightweight is due to the cellular or high internal porous structure, which gives this type of aggregate a low specific gravity. They have high absorption values, which requires a modified approach to concrete proportioning for specific strength and durability requirements.

12.3.2 Light weight aggregates namely sintered fly ash aggregates shall only be used for making light weight structural concrete (a type of special concrete). However use of lightweight aggregates for structural concrete should be limited to normal structures up to four storey height only. Its use in prestressed concrete and special structures like chimneys and cooling towers etc. should be avoided. Other light weight aggregates may be used only for non structural purpose with permission of competent project authority only.

NOTE For design and durability considerations using any other light weight aggregate, specialist literature should be referred.

12.3.3 Guidelines for use of Sintered Fly Ash Lightweight Coarse Aggregate in Structural Concrete

Sintered flyash coarse aggregates when used shall conform to IS 9142 (Part 2).

The combination of lightweight fine aggregate and normal weight fine aggregate together in concrete is not recommended.

12.3.3.1 Mechanical properties of sintered fly ash lightweight aggregate concrete

12.3.3.1.1 Lightweight concrete up to grade M35 and below may be made and used for structural use in non-wearing surfaces. There is normally little gain of strength beyond 28 days. The design should be based on 28 days characteristic strength of concrete. The flexural and splitting tensile strengths shall be obtained as described in IS 516 and IS 5816, respectively. For estimating the tensile strength from the compressive strength, the following formula may be used:

$$\text{Flexural strength, } f_{cr} = 0.6 \sqrt{f_{ck}} \text{ N/mm}^2$$

where, f_{ck} is the characteristic cube compressive strength of concrete in N/mm^2

12.3.3.1.2 The modulus of elasticity is primarily influenced by the elastic properties of the aggregate. The modulus of elasticity of concrete can be assumed as follows:

$$E_c = 3\,000 \sqrt{f_{ck}}$$

where E_c is the short term static modulus of elasticity in N/mm^2 . Actual measured values may differ by ± 20 percent from the values obtained from the above expression.

12.3.3.1.3 Poisson's ratio of 0.20 may be usually assumed for practical design purposes.

12.3.3.1.4 The linear thermal expansion coefficients for lightweight concrete ranges from 7 to 11 $\times 10^{-6}$ mm/mm/°C depending on the amount of natural sand used.

12.3.3.2 Durability of lightweight aggregate concrete

One of the main characteristics influencing the durability of concrete is its permeability to the ingress of water, oxygen, carbon dioxide, chloride, sulphate and other potentially deleterious substances. Light weight concrete generally have higher permeability compared to normal structural concrete. The factors influencing durability include

- a) Environment;
- b) Cover to embedded steel;
- c) Type and quality of constituent materials;
- d) Cement content and water/cement ratio of the concrete;
- e) Workmanship, to obtain full compaction and efficient curing; and
- f) Shape and size of the member.

Use of lightweight concrete is not recommended in and exposure classes.

12.3.3.2.1 *Exposure to sulphate attack*

Recommendations for the type of cement, minimum cement content and other precautions in case of exposure to sulphate attack shall be as per 9.5. However, the maximum free water/cement ratio shall be further low by 0.05 in all classes.

12.3.3.2.2 *Requirement of concrete cover*

The protection of the steel in concrete against corrosion depends upon an adequate thickness of good quality concrete. The nominal cover to the reinforcement to meet the durability requirement in case of lightweight concrete shall be increased by 10 mm over and above as recommended in this standard.

12.3.3.3 *Concrete Mix Proportions*

The free water-cement ratio is an important factor in governing the durability of concrete and should always as low as possible. Appropriate values for minimum cement content given in this standard for different exposure conditions shall also be applicable for lightweight concrete. However, the maximum free water-cement ratio given in this standard shall be further reduced by 0.05 in all exposure classes.

The mix design shall be such that the concrete mix is viscous enough to avoid floating of aggregates during compaction.

12.3.3.4 *Other aspects*

Specialist literature should be consulted to take into consideration the important aspects of design such as development length, durability, cover for fire resistance, shear and torsion resistance of beams, deflection of beams, additional moments in slender columns etc. when lightweight aggregates are used.

12.4 Polymer Modified Concrete

12.4.1 General

12.4.1.1 Polymer-modified cementitious mixtures, also called polymer modified concrete (PMC) or polymer portland-cement concrete (PPCC) and latex modified concrete (LMC). They are defined as hydraulic cement combined at the time of mixing with organic polymers that are dispersed or redispersed in water, with or without aggregates.

12.4.1.2 The organic polymer is supplied in three forms: as a dispersion in water that is called latex; as a redispersible powder; or as a liquid that is dispersible or soluble in water. Dispersions of polymers in water and redispersible polymer powders have been in use for many years as admixtures to hydraulic-cement mixtures. These admixtures are called polymer modifiers. The dispersions of these polymer modifiers are called latexes.

12.4.1.3 *Types of Polymer*

- a) *Latexes* – Much of the PMC placed today uses a polymer in latex form. Latex is defined as a dispersion of organic polymer particles in water. The average particle size varies from 3.9×10^{-6} to 78×10^{-6} (100 to 2000 nm). Most latexes are made by a process known as emulsion polymerization, where the polymer particles are formed in water. The surfactants used to stabilize these latexes are usually non-ionic, such as polyvinyl alcohol or alkyl phenols reacted with ethylene oxide.
- b) *Polyvinyl acetate* – This type of latex should not be used in PMC because of the poor water resistance.
- c) *Vinyl acetate copolymers* – Typical comonomers are ethylene, BA, and the vinyl ester of versatic acid (VEOVA). The incorporation of these comonomers reduces the tendency of the vinyl acetate to hydrolyze, and the reaction products are less water soluble.
- d) *Acrylic esters and copolymers (PAE and S-A)* – These latexes have been used for many years, primarily in tile adhesives, floor overlayments, stuccos, exterior insulation finish systems (EIFS), and repair of concrete. Acrylic esters are the latex of choice where colour and colour fastness are important
- e) *Styrene-butadiene copolymers (S-B)* – This type of latex is widely used where either adhesion or water resistance is required, and providing colour fastness is not a requirement.
- f) *Redispersible powders* – A substantial portion of PMC uses polymers in powdered form that redisperse when mixed with water or aqueous mixtures. Typically, the polymer powder is premixed with the cement and aggregates, and water is added at the job site. This operation allows for more accurate mixture proportioning and is more convenient than using a two-component latex system.
- g) *Epoxy resins* – Epoxy resins have been used as polymer modifiers for Portland cement mixtures.

12.4.2 *General requirements of polymer modifier to be used in concrete*

PMC to be used should meet the specific requirements with respect to improvements in particular property of concrete, like improvement in adhesion, water resistance, resistance to chloride ingress or reduced drying shrinkage etc. Relevant tests shall be conducted to ensure the improvement is as per contract specification / claimed by manufacturer of the polymer modifier.

In addition to above, the polymer modifier shall produce polymer modified concrete (PMC) which conforms to the basic requirements as listed in Table 104. Proportioning for making test concrete and reference concrete will be as per **12.4.3**.

Table 104 Basic Requirements for Concrete made with Polymer Modifier
(Clause 12.4.2)

Sl No.	Characteristic	Requirements	Method of test
(1)	(2)	(3)	(4)
i)	Air Content, % maximum	7.0	IS 1199 Part4
ii)	Time of Setting, allowable deviation from reference mixture*, hr:min:		IS 1199 Part7
	Initial:		
		-1:00	
		+3:30	
	Final:		
		-1:00	
		+3:30	
iii)	Compressive Strength, minimum % of reference mixture at each age	80	IS 516

12.4.3 Proportioning of concrete for test concrete and reference concrete

- Make the test and reference concrete with proportions similar to those shown for a nominal cubic-meter batch.
- Make the test concrete conforming to the following requirements:

Materials	Parts by Mass, kg
Ordinary Portland cement	390 ± 3
Fine aggregate (saturated surface dry)	975 ± 6
Coarse Aggregate (saturated surface dry)	780 ± 6
Polymer Modifier	The level recommended by the manufacturer.
Water	The amount of water to give a slump of 90 ± 15.

c) Make the reference concrete to give an air content within 2 percent of the test concrete and not exceeding 7 percent, conforming to the following requirements:

<i>Materials</i>	<i>Parts by Mass, kg</i>
Ordinary Portland cement	390 ± 3
Fine aggregate (saturated surface dry)	975 ± 6
Coarse Aggregate (saturated surface dry)	780 ± 6
Potable water	The amount of water to give a slump of 90 ± 15 .

d) The concrete shall be made in a mechanical power-driven mixer and shall be compacted as per IS 1199 Part 5.

12.4.4 Mixture proportioning

The proportioning of ingredients and mixing procedures are similar to those for unmodified mixtures. Specific mixture proportioning depends on the particular application and the type of polymer being used. In general, polymer levels of 10 to 25 percent by mass of the hydraulic cement are required for optimum performance. The w/cm of PMC is typically 0.30 to 0.40 parts by mass. These low ratios result from the water-reducing qualities attributable to the polymers. When latexes are used, the water content of the latex should be included when calculating the w/cm.

