IS 11447 (1985): Code of practice for construction with large panel prefabricates [CED 51: Planning, Housing and pre-fabricated construction]

BLANK PAGE

IS : 11447 - 1985
(Reaffirmed 2003)

Indian Standard

CODE OF PRACTICE FOR CONSTRUCTION WITH LARGE PANEL PREFABRICATES

UDC 697·353·6 : 691·81 : 006·76

INDIAN STANDARDS INSTITUTIONS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

Gr 8

September 1986

Indian Standard

CODE OF PRACTICE FOR CONSTRUCTION WITH LARGE PANEL PREFABRICATES

Prefabricated and Composite Construction Sectional Committee, BDC 32

Chairman	*Representing*
SHRI A. RAMAKRISHNA	Larsen & Toubro Limited (ECC Construction Group), Madras
Members	
SHRI S. SUBRAMANIAN (*Alternate* to Shri A. Ramakrishna)	
SHRI P. S. AMBIKE	Ambike Consultants, New Bombay
DR N. S. BHAL	Structural Engineering Research Centre (CSIR), Roorkee
SHRI K. C. NAITHANI (*Alternate*)	
BRIG C. A. BARETTOO	Ministry of Defence (Engineer-in-Chief's Branch)
SHRI K. S. TECKCHANDANI (*Alternate*)	
SHRI L. S. BASSI	Ministry of Shipping and Transport (Roads Wing), New Delhi
SHRI R. GOPALAKRISHNAN (*Alternate*)	
SHRI B. K. CHAKRABORTY	Housing & Urban Development Corporation, New Delhi
SHRI S. M. GOEL (*Alternate*)	
SHRI S. C. CHAKRABARTI	Central Building Research Institute (CSIR), Roorkee
SHRI A. K. CHATTERJEE	Gammon India Ltd, Bombay
SHRI A. C. ROY (*Alternate*)	
DR P. RAY CHAUDHRY	Central Road Research Institute (CSIR), New Delhi
SHRI A. K. GARG (*Alternate*)	
CHIEF ENGINEERS (DESIGNS)	Central Public Works Department (CDO), New Delhi
SUPERINTENDING ENGINEER (DESIGNS) (*Alternate*)	
SHRI C. S. DABKE	Howe India (Private) Ltd, New Delhi
SHRI B. RAMAMURTHY (*Alternate*)	
DIRECTOR (C & MDD-I)	Central Water Commission, New Delhi
DEPUTY DIRECTOR (C & MDD-I) (*Alternate*)	

(*Continued on page* 2)

(*Continued from page* 1)

Members	*Representing*
SHRI A. GHOSHAL	In personal capacity (*12 Darga Road, 6th Floor, Calcutta*)
SHRI G. K. MAJUMDAR	Hospital Services Consultancy Corporation (India) Ltd, New Delhi
JOINT DIRECTOR STANDARDS (B & S) CB	Research, Designs and Standards Organization, Lucknow
DEPUTY DIRECTOR STANDARDS (B & S) CB (*Alternate*)	
DR V. PARAMASIVAM	Indian Institute of Technology, Madras
DR A. G. MADHAVA RAO	Structural Engineering Research Centre (CSIR), Madras
SHRI G. ANNAMALAI (*Alternate*)	
SHRI K. S. RAKSHIT	Indian Roads Congress, New Delhi
DR N. RAGHAVENDRA	Cement Research Institute of India, New Delhi
SHRI H. K. JULKA (*Alternate*)	
SHRI H. N. SUBBA RAO	Hindustan Steel Works Construction Ltd, Calcutta
SHRI S. N. ADHIKARI (*Alternate*)	
DR R. C. SONPAL	Institution of Engineers (India), Calcutta
BRIG L. V. RAMAKRISHNA (*Alternate*)	
SHRI SHIRISH B. PATEL	Shirish Patel & Associates, Bombay
SHRI P. H. SRINIVASACHAR (*Alternate*)	
SHRI K. A. PATNAIK	Bureau of Public Enterprises (Ministry of Finance), New Delhi
SHRI K. S. SRINIVASAN	National Buildings Organization, New Delhi
SHRI SUNIL BERY (*Alternate*)	
SHRI M. C. TANDON	STUP Consultants Ltd, Bombay
SHRI K. VEERARAGHAVACHARY	Bharat Heavy Electricals Ltd, Hyderabad
REPRESENTATIVE	Hindustan Prefab Ltd, New Delhi
SHRI M. KUNDU (*Alternate*)	
SHRI G. RAMAN, Director (Civ Engg)	Director General, ISI (*Ex-officio Member*)

Secretaries

SHRI S. K. JAIN
Deputy Director (Civ Engg), ISI
SHRI A. K. AVASTHY
Deputy Director (Civ Engg), ISI