12.4.5 Curing

The curing regime for polymer modified concrete requires initial moist curing to prevent plastic-shrinkage cracking, followed by air curing. PMC specimens should be immediately covered with polyethylene bag/ plastic sheet and store at $27 \pm 2^\circ\text{C}$ for 24 hours. Thereafter, polyethene bag cover/ plastic sheet should be removed and the specimens should be stored in laboratory environment at $27 \pm 2^\circ\text{C}$ and RH 65±5 percent (that is air curing) until age of testing.

12.5 Self-Compacting Concrete

Self-compacting concrete (SCC) is highly flowable, non-segregating concrete that fills uniformly and completely every corner of formwork by its own weight and encapsulate reinforcement without any vibration, whilst maintaining homogeneity. The mix proportioning of SCC can be done as per IS 10262.

12.5.1 Features of fresh Self Compacting Concrete

A concrete mix can only be classified as self-compacting concrete if the requirements for all below mentioned characteristic are fulfilled:

- a) Filling ability (Flowability),
- b) Passing ability,
- c) Segregation resistance, and
- d) Viscosity

The above tests shall be carried out as per IS 1199 (Part 6).

a) *Filling Ability (flowability)*

This is the ability of fresh concrete to flow into and fill all spaces within the formwork, under its own weight. Slump-flow test is performed to test the flowability. Slump-flow value describes the flowability of a fresh mix in unconfined condition. Visual observation during the test can be additional information on the segregation resistance and uniformity. The following are typical slump-flow classes for a range of applications:

- 1) *SF1 (slump flow 550 mm - 650 mm)*: This class of SCC is appropriate for:
 - i. Unreinforced or lightly reinforced concrete structures that are cast from the top with free displacement from the delivery point (for example, housing slabs)
 - ii. Casting by a pump injection system (for example, tunnel linings)
 - iii. Sections that are small enough to prevent long horizontal flow (for example, piles and some deep foundations).
- 2) *SF2 (slump flow 660 mm - 750 mm)*: This class of SCC is suitable for normal applications (for example, walls, columns).
- 3) *SF3 (slump flow 760 mm - 850 mm)*: This class of SCC is used for vertical applications in heavily reinforced structures, structures with complex shapes, or for filling under formwork. SF3 will often give better surface finish than SF 2 for normal vertical applications but segregation resistance is more difficult to control.

b) *Passing Ability (free from blocking at reinforcement)*

Passing ability describe the capacity of the fresh mix to flow through confined spaces and narrow openings such as areas of congested reinforcement without segregation. If there is little or no reinforcement, there may be no need to specify passing ability as a requirement. L-box test is performed to check the passing ability. The minimum ratio of the height in the horizontal section relative to the vertical section is considered to be 0.8. If the SCC flows as freely as water, it will be completely horizontal, and the ratio will be equal to 1.0.

c) *Segregation Resistance (Stability)*

The ability of fresh concrete to remain homogeneous in composition while in its fresh state. Segregation resistance (sieve) test is performed to check this

property of fresh concrete. After sampling, the fresh concrete is allowed to stand for 15 min and any separation of bleed water is noted. The top part of the sample is then poured into a sieve with 4.75 mm square apertures. After 2 minute, the weight of material which has passes through the sieve is recorded. The segregation ratio (SR) is then calculated as the proportion of the sample passing through the sieve.

There are two classes of segregation resistance, namely SR1 & SR2. SR1 is generally applicable for thin slabs and for vertical applications with a flow distance of less than 5 m and a confinement gap greater than 80 mm. SR2 is preferred in vertical applications if the flow distance is more than 5 m with a confinement gap greater than 80 mm in order to take care of segregation during flow. Segregation resistance becomes an important parameter with higher slump-flow classes and/or the lower viscosity class, or if placing conditions promotes segregation. If none of these apply, it is usually not necessary to specify a segregation resistance class. For SR1 class segregation resistance should be 15 to 20 percent and for SR2 it should be less than 15 percent.

d) Viscosity

Viscosity can be assessed by the V-funnel flow time as per IS 1199 (Part 6). Concrete with a low viscosity will have a very quick initial flow and then stop. Concrete with a high viscosity may continue to creep forward over an extended time.

A V-shaped funnel is filled with fresh concrete and the time taken for the concrete to flow out of the funnel is measured and recorded as the V-funnel flow time. The viscosity is divided into two classes, that is, V1 & V2. V1 has good filling ability even with congested reinforcement. It is capable of self-leveling and generally has the best surface finish. V2 class viscosity is more likely to exhibit thixotropic effects, which may be helpful in limiting the formwork pressure or improving segregation resistance. But it may cause negative effects on surface finish and sensitivity to stoppages or delays between successive lifts.

For V1 class, the time taken to pass the concrete from V-funnel should be ≤ 8 s and for V2 class the time taken to pass the concrete from V-funnel should be between 9 s to 25 s.

12.5.2 Physical Properties of SCC

Physical properties of SCC are generally same as of normal concrete except some properties like shear, creep and shrinkage for which relevant clauses of this code and specialist literature should be referred.

12.6 Roller Compacted Concrete

Roller-compacted concrete (RCC) is a concrete of no-slump consistency in its unhardened state that is transported, placed, and compacted using vibratory roller.

Roller-Compacted Concrete (RCC) can be considered as both a construction material and a construction method. It is mainly used in mass concrete, roadway and paving applications. The layer thickness can vary from 150 mm to 300 mm. It should be ensured that equipment used for compaction is capable of properly compacting the adopted layer thickness.

12.6.1 *Mix Design*

12.6.1.1 Mix design for RCC is totally different from the design of mix for a conventional cement concrete pavement as the Abrahm's water/ cement ratio law does not hold good. RCC is a no-slump concrete.

12.6.1.2 The mix shall be proportioned by weight of all ingredients such that the desired target mean strength is achieved. The moisture content shall be selected so that the mix is dry enough to support the weight of a vibratory roller, and yet wet enough to permit adequate distribution of paste throughout the mass during mixing, laying and compaction operations. The water content by weight may be in the range of 4 -7 per cent and shall be determined by trial mixes having water content changing at intervals of 0.5 percent. The optimum water content is determined using a loaded vee bee test apparatus or Modified proctor density test. Loaded Vee Bee time for RCC to be used in dams is generally in the range of 20±10 seconds and for pavements a the range of loaded veebee time is 50+10 sec. Wet Density achieved shall also be determined and compared with theoretical air free density. For pavements where RCC is used for top layer and no wearing course is proposed, a drier mix is preferred which gives less undulations. In this case Modified proctor test is preferred over loaded vee beetest to determine Optimum water content. The coarse aggregates by volume shall be in the range of 52 - 56 percent and the volume of sand shall be varied to optimize the strength by trials. The fly-ash/ cement ratio and the water/cementitious material (cement + fly ash) ratio shall be selected so as to achieve the required strength. Fly-ash quantity shall be so proportioned that it replaces cement to an extent of 20 - 50 per cent by weight.

12.6.2 *Construction*

12.6.2.1 *General*

Prior to starting the main construction works, it is strongly recommended that the proposed RCC is mixed in the concrete plant and then placed, spread and compacted in a full-scale trial using the same equipment and procedures intended for construction. A full-scale trial can also be used to visually examine the segregation potential of the RCC mix under the specific site conditions, the condition of the lift surfaces, the necessary treatment for those surfaces and any other aspects of construction that may require review.

12.6.2.2 *Batching, mixing, transportation, laying, compaction and curing*

12.6.2.2.1 The batching and mixing shall be done in a batching plant capable of proportioning the materials by weight each type of material being weighed separately. The capacity of the batching and mixing plant shall be at least 25 percent higher than the proposed capacity for laying the concrete for structure. The batching and mixing shall be carried out preferably in a forced-action central batching and mixing plant having necessary automatic controls to ensure accurate proportioning and mixing. The weighing balances shall be calibrated by weighing the aggregates, cement, fly-ash and water on a large weighing machine.

12.6.2.2.2 Concrete shall be discharged from the mixer and transported directly to the point where it is to be laid. The capacity of the transport equipment shall match the production capacity of the batching plant and the laying equipment to ensure construction to go on at a uniform speed in an uninterrupted manner.

12.6.2.2.3 The concrete shall be laid by a paver, with electronic sensors. The paver shall be capable of laying the material in one layer in an even manner without segregation.

12.6.2.2.4 Compaction shall be carried out immediately after the material is laid and levelled. Rolling shall be done with a vibratory roller of the double drum smooth wheel type. The number of passes required to obtain the maximum compaction shall be determined during the trial run by increasing the in-situ density. In addition to the number of passes required for compaction, there shall be a preliminary pass without vibration to bed the concrete down, and again a final pass without vibration to remove the roller marks and to smoothen the surface.

12.6.2.2.5 The spreading, compacting and finishing of the concrete shall be carried out as rapidly as possible and the operation shall be so arranged as to ensure that the time between the mixing of the first batch of concrete in any transverse section of the layer and the final finishing of the same shall not exceed 90 minutes when the concrete temperature is above 25°C and below 30°C, and 120 minutes if less than 25°C. This period may be reviewed in light of the results of the trial run, but in no case shall it exceed 2 hours. Work shall not proceed when the temperature of the concrete exceeds 30°C. If necessary, chilled water or addition of ice may be resorted to for bringing down the temperature. It is desirable to stop concreting when the ambient temperature is above 35°C. After compaction has been completed, the roller shall not stand on the compacted surface for the duration of the curing period, except during commencement of next day's work near the location where the work was terminated the previous day. If the RCC is too wet or too dry for compaction upon laying, the water content shall be adjusted at the plant to correct this. Only minor changes in water content from the design mix should be made; otherwise a new mix design may be needed.

12.6.2.2.6 In view of the low water content in RCC, soon after the compaction is over, depending upon weather, wind velocity and humidity, curing shall start within one hour to two hours or alternatively curing compound can also be used.

12.6.2.3 Joints

The bond between the layers of RCC is produced by two mechanisms: cementitious (chemical) bond and penetration of the aggregates from the new layer into the surface of the previously placed layer (mechanical bond). As the open exposure time between placement of the layers increases, the chemical bond becomes the most prevalent factor, because the potential for mechanical penetration of the aggregates decreases faster than the chemical bond. The treatment of the joint surfaces between RCC layers differs from traditionally placed mass concrete, because bleeding does not normally occur in RCC with a reasonable water/cementitious ratio. However, it is common for the full compaction / consolidation of RCC layers to bring paste to the surface, which if properly cured, does not have to be removed prior to placement of the next layer. In addition, the joint surfaces must be kept scrupulously clean. Cold Joints can significantly increase the construction time and as they cannot be completely avoided, the necessary actions must be taken to minimise these in number and their total area.

12.6.2.4 Contraction joints and crack control

The current practice for RCC design is to control temperature cracking with contraction joints. Contraction joints are installed using several methods. One method that has been used on several RCC construction projects is to create a crack or joint in the RCC by installing a galvanized steel sheet into the compacted RCC lifts at and along a predetermined joint location. This galvanized steel sheets act as a crack inducer. Other methods include the use of alternative bond breaker materials, such as plastic sheets.

12.6.3 Physical Properties of RCC

Physical properties of RCC are generally similar to normal concrete except that variation in strength and thus standard deviations may be higher. Ratios of tensile strength vs compressive strength may also differ and shall be determined at mix design stage.

12.7 Alkali Activated Concrete

12.7.1 Alkali activated concrete or geopolymer concrete is a type of concrete that is made by reacting aluminate and silicate bearing materials such as fly ash or slag from iron and metal production with an alkaline activator. Generally no cement is used in alkali activated concrete and thus it can be a substitute for Portland cement concrete. However, in certain cases, a small percentage of cement is added to bring down the curing temperature requirements.

12.7.2 The advancements in material sciences have led to development of altogether alternative binding material to concrete in the form of alkali activated concrete or geopolymer concrete (which is considered part of broad group of alkali activated materials). Alkali activated concrete are a relatively newer class of building material.

Owing to increased environmental concerns as well as diminishing natural resources alkali activated concrete are spotted as a valuable alternative for Portland cement. Alkali activated concrete can have similar cementing characteristics as Portland cement, but they can be produced out of by-products from other industry (for example, fly ash) or less energy craving and CO₂ emitting materials (for example, calcined clay) using alkali activators.

12.7.3 The alkali activated concrete can be used for manufacture of precast unreinforced products for non structural applications like paver blocks, kerb stones, blocks, etc as per guidelines given in IS 17452. The grades shall be limited up to M 40.

ANNEX A
(Clause 2)**LIST OF CROSS REFERRED STANDARDS**

<i>IS No.</i>	<i>Title</i>
IS 2 : 2022	Rules for rounding off numerical values (<i>second revision</i>)
IS 269:2015	Ordinary portland cement- Specification (<i>sixth revision</i>)
IS 383:2016	Coarse and fine aggregate for concrete- Specification (<i>third revision</i>)
IS 432(Part 1):1982	Specification for Mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part 1 Mild steel and medium tensile steel bars
IS 455:2015	Portland slag cement- Specification (<i>fifth revision</i>)
IS 457:1957	Code of practice for general construction of plain and reinforced concrete for dams and other massive structures
IS 516	Method of tests for strength of concrete
(Part 1/Sec 1):2021	Testing of strength of hardened concrete: Sec 1 compressive, flexural and split tensile strength (<i>first revision</i>)
(Part 2/Sec 3):2022	Properties of Hardened Concrete other than Strength: Sec 3 Oxygen Permeability Index (<i>first revision</i>)
(Part 2/Sec 4):2021	Properties of hardened concrete other than strength: Sec 4 Determination of the carbonation resistance by accelerated carbonation method (<i>first revision</i>)
(Part 4):2018	Sampling, preparing and testing of concrete cores (<i>first revision</i>)
(Part 5/Sec 1):2018	Non-destructive testing of concrete Section 1 Ultrasonic pulse velocity testing (<i>first revision</i>)
(Part 5/Sec 4):2020	Non-destructive testing of concrete Section 4 Rebound hammer test (<i>first revision</i>)
IS 875	Code of practice for design loads (other than earthquake) for buildings and structures
(Part 1):1987	Dead loads- Unit weights of building material and stored materials (<i>Incorporating IS 1911:1967</i>)
(Part 2):1987	Imposed loads
(Part 3):2015	Wind loads (<i>third revision</i>)
(Part 4):2021	Snow loads
(Part 5):1987	Special loads And combinations
1199:1959	Methods of sampling, testing and analysis
(Part 4):2018	Determination of air content of fresh concrete (<i>first revision</i>)
(Part 5):2018	Making and curing of test specimens (<i>first revision</i>)
(Part 6):2018	Tests on fresh self compacting concrete (<i>first revision</i>)
(Part 7):2018	Determination of setting time of concrete by penetration resistance (<i>first revision</i>)
IS 1489	Portland pozzolana cement- Specification
(Part 1):2015	Fly ash based (<i>fourth revision</i>)
(Part 2):2015	Calcined clay based (<i>fourth revision</i>)
IS 1566:1982	Hard-drawn steel wire fabric for concrete reinforcement (<i>second revision</i>)

IS 1599:2023/ ISO 7438:2020	Metallic- Bend test
IS 1608:2005	Mechanical testing of metals- tensile testing
(Part 1):2022	Method of test at room temperature
(Part 3):2018	Method of test at low temperature
IS 1785(Part 1):1983	Plain hard-drawn steel wire for prestressed concrete: Part 1 Cold drawn stress-relieved wire
IS 1786:2008	High strength deformed steel bars and wires for concrete reinforcement (<i>fourth revision</i>)
IS 1791:2020	General requirements for batch type concrete mixers (<i>third revision</i>)
IS 1893	Design earthquake hazard and criteria for earthquake resistant design of structures (<i>fifth revision</i>)
(Part 1):2025	General provisions
(Part 2):2025	Buildings
(Part 3):2025	Liquid retaining tanks
(Part 4):2025	Bridges and retaining walls
(Part 5):2025	Industrial structures and stack-like structures
(Part 6):2025	Base isolated buildings
(Part 7):2025	Pipelines
(Part 8):2025	Dams and embankments
(Part 9):2025	Coastal structures
(Part 10):2025	Steel towers
(Part 11):2025	Tunnels
IS 1904:2021	General requirements for design and construction of foundations in soils code of practice (<i>fourth revision</i>)
IS 2062:2011	Hot rolled medium and high tensile structural steel (<i>seventh revision</i>)
IS 2090:1983	High tensile steel bars used in prestressed concrete (<i>first revision</i>)
IS 2770(Part 1):1967	Methods of testing bond in reinforced concrete:Part 1 Pull-out test
IS 2386(Part 3):1963	Methods of test for aggregates for concrete: Part 3 Specific gravity, density, voids, absorption and bulking
IS 2386(Part 7):1963	Methods of test for aggregates for concrete: Part 7 Alkali aggregate reactivity
IS 2502:1963	Code of Practice for Bending and Fixing of Bars for Concrete Reinforcement
IS 2505:2023	Immersion type concrete vibrators - general requirements (<i>fourth revision</i>)
IS 2506:2022	General requirements for concrete vibrators screed board type (<i>second revision</i>)
IS 2514:2023	Concrete vibrating tables - Specification (<i>first revision</i>)
IS 2911(Part 1/Sec 1):2010	Design and construction of pile foundations — code Of practice :Part 1 Concrete piles Section 1 Driven cast in-situ concrete piles (<i>second revision</i>)
IS 3025	Methods of sampling, test (physical and chemical) for water used in industry
(Part 17):2022	Non-filterable residue total suspended solids at 103° C - 105° C (<i>second revision</i>)