Prefabrication Systems Subcommittee, BDC 32 : 1

Convener	
SHRI G. K. MAJUMDAR	Hospital Services Consultancy Corporation (India) Ltd, New Delhi
Members	
SHRI P. S. AMBIKE	Ambike Consultants, New Bombay
LT-COL ARUN KUMAR	Engineer-in-Chief's Branch, Army Headquarters (Ministry of Defence), New Delhi
SHRI DINESH SIKAND, EE (*Alternate*)	

(*Continued on page* 39)

Indian Standard

CODE OF PRACTICE FOR CONSTRUCTION WITH LARGE PANEL PREFABRICATES

0. FOREWORD

0.1 This Indian Standard was adopted by the Indian Standards Institution on 30 September 1985, after the draft finalized by the Prefabricated and Composite Construction Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 This is a brief review of the proposed code of practice for the structural analysis, design and construction of buildings with large panel prefabricates for non-seismic and seismic zones. At present there is no national standard for the design of large panel buildings. In this context, the present code will serve as guidelines for the design of large panel buildings in India.

0.3 This code is to be read in conjunction with other relevant standards, such as IS : 456-1978*, IS : 1343-1980† and IS : 1905-1980‡.

0.4 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 : 2-1960§. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

---

1. SCOPE

1.1 This code deals with the design, structural analysis of members and buildings constructed with large panel prefabricates.

1.2 The requirements given in this code shall be applied in the design of precast units made of plain, reinforced and lightweight concrete and in the design of apartment and public utility buildings with these units.

1.3 Special requirements of panels for walls, facades, sanitary code and staircase, etc, used in large panel construction are not covered in this code.

---

*Code of practice for plain and reinforced concrete (*third revision*).
†Code of practice for prestressed concrete (*first revision*).
‡Code of practice for structural safety of buildings: Masonry walls (*second revision*).
§Rules for rounding off numerical values (*revised*).

2. TERMINOLOGY

2.0 For the purpose of this code, the definitions given in IS : 456-1978* and the following shall apply.

2.1 Large Wall Panel Units or Wall Panel — An element primarily of height not less than one storey height and width usually between walls.

2.2 Large Floor Slab Unit or Floor Slab — An element primarily loaded perpendicular to its plane with its length equal to the span of the room.

2.3 Large Panel Building — A building using wall and floor panel as defined in **2.1** and **2.2**.

2.4 Load-Bearing Wall — A wall designed to carry an imposed vertical load in addition to its weight together with any lateral load.

2.5 Shear Wall — A wall designed to carry horizontal forces acting in its plane with or without vertical imposed loads thus assuring the stability of the building.

2.6 Non-Load Bearing Walls — A wall panel used for partition which does not transfer any load other than its self-load.

2.7 Tie Beams — A beam generally provided at floor level all along the structural walls and along the perimeter of the building to obtain monolithic action of walls and floors.

2.8 Progressive Collapse — Progressive collapse is the consequence of a local failure which precipitates failure of a major area of several floors of a building.

3. SYMBOLS

3.1 For the purpose of this standard, the following letter symbols shall have the meaning indicated against each; where other symbols are used, they are explained at the appropriate place:

A — Area

b — Breadth of beam or shorter dimension of a rectangular column

b_{ef} — Effective width of slab

b_f — Effective width of flange

b_w — Breadth of web or rib

*Code of practice for plain and reinforced concrete (*third revision*).

D — Overall depth beam or slab or diameter of column; dimension of a rectangular column in the direction under consideration

D_f — Thickness of flange

DL — Dead load

d — Effective depth of beam or slab

d' — Depth of compression reinforcement from the highly compressed face

E_t — Modulus of elasticity of concrete

EL — Earthquake load

E_s — Modulus elasticity of steel

e — Eccentricity

f_{ck} — Characteristic compressive strength of concrete

f_{cr} — Modulus of rupture of concrete (flexural tensile strength)

f_{ct} — Split tensile strength of concrete

f_d — Design strength

f_y — Characteristic strength of steel

I_{ef} — Moment of inertia of the gross section excluding reinforcement

I_r — Moment of inertia of cracked section

K — Stiffness of member

k — Constant or coefficient or factor

L_d — Development length

LL — Live load or imposed load

l — Length of a column or beam between adequate lateral restraints or the unsupported length of a column

l_{ef} — Effective span of beam or slab or effective length of column

l_{ex} — Effective length about x-axis

l_{ey} — Effective length about y-axis

l_n — Clear span, face-to-face of supports

l'_n — For shorter of the two spans at right angles

l_x — Length of shorter side of slab

l_y — Length of longer side of slab

l_0 — Distance between points of zero moment in a beam

l_1 — Span in the direction in which moments are determined centre-to-centre of supports

l_2 — Span transverse to l'_1 centre-to-centre of supports

l'_2 — l_2 for the shorter of the continuous spans

M — Bending moment

m — Modular ratio

n — Number of samples

P — Axial load on a compression member

q_0 — Calculated maximum bearing pressure of soil

r — Radius

s — Spacing of stirrups or standard deviation

T — Torsional moment

t_w — Thickness of wall

V — Shear force

W — Total load

WL — Wind load

w — Distributed load per unit area

wd — Distributed dead load per unit area

wl — Distributed imposed load per unit area

x — Depth of neutral axis

Z — Modulus of section

z — Lever arm

α, β — Angle or ratio

γ_f — Partial safety factor for load

γ_m — Partial safety factor for material

δ_m — Percentage reduction in moment

Σ_{cc} — Creep strain of concrete

σ_{cbc} — Permissible stress in concrete in bending compression

σ_{cc} — Permissible stress in concrete in direct compression

σ_{mc} — Permissible stress in metal in direct compression

σ_{sc} — Permissible stress in steel in compression

σ_{st} — Permissible stress in steel in tension

σ_{sj} — Permissible tensile stress in shear reinforcement

τ_{bd} — Design bond stress

τ_{c} — Shear stress in concrete

τ_{cmax} — Maximum shear stress in concrete with shear reinforcement

τ_{v} — Nominal shear stress

ϕ — Diameter of bar

4. STRUCTURAL REQUIREMENTS AND DESIGN PRINCIPLES

4.1 Building Stability and Assembly of Prefabricates

4.1.1 The safety and serviceability of the prefabricates and the entire structures shall be checked both for construction stage which includes production, storage, transportation and assembly of the prefabricates as well as the stage after which the building has been completed.

4.1.2 Building stability shall be provided by means of frames, shear walls, shear cores or by any other suitable methods connected by horizontal diaphragms provided by the floors or by any other means. Where shear walls are used these shall preferably be distributed evenly over the whole building to reduce deformation due to torsion. The shear walls should extend from the foundation to the top of the building to avoid torsion in the floors.

4.1.3 In the stage of the completed building, all the prefabricates offering stability of the building as a whole shall be mutually connected by proper joints and/or tie-beams over supports of floor and around the building to ensure their interaction and offer structural response under both horizontal and vertical loads.

4.1.4 The detailing of the joint shall take into account the behaviour of the structure after local failure of a room size portion of the wall. In order to diminish the risk of progressive collapse an alternate load path capable of maintaining the stability of the structure shall be provided.

4.1.5 The joints shall be so designed as to take care of shrinkage, temperature changes and any other aspects. Vertical joints between external wall panels shall be so designed and built that water tightness and structural integrity are assured. Due consideration shall be given to the capillary action, pressure built up by the beating of rain, shrinkage of joint, etc, provision of external sealants, water barrier and a vertical drainage canal are suggested. Some suggested configurations of joints are given in Fig. 1.

It is also necessary that the *in-situ* joints have a minimum of width for concreting have loops or projecting reinforcements from side panels, and vertical bars in joints (*see* **4.6.5**).