(Part 18):2022	Volatile and fixed solids (total, filterable and non-filterable) at 550° C
(Part 22):2024	Acidity
(Part 23):2023	Alkalinity
(Part 24/sec 1):2022	Method of sampling and test (physical and chemical) for water and waste water Part 24:sec 1 Gravimetric and turbidity methods (<i>second revision</i>)
(Part 24/sec 2):2021	Method of sampling and test (physical and chemical) for water and waste water Part 24:sec 2 Method by continuous flow analysis
(Part 32):1988	Chloride (<i>first revision</i>)
IS 3370	Concrete Structures for Retaining Aqueous Liquids - Code of Practice
(Part 1):2021	General Requirements (<i>second revision</i>)
(Part 2):2021	Plain and Reinforced Concrete Structures (<i>second revision</i>)
(Part 3):2021	Prestressed Concrete Structures (<i>first revision</i>)
(Part 4):2021	Design tables (<i>Incorporated Amendment Nos. 1 & 2</i>)
(Part 4) Sec 1:2021	Plates
(Part 4) Sec 2:2021	Rectangular tanks
(Part 4) Sec 3:2021	Circular Tanks
IS 3414:1968	Code of Practice for Design and Installation of Joints in Buildings
IS 3812	Fly ash for use as pozzolana and admixture
(Part 1):2013	For use as pozzolana in cement, cement mortar and concrete (<i>third revision</i>)
(Part 2):2013	For use as admixture in cement, cement mortar and concrete (<i>third revision</i>)
IS 3951 (Part 1):2023	Hollow Clay Tiles For Floors And Roofs Specification: Part 1 Filler Type
IS 4031 (Part 5):1988	Methods of physical tests for hydraulic cement: Part 5 Determination of initial and final setting times (<i>first revision</i>)
IS 4082:1996	Recommendations on stacking and storage of construction materials and components at site (<i>second revision</i>)
IS 4845:1968	Definitions and terminology relating to hydraulic cement
IS 4926:2003	Code of practice ready-mixed concrete (<i>second revision</i>)
IS 4948:2020	Welded steel wire fabric for general use - specification (<i>third revision</i>)
IS 4991 (Part 1):2024	Criteria for blast resistant design of structures Part 1 Above-ground explosions (<i>first revision</i>)
IS 5816:1999	Method of test splitting tensile strength of concrete (<i>first revision</i>)
IS 6003:2010	Indented wire for prestressed concrete (second revision)
IS 6006:2014	Uncoated stress relieved strand for prestressed concrete-Specification
IS 6452:1989	Specification for high alumina cement for structural use
IS 6461	Cement concrete- Glossary of terms
(Part 1):2024	Concrete aggregate (<i>first revision</i>)

(Part 2):2024	Materials (Other than Cement and Aggregate) (<i>first revision</i>)
(Part 3):2024	Concrete Reinforcement (<i>first revision</i>)
(Part 4):2024	Types of concrete (<i>first revision</i>)
(Part 5):2024	Formwork for concrete (<i>first revision</i>)
(Part 6):2024	Equipment, Tools and Plant (<i>first revision</i>)
(Part 7):2024	Mixing, laying, compaction, curing and other construction aspects (<i>first revision</i>)
(Part 8):2024	Properties of concrete (<i>first revision</i>)
(Part 9):2024	Structural aspects (<i>first revision</i>)
(Part 10):2024	Tests and testing apparatus (<i>first revision</i>)
(Part 11):2024	Prestressed concrete (<i>first revision</i>)
(Part 12):2024	Miscellaneous terms (<i>first revision</i>)
IS 6909:1990	Specification for supersulphated cement (<i>first revision</i>)
IS 6922:1973	Criteria for safety and design of structures subject to underground blasts
IS 8041:1990	Specification for rapid hardening Portland cement (<i>second revision</i>)
IS 8042:2015	White Portland cement - Specification (<i>third revision</i>)
IS 8043:1991	Specification for hydrophobic Portland cement (<i>second revision</i>)
IS 9013:1978	Method of making, curing and determining compressive strength of accelerated cured concrete test specimens
IS 9103:1999	Specification for concrete admixtures (<i>first revision</i>)
IS 9142	Specification for Artificial Lightweight Aggregates for Concrete Masonry Units
(Part 1):2018	For concrete masonry block and for applications other than for structural concrete Sintered Fly ash coarse aggregate (<i>first revision</i>)
(Part 2):2018	Sintered Fly ash coarse aggregate (<i>first revision</i>)
IS 10262:2019	Concrete mix proportioning — guidelines (<i>second revision</i>)
IS 10500:2012	Drinking water Sintered Fly ash coarse aggregate (<i>second revision</i>)
IS 11384:2022	Composite construction in structural steel and concrete - Code of Practice
IS 11817:1986/ ISO 7727:1984	Classification of joints in buildings for accommodation of dimensional deviations during construction
IS 12089:1987	Specification for Granulated Slag for the Manufacture of Portland Slag Cement
IS 12119:1987	General requirements for pan mixers for concrete
IS 12330:1988	Specification for sulphate resisting Portland cement
IS 12600:1989	Specification for low heat Portland cement
IS 13920	Earthquake resistant design and detailing of structures - Code of practice (<i>second revision</i>)
(Part 1):2025	General provisions
(Part 2):2025	Buildings
(Part 3):2025	Liquid retaining tanks
(Part 4):2025	Bridges and retaining walls
(Part 5):2025	Industrial structures and stack-like structures

(Part 6):2025	Base isolated buildings
(Part 7):2025	Pipelines
(Part 8):2025	Dams and embankments
(Part 9):2025	Coastal structures
(Part 10):2025	Steel towers
(Part 11):2025	Tunnels
IS 13935	Earthquake safety assessment and retrofitting of structures – Code of practice (<i>second revision</i>)
(Part 1):2025	General provisions
(Part 2):2025	Buildings
(Part 3):2025	Liquid retaining tanks
(Part 4):2025	Bridges and retaining walls
(Part 5):2025	Industrial structures and stack-like structures
(Part 6):2025	Base isolated buildings
(Part 7):2025	Pipelines
(Part 8):2025	Dams and embankments
(Part 9):2025	Coastal structures
(Part 10):2025	Steel towers
(Part 11):2025	Tunnels
IS 14268:2022	Uncoated stress relieved low relaxation seven-wire (ply) strand for prestressed concrete-Specification(second revision)
IS 14959	Method of Test determination of water soluble and acid soluble chlorides in mortar and concrete:
(Part 1):2001	Part 1 Fresh mortar and concrete
(Part 2):2001	Part 2 Hardened mortar and concrete
IS 15388:2003	Specification for Silica fume
IS 15883	Construction project management - Guidelines
(Part 1):2009	General
(Part 2):2013	Time Management
(Part 3):2015	Cost Management
(Part 4):2015	Quality Management
(Part 5):2013	Health and Safety Management
(Part 6):2015	Scope Management
(Part 7):2021	Procurement Management
(Part 8):2015	Risk Management
(Part 9):2018	Communication Management
IS 16172:2023	Reinforcement couplers for mechanical splices of steel bars In concrete – Specification(first revision)
IS 16353:2015	Portland cement clinker - Specification
IS 16354:2015	Metakaolin for use in cement, cement mortar and concrete - Specification
IS 16415:2015	Composite cement - Specification
IS 16712:2018	Automotive Vehicles- Spray Suppression Systems for Two Wheeled Motor Vehicles
IS 16714:2018	Ground granulated blast furnace slag for use in cement, mortar & concrete- Specification

IS 16715:2018	Ultrafine ground granulated blast furnace slag — Specification
IS 16993:2018	Microfine ordinary portland cement — Specification
IS 17452:2020	Use of Alkali Activated Concrete for Precast Products - Guidelines
SP 7:2016	National Building Code of India (<i>third revision</i>)

ANNEX B

(Clause 9.5.4.3.3.9(b) and Table 66)