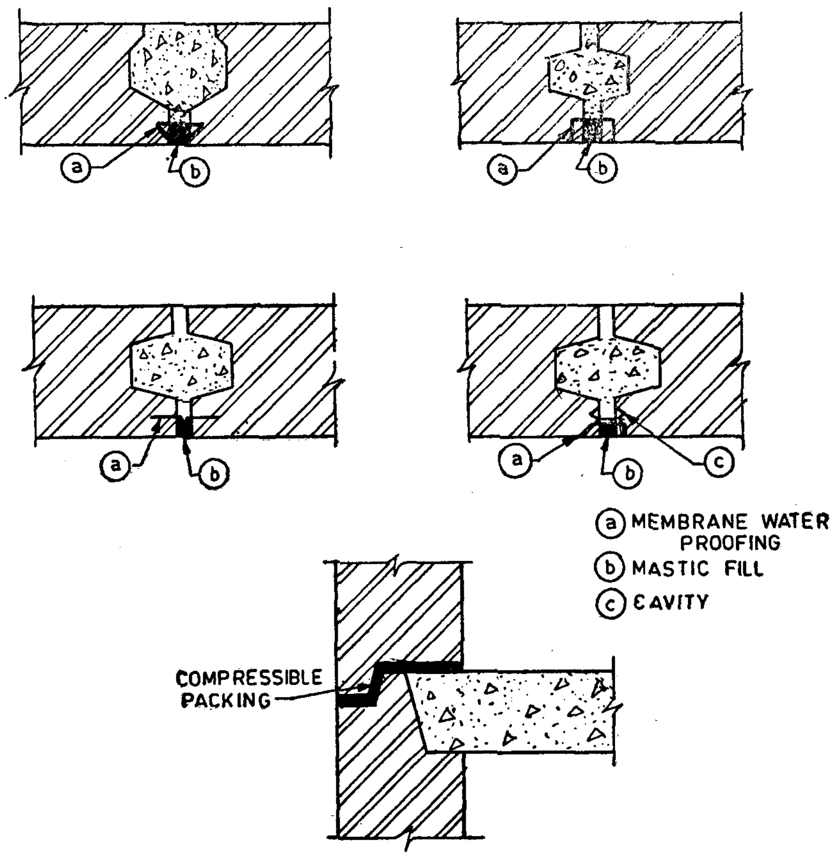


FIG. 1 TYPICAL JOINTS

4.1.6 For panels the adequacy of reinforcement should be checked for frictional forces due to adhesion between mould and panel during demoulding.

4.1.7 It is recommended that the wall panels are cast on a tilting mould so that the demoulding of the panels done vertically.

4.2 Expansion Joints — Provisions of expansion joints as specified in **26** of IS : 456-1978* shall be followed.

4.3 Tolerances — Permissible tolerances shall be as per **4.3.1** and **4.3.2**.

4.3.1 *Casting Tolerances*

a) Length/breadth : ± 0.1 percent subject to maximum of $+5$ mm or -10 mm

b) Thickness : ± 2 mm up to 300 mm
± 3 mm $>$ 300 mm

c) Bow to straightness : $\frac{1}{750}$ of the larger dimension subject to maximum of ± 5 mm

d) Squarenss in plane : Longer side considered straight, the shorter side may not be out of square line for more than $+2$ mm and -5 mm at end of the line. Imperfections along the considered square lines to the same extent of $+2$ mm and -5 mm may be ignored (*see* Fig. 2)

e) Twist in plane : Straight lines joining the four corners of plane shall not show deviation out of plane more than $\pm \frac{1}{1\,500}$ of the dimension or ± 5 mm, smaller of the values shall be considered as the permissible tolerances (*see* Fig. 3)

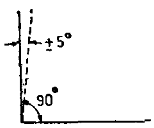


FIG. 2 SQUARENESS IN PLANE

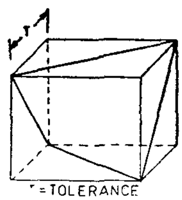


FIG. 3 TWIST IN PLANE

*Code of practice for plain and reinforced concrete (*third revision*).

f) Flatness : The maximum deviation from a straight edge placed between two points, 1·5 m apart, shall not be more than ±3 mm

4.3.2 *Erection Tolerances*

a) Level differences between suppport lines of floor or wall panels	±5 mm
b) Plumb lines of wall panels	±5 mm
c) Bearing for precast floor panels	±5 mm
d) Joint dimensions	±5 mm

e) Maximum accumulated deviation $\frac{1}{1\,250}$ of height/length or ±20 mm whichever is smaller

4.4 Floor Panels

4.4.1 *Requirements on Concrete Strength and Geometry* — For reinforced concrete floors, concrete of minimum grade M 20 is recommended. Prestressed concrete floor units shall satisfy the strength requirements followed in usual design practice, namely, a minimum of M 35 for post-tensioned works and a minimum of M 40 for pretensioned work. However, when reinforced concrete panels are prestressed, for resisting handling stresses using deformed bars, a lower concrete strength may be acceptable subject to the approval of designer. For floor using cellular concrete blocks as filler blocks, a minimum strength of 5 N/mm^2 is recommended. Floors incorporating clay blocks or other fillers shall have such blocks of a minimum strength of 5 N/mm^2.

4.4.2 The thickness of the floor panel shall be such that the serviceability requirements are satisfied. The minimum thickness of concrete layer for cored slab is 35 mm. Ribbed slabs shall have a minimum thickness of 35 mm for the compression flange. In cases where structural screeding concrete is laid on the precast units, its minimum thickness shall be 35 mm. In case of non-structural screed a thickness of 25 mm for the screed may be sufficient.

NOTE — In corrosive climates, the minimum thickness should be increased depending upon cover requirement.

4.4.3 Panels may be designed in accordance with the recommendations given in IS : 456-1978* governing reinforcement and detailing. In addition, the anchoring of lifting hooks, provisions of distribution steel around

*Code of practice for plain and reinforced concrete (*third revision*).

lifting anchors, minimum steel for its location and shape near supports and at joints with *in-situ* beams around shall also be considered and provided for.