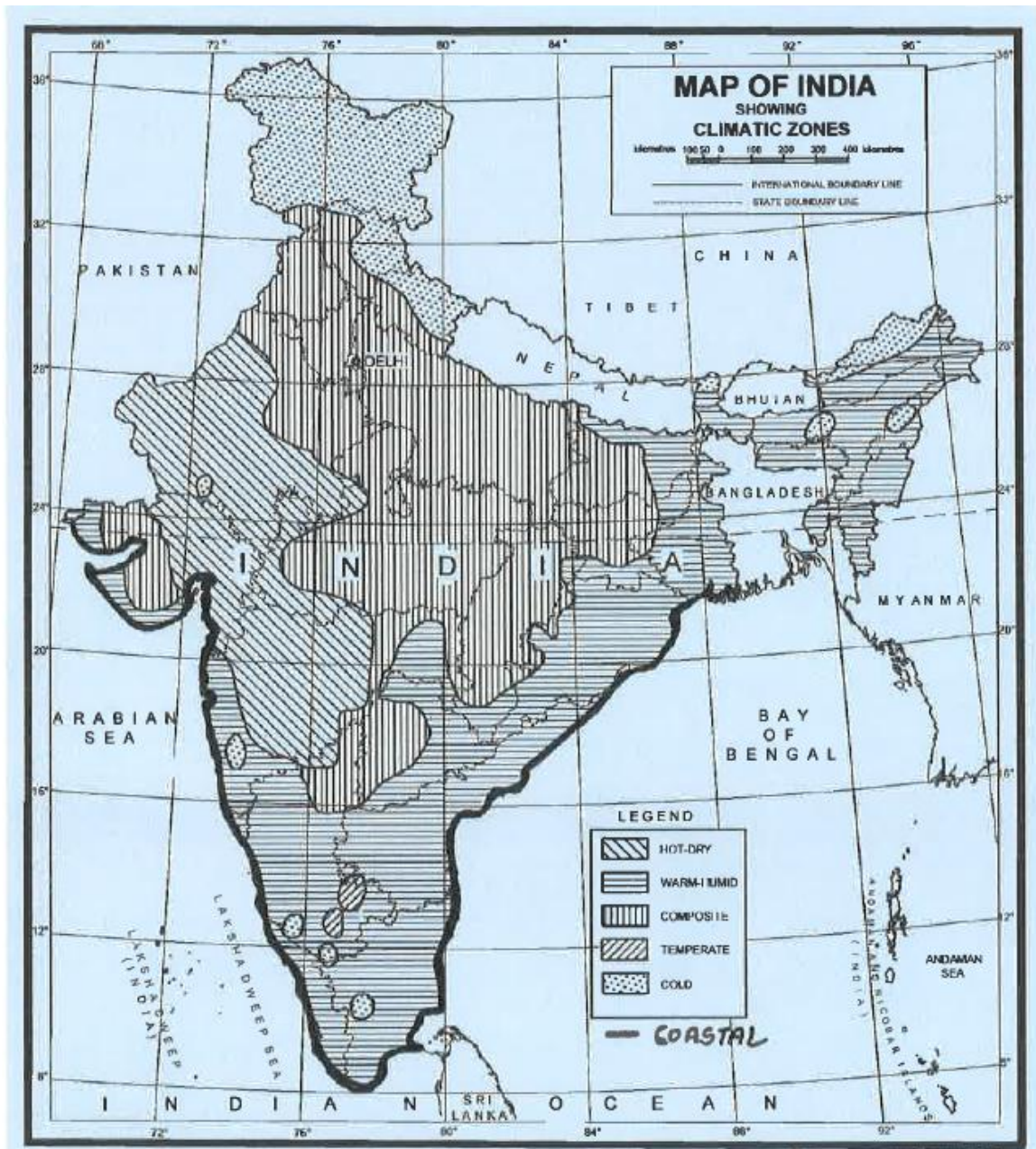


FIG. 89 CLIMATIC ZONES AS CLASSIFIED IN INDIA

ANNEX C
(Clause 10.6.3.1.4)**TESTING OF SYSTEMS WITH MECHANICAL ANCHORAGES****C-1 STATIC LOAD TEST WITH TENDON-ANCHORAGE ASSEMBLY**

The aim of the test is to assess the performance of the tendon-anchorage assembly and to determine any decrease of the breaking load of the prestressing steel due to the influence of the anchorage. The test is also relevant for couplings.

C-1.1 Test Specimen

The tendon and anchorage to be tested should be assembled according to the envisaged application, using all the components necessary for anchoring the tendon. The geometrical configuration of the individual wires, strands or bars in the specimen should be identical to that of the actual tendon-anchorage assembly and so the following properties should be established:

- a) Main geometrical and mechanical properties of the prestressing steel used in the test;
- b) Actual mean breaking load;
- c) Mean total elongation at maximum load;
- d) Mean cross-section;
- e) Surface characteristics, etc; and
- f) Minimum of three single unit tensile tests.

Relevant geometrical and mechanical properties of anchorage components should also be determined. The free length of the individual wires, strands or bars in the specimen to be tested should be not less than 3.0 m.

If one grade of prestressing steel of the same type is to be used with the same type of anchorage, the tests should be performed using the grade with the highest characteristic tensile strength.

C-1.2 Test Procedure

The tendon specimen is mounted in a calibrated test rig or testing machine and should be stressed in increments of force corresponding to 20 percent, 40 percent, 60 percent and 80 percent of the characteristic tensile strength of the prestressing steel, f_p . In each increment the force is increased at a constant force rate corresponding to about 100 MPa/min. At the 80 percent level, the force is held constant for one hour. Subsequently, the force is increased gradually to total failure.

The described test procedure is normally applicable to systems where the prestressing steel is connected to the anchorage before applying the stressing force. In systems where the prestressing level is locked in the anchorage after the stressing force has been applied, the above test procedure should be adapted as follows:

- a) The tendon should be mounted in the test rig or testing machine, using an identical geometrical configuration of the individual wires, strands or bars in the specimen to that in the actual tendon-anchorage assembly.
- b) The tendon should be stressed at one end with the equipment used on the construction site, in steps as described, to 80 percent of the characteristic tensile strength of prestressing steel, f_p .
- c) After the tendon has been locked at this force, the further testing procedure should be as previously described.

The test can be carried out in the manner described in the first instance if it can be proved that locking the prestressing steel in anchorage after the stressing force has been applied has no significant influence on the behaviour of the tendon-anchorage assembly.

C-1.3 Measurements and Observations

The following measurements and observations should be made and recorded:

- a) Relative force and time dependent displacement Δa of the prestressing steel with respect to the anchorage (see Fig. 90),
- b) Relative force and time dependent displacement Δb between the individual components of the anchorage (see Fig. 90),
- c) Deformation of the anchorage components,
- d) Elongation of the tendon on free length up to failure,
- e) Measured ultimate force F_{Tu} , and
- f) Location and mode of failure.

C-1.4 Test Requirements

The test result should meet the following requirements:

Residual deformations of anchorage components after the test should confirm the reliability of the anchorage.

The increase in the displacements between the anchorage components as well as between the prestressing steel and anchorage components should not be disproportionate to the increase in tendon force.

The above relative displacements during the $0.8 f_p$ load stage should stabilize within the first 30 min of the load duration of 1 h.

The mode of failure of tendon should be by the fracture of the prestressing steel. Failure of the tendon should not be induced by the failure of anchorage components.

The measured anchorage efficiency is:

$$\eta_A = \frac{F_{Tu}}{F_{pm}} \geq 0.95$$

F_{pm} = actual mean breaking load

The total elongation in free length of the tendon under the load F_{Tu} should be not less than 2 percent.

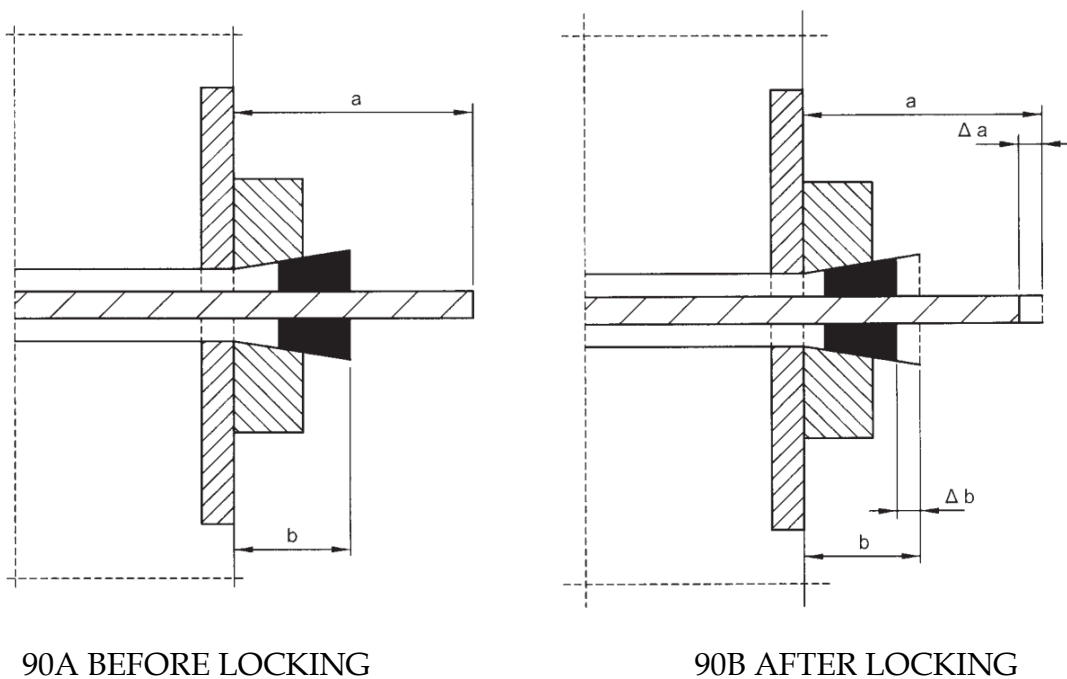


FIG. 90 DISPLACEMENTS DURING TESTING

C-2 DYNAMIC LOAD TEST WITH TENDON ANCHORAGE ASSEMBLY

The aim of the test is to determine the capacity of the tendon-anchorage assembly under load fluctuations as an indication of the reliability and durability of the assembly.

C-2.1 Test Specimen

The type of specimen should correspond to the requirements of C-1.1. If one grade of prestressing steel of the same type is to be used with the same type of anchorage, the tests should be performed with prestressing steel using the grade with the highest characteristic tensile strength.

Where the capacity of the testing machine to be used could be exceeded, the number of wires, strands or bars in the tendon-anchorage assembly to be tested may be reduced, provided that the prestressing steel units with the most severe angular deviation of tendon axis from the prescribed are included and that the test results are representative for the complete tendon.

C-2.2 Test Procedure

The test should be performed in a tensile testing machine with a pulsator at a constant force frequency of not more than 500 load cycles per min, and with a constant upper force of 65 percent of the characteristic strength of the prestressing steel. The range of forces, $\Delta f = F_{Max} - F_{Min}$ should be maintained at a constant level throughout testing.

The specimen should be tested in such a way that secondary oscillations are precluded. When assembling the specimen and fitting it in the testing machine, special care should be taken to ensure that the force is evenly distributed to all the tensile elements of the tendon. In the case of grouted tendons special devices may be used to avoid premature fretting failure of the prestressing steel remote from the anchorage.

C-2.3 Measurements and Observations

The following measurements and observations should be made and recorded:

- a) Relative displacement between prestressing steel and individual anchorage components as well as between the anchorage components themselves, depending on load and on number of load cycles (see Fig. 90).
- b) Examination of anchorage components and prestressing steel after test with respect to fatigue damage and deformation.