4.4.4 The end cover for the bottom reinforcement shall be not more than 5 mm to avoid undesirable displacement of the reinforcement during production. The reinforcement at the bottom of floor panel shall also have sufficient anchorage at the support. When the cross reinforcement is not welded to the main reinforcement, the minimum length of the bar behind the support edge shall be at least 5 times the diameter of the bar (*see* Fig. 4 A). When the cross reinforcement is welded to the main reinforcement, the minimum length may be reduced to 4 times the diameter of bar. In case of supports formed by brackets these values may be increased by 25 percent. The above values correspond to panels not reinforced for shear. At least 1/3 of the total main reinforcement at midspan shall be taken up to the support. When the panels are to be reinforced for shear, the minimum length of the bar behind the support edge shall be at least 15 times the diameter of bar provided that not less than 1/3 of the total bars at midspan are taken up to the support. This may be reduced to 10 times the diameter of bar when 2/3 of the total bars at midspan are taken up to the support (*see* Fig. 4B).

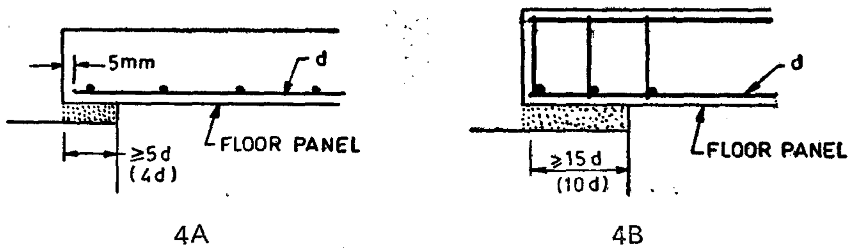
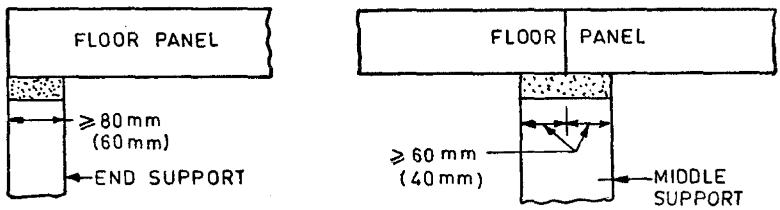


FIG. 4 ANCHORAGE LENGTH FOR REINFORCEMENT

4.4.5 When the floor is continuous, the area of cross-section of the top bars of the panel is provided as per the calculations and in accordance with general requirements for reinforced concrete. In this case the length of the projected bar shall be such that it will develop full anchorage. In the case of simply supported floor panels in which negative bending moment is caused due to fixity provided by the walls at the supports, the top reinforcement of the panel shall be at least 0·2 times the area of the main reinforcement at bottom. This reinforcement may be in the form of projected bars or loops. It may also be welded. When top bars are projected the length of the bar beyond the support shall be at least 15 times the diameter of the bar for anchorage. The length of the top bars shall not be less than 0·15 of the span of the floor panel.

4.4.6 For cellular concrete panels, top reinforcement shall be provided throughout the length of the panel (*see* IS : 6073-1971*). The cross-sectional area of the top reinforcement shall be at least 1/3 of the main reinforcement. Anchorage of main reinforcement by welded cross bars is essential for cellular concrete panels and they shall be placed at close spacings near the ends. The number of distribution bars that are to be provided at the ends depends on the diameter of longitudinal and distribution reinforcement, and the density of the concrete.

4.4.7 *General* — The requirement for width of support for floor panels depends on whether they are spanning in one direction or two directions and whether they have top reinforcement for anchorage. The width of the support shall be sufficient to ensure proper anchorage for the bottom reinforcement. For floors supported freely on two edges, a minimum bearing of 80 mm is recommended. In particular cases of good workmanship this may be reduced to 60 mm. For continuous supports the bearing for individual panels on either side of supports shall be at least 60 mm in general and may be reduced to 40 mm in case of good workmanship and if there are temporary supports and projected bars with full anchorage. For floors supported on three or four edges, a minimum width of 40 mm for support is recommended (*see* Fig. 5).



5A Floor Panel Supported Freely on Two Edges

5B Floor Panel with Continuous Supports

FIG. 5 BEARING LENGTH FOR FLOORS

4.4.8 In case mortar layer is required for levelling, the thickness of mortar layer shall not exceed 15 mm. The mortar shall have a minimum compressive strength of 20 N/mm^2 or equal to the strength of panel it supports.

4.4.9 The joint along the longitudinal edges of the floor panel (in the direction of the span in between the suppoıts) shall be able to resist the

*Specification for autoclaved reinforced cellular concrete floor and roof slabs.

forces acting in the joint due to differential loading of connected panels and to interact in the transverse direction. They shall also resist shear forces in the plane of the floor. The details of the keyed joint as shown in Fig. 6 which may or may not be provided with screeding. A minimum gap of 40 mm for these joints is recommended between the panels. The depth of the joint shall be at least 75 percent of the total depth of the floor panel. When screeding concrete is provided for structural purposes it is recommended to provide screeding concrete over the precast units with a positive connection with the joint or as an alternative a minimum reinforcement of 0·5 cm^2/m length of the panel. The gap in the joint shall be filled with cement mortar having a maximum compressive strength one grade higher than the connecting members.

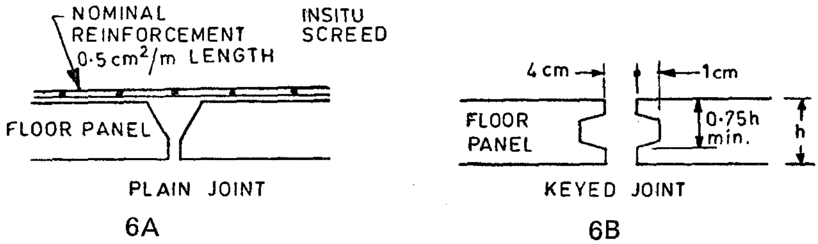


FIG. 6 JOINT BETWEEN FLOOR PANELS ALONG LONGITUDINAL EDGES

4.4.10 The joint between floor panel over support may be continuous or hinged depending upon the type of structure (*see* Fig. 7). In the case of floor panel, it is recommended to project top steel bars into the joint. For buildings in high seismic zones, continuous connection may be achieved by projected or overlapped bars or by welding the bars with adequate steel plates or by providing loops (when loops are provided they shall be tied to the longitudinal bars in the joint as shown in Fig. 7D).

4.5 Walls

4.5.1 *General* — Internal structural walls may be homogenous walls of cement concrete (reinforced or unreinforced) or light-weight concrete. It is recommended to design walls as plain concrete with secondary reinforcement for production and erection purposes only and which is not taken into account while checking the safety of the wall. For plain concrete walls, concrete of grade not less than than M 10 shall be used. In case steel is required in the panel, the joint should be designed to provide continuity in bars.

4.5.2 *Slenderness Effects* — The minimum thickness of walls shall be 100 mm from the slenderness and production points of view. Smaller thickness may be applied to concrete box ribbed or other similar units. While designing the structural walls, their slenderness ratio shall also be