C-2.4 Steel Requirements

Each test result should meet the following requirements:

- a) Fatigue failure of anchorage components should not occur.
- b) The minimum fatigue strength of a posttensioning system should be

$$\Delta\sigma_{p \text{ Min}} = 80 \text{ MPa}$$

The fatigue strength is defined as the stress range $\Delta\sigma_p$ which is endured for 2 million cycles without failure of 5 percent of the initial cross-section of the tested tendon at beginning of the test.

C-3 LOAD TRANSFER TEST

With the load transfer test, the transfer of the prestressing force from the mechanical anchorage and its components to the concrete should be verified.

C-3.1 Test Specimen

The test specimen is schematically depicted in Fig. 91. The specimen should contain those anchorage components which will also be embedded in the structural concrete. Their arrangement should comply with the practical application and with the prestressing contractor's specification.

The test specimen should be a concrete prism tested in axial compression.

The part of the specimen containing the anchorage should be provided with bursting reinforcement of the same size and configuration as specified for the particular system and tendon respectively. Reinforcement may be used for the assembly of the bursting reinforcement. It should not be considered part of the technical approval if the following conditions are not complied with:

- a) Longitudinal bars have a total cross-section more than 200 mm².
- b) Stirrups, that is, the secondary reinforcement, uniformly distributed along the height of specimen, should constitute more than 0.05 t steel/m³ concrete.

The concrete of the test specimen should correspond to the actual concrete used for prestressed concrete structures with respect to materials, composition, compaction and its characteristic strength f_{ck} . After casting the specimen should be demoulded after one day and then moist-cured until testing. The cubes cast for the determination of compressive strength should be treated similarly.

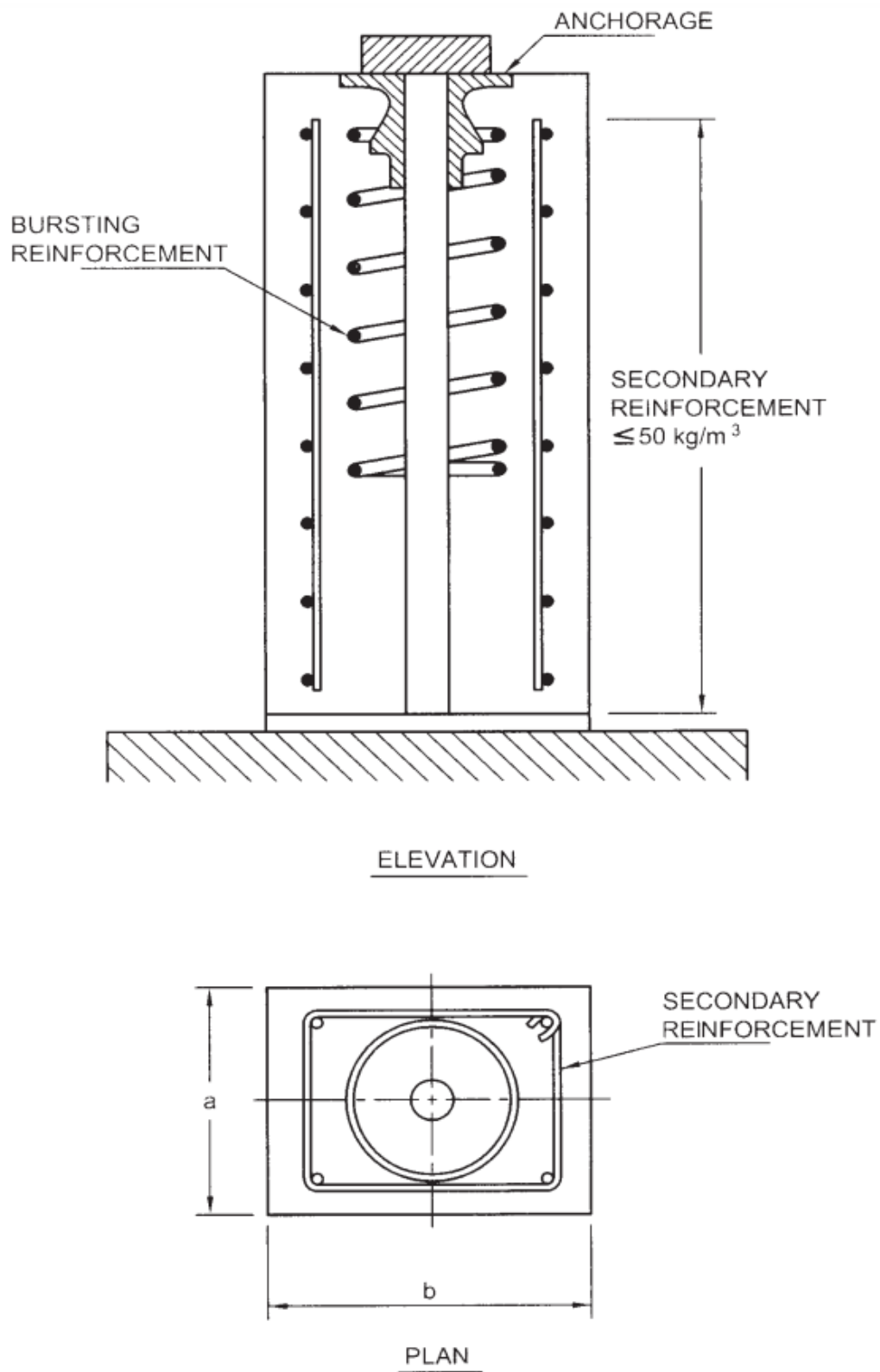


FIG. 91 TEST SPECIMEN FOR LOAD TRANSFER TEST

C-3.2 Test Procedure

The specimen should be mounted in a calibrated test rig or testing machine. The load should be applied corresponding to actual loading conditions in the structure either by loading the tendon or by applying the force directly to the anchorage. The force is increased in increments of 20 percent from 0 percent up to 80 percent of the characteristic strength of the prestressing steel, f_p (see Fig. 92).

After reaching the load $0.8 f_p$ at least ten slow load cycles should be performed, with at least $0.8 f_p$ and $0.12 f_p$ being the upper and lower load limits respectively.

The necessary number of load cycles depends upon the stabilization of strain readings and crack widths as described below. Following cyclic loading, the specimen should be loaded incrementally to failure.

During cyclic loading measurements should be taken at the upper and lower loads of several cycles in order to decide whether a satisfactory stabilization of strains and crack widths is being attained. Cyclic loading should be continued until stabilization is satisfactory (see C-3.4). Fig. 92 shows the sequence of loading and measurements.

At the final test to failure the mean compressive strength of concrete of specimen should be:

$$f_{cm,e} \leq f_{ck,o} \text{ and } f_{cm,e} \leq 0.85f_{ck}$$

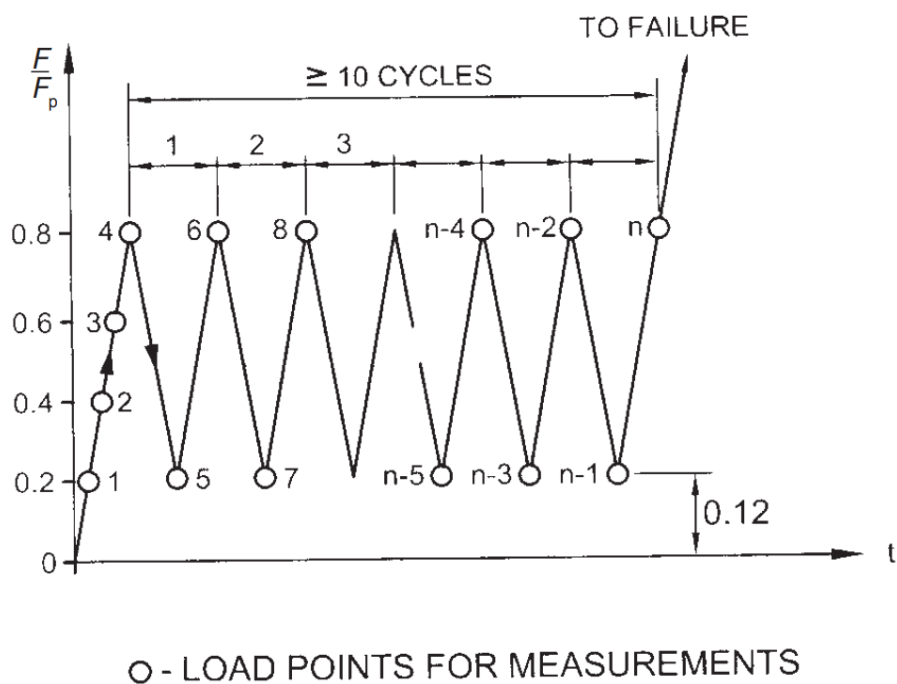


FIG. 92 PROCEDURE FOR THE LOAD TRANSFER TEST

C-3.3 Measurements and Observations

The following measurements and observations should be taken and recorded:

- a) Longitudinal and transverse concrete strains on the side faces of the specimen in the region of maximum bursting effect under the upper and lower load at different times for increasing numbers of load cycles.
- b) Formation, width and propagation of cracks on the side faces of the specimen, as mentioned above.
- c) Visual inspection and/or measurement of deformation of anchorage components in contact with the concrete.
- d) Location and mode of failure.