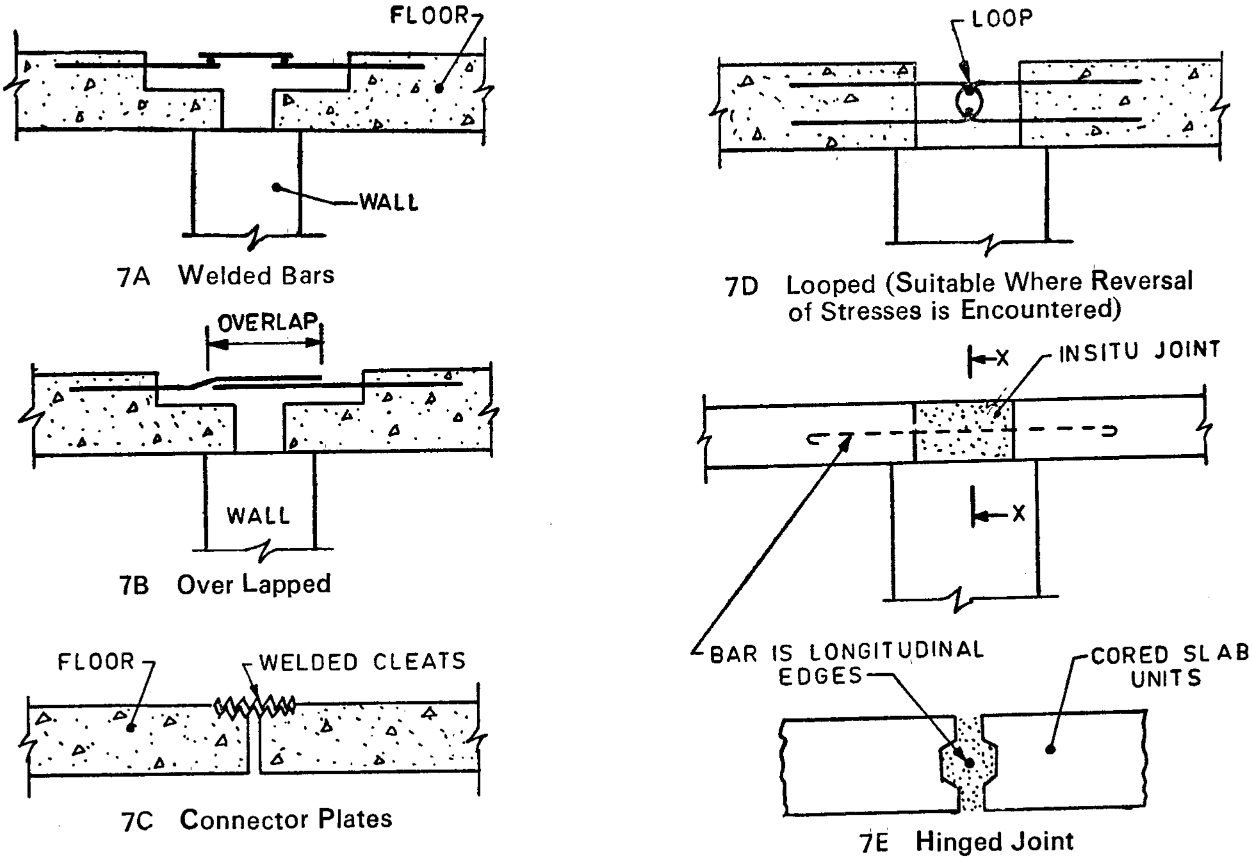


FIG. 7 REINFORCEMENT OVER SUPPORT

considered. Where the wall is stiffened by cross walls, the slenderness ratios may be determined as in IS : 1905-1980*.

4.5.3 *Reinforcement in Plain Concrete Walls* — The wall panels designed as plain concrete walls shall be provided with minimum reinforcement so as to resist the forces in the wall panel including those due to shrinkage of concrete occurring during different stages of production, transportation, storage and assembly.

In case of wall panels having a width greater than 2 m reinforcement along the perimeter is recommended (*see* Fig. 8A). The area of vertical and horizontal reinforcement along the perimeter shall be 1 cm^2 and shall consist of a minimum of 2 bars. The two bars shall be connected by means of ties at a spacing of not less than 0·5 m. For panels of length over 3·6 m it is recommended to provide additional vertical reinforcement in the middle similar to the vertical peripheral reinforcement (*see* Fig. 8B). In case of wall panels with openings additional reinforcement around the openings shall be provided (*see* Fig. 8C). The bars around the opening shall be provided for a distance of not less than 50 times the diameter of bars behind the edges of the opening. Inclined reinforcement may also be provided at the corners. The vertical segments of wall panels (for example, separated by openings) and having a width less than 500 mm shall be reinforced with a minimum of 4 numbers of 12 mm diameter bars with stirrups at a distance of 300 mm (*see* Fig. 8D) segments wider than 500 mm shall follow reinforcement as for around openings.

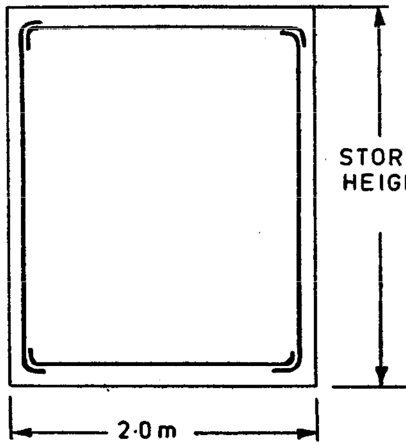
4.5.4 *Reinforcement in Reinforced Concrete Walls* — The amount of reinforcement in reinforced concrete wall panels shall be designed according to normal design practice. The minimum ratio of the vertical reinforcement to gross concrete area shall conform to IS : 456-1978†.

4.6 Joints

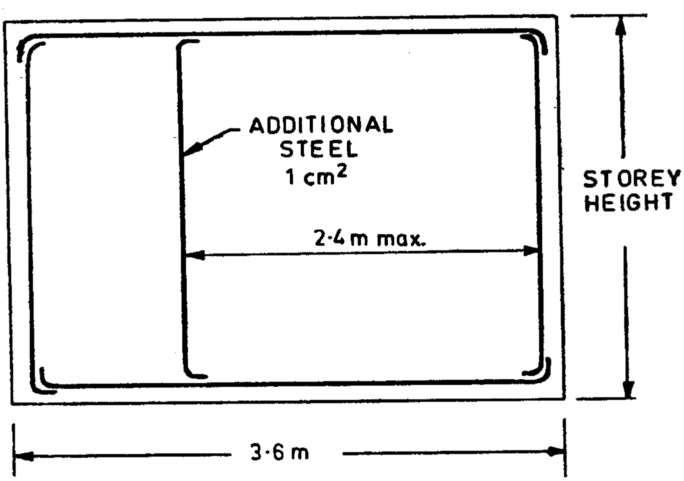
4.6.1 Joints between internal and external wall panels shall be designed to resist the forces acting on them without excessive deformation and cracking. They shall also be able to accommodate the deviations in the dimensions of the wall panels during production and erection.

4.6.2 The horizontal joint for wall panels depends on their method of positioning. Wall panels may be positioned on mortar bed, on bolts projecting from the lower panel or on projections from the floor panels. When wall panels are positioned on mortar bed laid on lines which are marked on the layout prior to positioning (Fig. 9A and 9B), the thickness of mortar shall not exceed 15 mm. The minimum compressive strength of the mortar shall be 20 N/mm^2 or be of the same grade as of panel.