C-3.4 Requirements

Each specimen should meet the following requirements (*see* Fig. 93):

- a) Crack width upon first attainment of upper force 0.8 (load point 4) should be ≤ 0.10 mm.
- b) Crack width upon last attainment of lower force 0.12 (load point n-1) should be ≤ 0.10 mm.
- c) Crack width upon final attainment of upper force 0.8 (load point n) should be ≤ 0.25 mm.
- d) Readings of longitudinal and transverse strains should have stabilized during cyclic loading. The strains can be assumed to have stabilized if the increase of strain in last two load cycles is less than 5 percent.
- e) Readings of crack widths should have stabilized during cyclic loading. The crack widths can be assumed to have stabilized if the crack width does not increase by 0.02 mm in the last two load cycles.
- f) The measured failure load F_u shall meet each of the following requirements:

$$F_u \geq F_p \frac{f_{cm,e}}{f_{ck,o}} \text{ and } F_u \geq 1.1 F_p$$

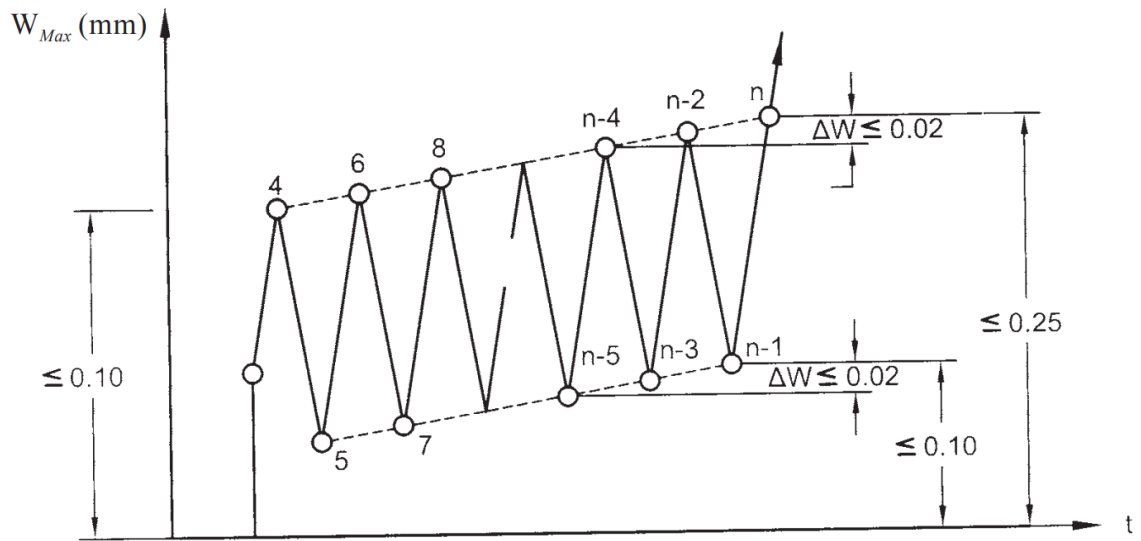


FIG. 93 REQUIREMENTS FOR CRACK WIDTHS

Annex D

[Clause 9.5.4.3.3.9(b)]

**DETERMINATION OF MEAN CARBONATION COEFFICIENT K_M FROM
ACCELERATED CARBONATION RESISTANCE TEST**

D-1 IS 516 (Part 2/Sec 4) covers the procedure for the measurement of carbonation depth by accelerated carbonation test. For the determination of mean carbonation coefficient K_m , test specimens shall be exposed to carbon dioxide levels between 3.0 to 3.5 % (by volume) for 70 days after 28 days water curing prior to conditioning period of 14 days. The testing will take 112 days period.

D-2 Measure mean depth of carbonation, C_m after testing the concrete specimens as per the procedure IS 516 (Part 2/Sec 4). The mean carbonation coefficient, K_m will be determined through following equation:

$$K_m = \frac{C_m}{t^x}$$

where:

C_m = mean depth of carbonation in mm measured at 70 days exposure to CO_2

x = constant which depends upon type of binder and shall be taken as

Concrete made with OPC = 0.60

Concrete made with PPC = 0.45

Concrete made with PSC = 0.45

t = Equivalent time (in years) represented by 70 days accelerated exposure.

D-3 For determining t (in years) from accelerated exposure period of 70 days following formula to be adopted

$$t = 70/365 * x * 10000/500, \text{ where } x \text{ is percent by volume of } CO_2 \text{].}$$

ANNEX E

(Clause 11.4.5)

DETERMINATION OF SURFACE ELECTRICAL RESISTIVITY OF CONCRETE USING FOUR POINT WENNER PROBE METHOD

E-1 The four-point electrode method is currently the most widely used method /technique for the measurement of concrete resistivity. During testing, a low frequency AC current is applied between the two outer electrodes while the voltage drop is measured in the two inner electrodes (refer to figure 83).

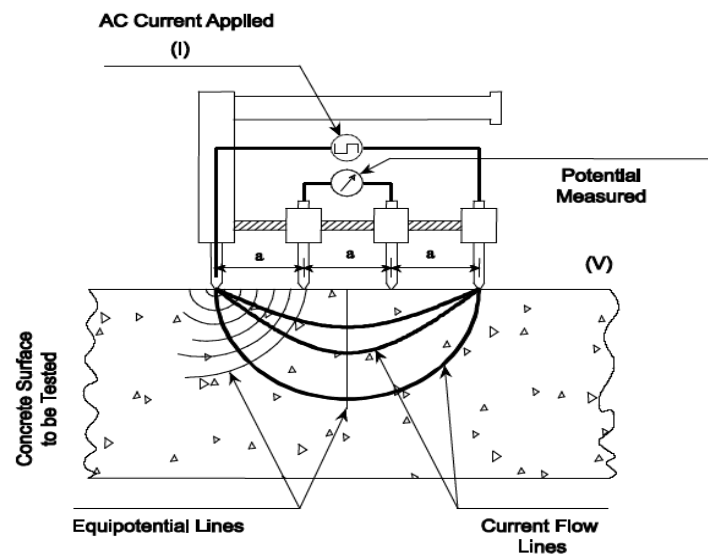


FIG. 94 Four-Point Wenner Array Probe Test Setup

E-2 The concrete resistivity for a semi-infinite region can be calculated as

$$\rho = 2\pi aV/I$$

where

ρ is the electrical resistivity (Kohm-cm),

a is electrode spacing (m),

V is voltage drop (V), and

I is the current (A).

E-3 APPARATUS

Wenner array probe capable of adjustment of the probe tip spacing up to 50 mm, a measuring range from 0 to 999±1 Kohm -cm , with a resolution of 0.1 Kohm-cm and an accuracy of +/- 2% of reading should be used for testing.

E-3.1 Test Specimen - concrete slabs (300 mmx300 mmx100 mm) (3 nos)

E-3.2 Test Procedure

The specimens should be at least 28 days water cured at the time of testing. The testing should be done at saturated surface dry condition. The test surface preferably shutter surface should be thoroughly cleaned to remove all laitance. At least 4 readings at different positions on the test surface of each specimen as shown in figure 84 should be taken. The average value of the readings taken from three specimens represents the surface electrical resistivity of the concrete.

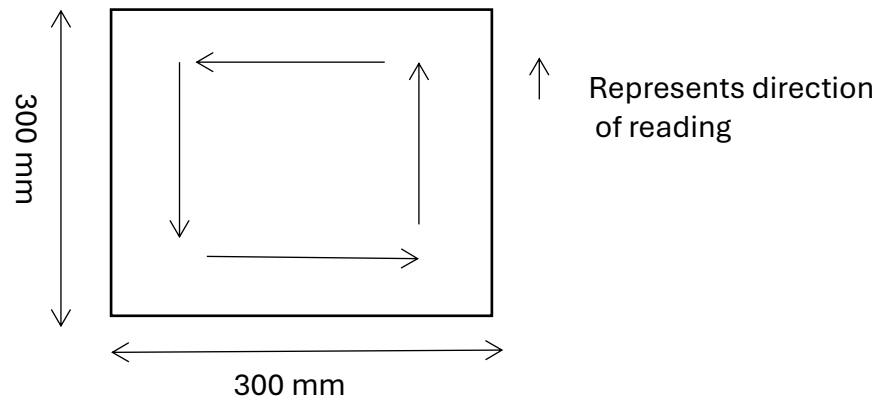


FIG. 95 CONCRETE SLAB (300 MM X 300 MM X 100 MM)

ANNEX F
(Clause 12.2.2)**RESIDUAL STRENGTH TEST****F-1 RESIDUAL TENSILE STRENGTH TEST**

F-1.1 The residual tensile strength shall be obtained using the testing arrangement shown in Fig. 96 and with the following criteria:

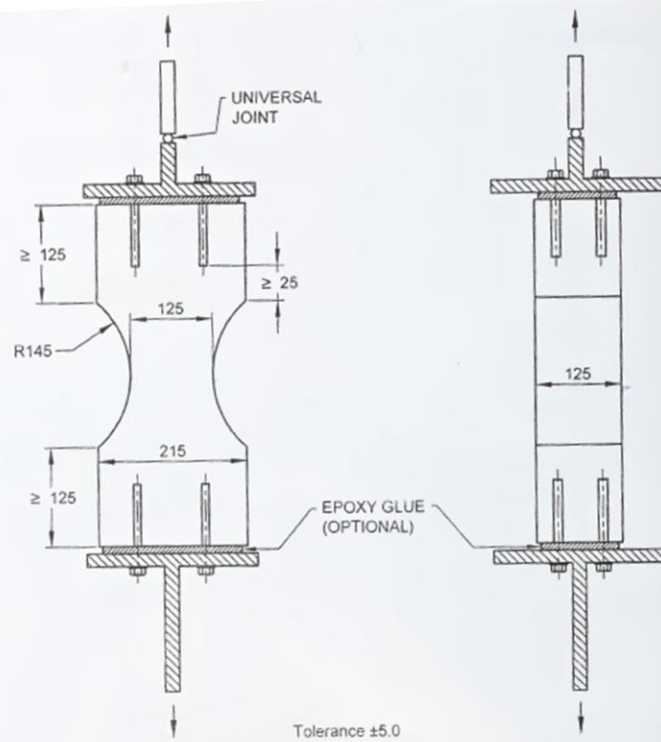
- a) The critical section shall be where the cross-sectional area is a minimum.
- b) The SFRC mix shall be batched to ensure a uniform distribution of fibres, and shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres. The SFRC shall be compacted using lightly applied external vibration.
- c) The specimen shall be connected to the testing machine in such a manner that the machine does not apply a load to the specimen during the process of tightening of the grips and prior to testing.
- d) One end of the specimen shall be connected to the testing machine through a universal joint such that no moment is applied to the end of the specimen.
- e) Displacement measurements shall be taken on each of the four sides with the COD taken as the average of these measurements.
- f) A minimum of twelve specimens shall be tested.
- g) Tests where the failure of the specimen is outside of the testing region, or where the results are influenced by the test specimen boundaries, shall be retested.
- h) The characteristic values of the tensile strength $f_{0.5m}$ and $f_{1.5m}$, corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 95 percentile confidence value assuming the population is normally distributed.
- j) The mean values of $f_{0.5m}$ and $f_{1.5m}$ corresponding to CODs of 0.5 mm and 1.5 mm, respectively, shall be determined statistically as the 50th percentile confidence value assuming the population is normally distributed.

The stress results obtained from the test shall be multiplied by the three-dimensional orientation factor k_1 , where,

$$k_1 = \frac{1}{0.94 + 0.6l_f/b} \leq 1$$

And l_f is the length and b is taken as the average of the width and depth of the specimen taken at the critical section.

NOTE – The factor k_1 removes the influence of the boundaries on the fibre distribution and converts the results of the test to a state where the fibres can be considered to be randomly orientated in three-dimensional space.



All dimensions in millimetres.

NOTE — There is considerable discussion in the literature on whether the ends of the specimens should be fixed at each end, pinned at each end or fixed at one end and pinned at the other. The adoption of one pinned and one fixed end is a pragmatic decision between the ideal condition (fixed at each end) and limitations in manufacturing of specimens that are perfectly square at their ends for alignment within the testing machine. With both ends fixed any misalignment induces a moment in the specimen during gripping. This moment can induce a tensile stress on one side of the specimen, weakening the specimen. In some situations, the moment induced may be sufficient to crack the concrete. The inclusion of the universal joint at one end eliminates residual tensions that may result from the gripping procedure.

FIG. 27 TESTING ARRANGEMENT FOR DIRECT TENSION

FIG. 96 TESTING ARRANGEMENT FOR DIRECT TENSION

F-2 RESIDUAL FLEXURAL TENSILE STRENGTH

The residual flexural tensile strength $f_{R,j}$ shall be determined from 3 point notched bending tests on 150 mm square section prisms. The notch depth shall be 25 mm and the test conducted in accordance with **12.2.2.4**. The force F shall be plotted against the crack mouth opening displacement (CMOD), as shown in Fig. 97, and the residual flexural stress calculated as:

where

b = width of the specimen, in mm;

h_{sp} = distance between tip of the notch and top of cross-section, in mm;

L - span, in mm; and

F_{Rj} = load recorded at CMOD_j (see Fig. 97).

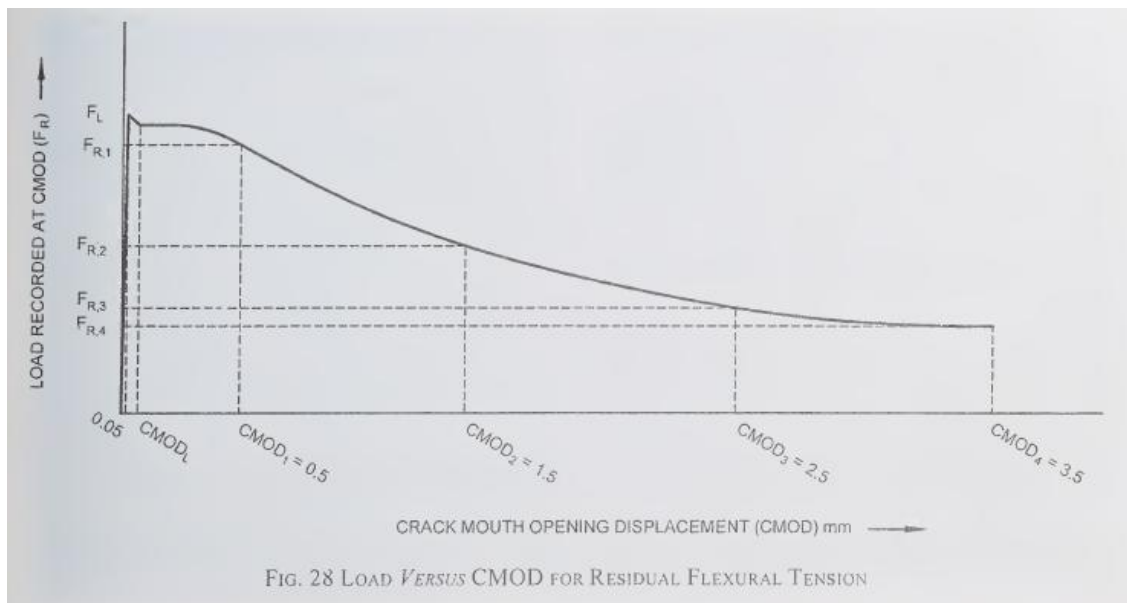


FIG. 97 LOAD VERSUS CMOD FOR RESIDUAL FLEXURAL TENSION

The average width of the specimen and distance between the tip of the notch and the top of the specimen in the mid-span section shall be determined from two measurements to nearest 0.1 mm width and distance in the notch port of the specimen using callipers.

When the CMOD is measured, the displacement transducer shall be mounted along the longitudinal axis at mid width of test specimen such that the distance 'y' between the bottom of specimen and line of measurement is 5 mm or less as mentioned in Fig. 98.

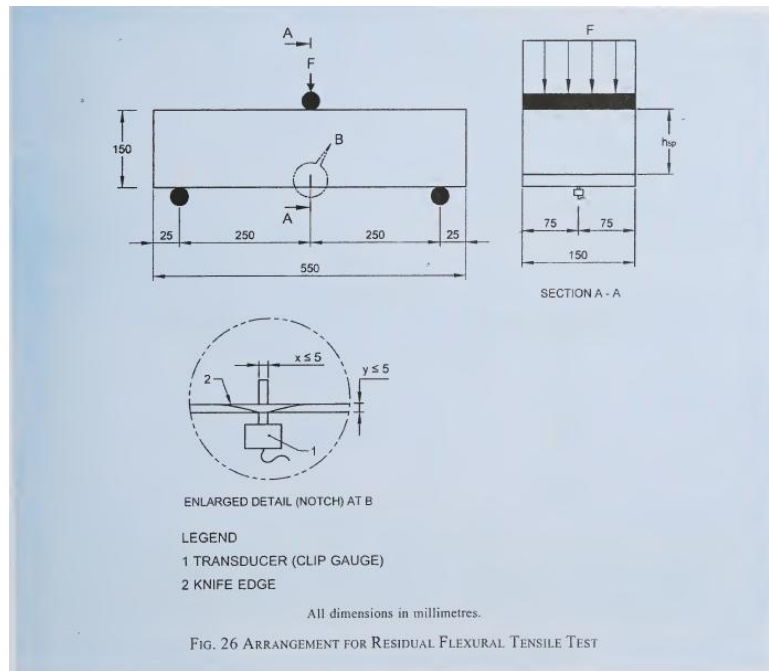


FIG. 98 ARRANGEMENT FOR RESIDUAL FLEXURAL TENSILE TEST

- a) The concrete mix shall be batched to ensure a uniform distribution of fibres, the SFRC shall be placed in the moulds in a manner that does not interfere with the distribution of the fibres and, the SFRC shall be compacted using lightly applied external vibration; and
- b) A minimum of 12 specimens shall be tested.

F-3 THE REFERENCE FACTOR $k_{R,4}$

The reference factor $k_{R,4}$ shall be determined as:

$$k_{R,4} = f_{1.5m} / f_{R,4m}$$

where

$f_{1.5m}$ = mean residual tensile strengths corresponding to a COD of 1.5 mm determined in accordance with F-1.1

$f_{R,4m}$ = mean residual flexural tensile strengths corresponding to a CMOD of 3.5 mm, determined in accordance with F-1.1

ANNEX G

(Foreword)

(Committee composition will be added after finalization)