*Code of practice for structural safety of buildings: Masonry walls (*second revision*).
†Code of practice for plain and reinforced concrete (*third revision*).



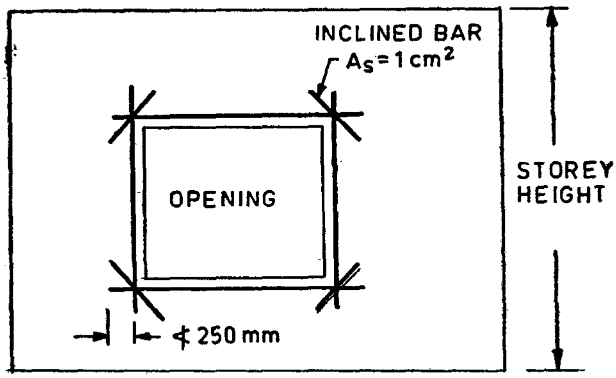
8A Reinforcement in Walls Having Width Greater than 2 m but Less than 3·6 m



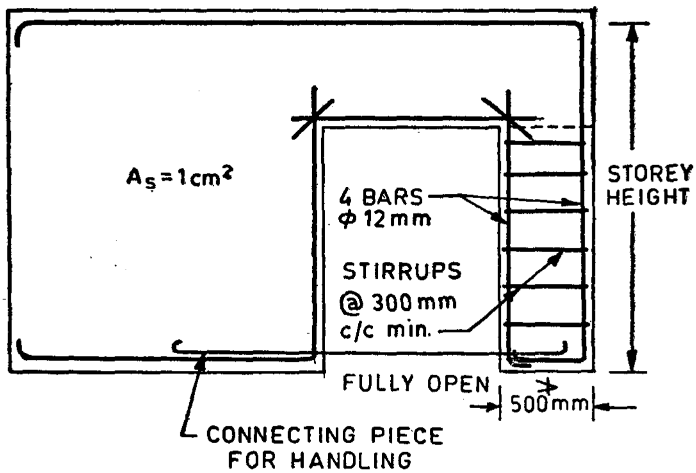
8B Vertical Bars in Walls Having Width 3·6 m or More

FIG. 8 REINFORCEMENT IN PLAIN CONCRETE WALLS—*Contd*

When the panels positioned by means of levelling bolts, care should be taken to release the load on the bolts after the concrete has set.



8C Reinforcement Around Openings



8D Reinforcement in Narrow Segments

FIG. 8 REINFORCEMENT IN PLAIN CONCRETE WALLS

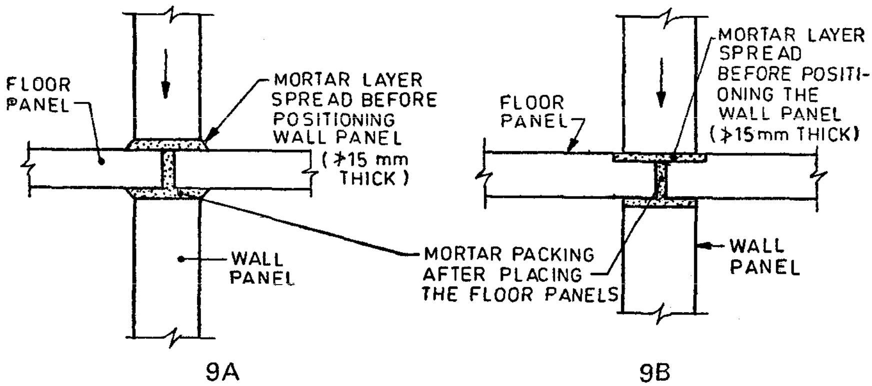


FIG. 9 HORIZONTAL JOINT BETWEEN WALL PANELS POSITIONING ON MORTAR BED

4.6.3 Vertical joints between wall panels shall be designed for the vertical shear forces induced in the joints by horizontal loading of the structure like wind force, seismic force, etc, as well as by differential vertical loading of the wall panels. Vertical joints may be designed with vertical grooves (Fig. 10A) or with keys (Fig. 10B). In the case of grooved joints the resistance is by the friction between the wall panel and the concrete in the joint and by the interaction of the tie-beam. In the case of keyed joints the shear forces will be resisted by the projections or castellations which act as shear-keys in addition to the friction between the concrete and the wall panels.

4.6.4 The shape and dimensions of the keyed joint shall be at least 100 mm. Its area of cross-section shall be at least 120 cm^2. The minimum compressive strength of concrete in the joint shall be same as of panels.

4.6.5 From the point of view of sealing of joints in external walls two essential types of joints may be suggested, namely, filled and open joint. Open joints may be unprotected or protected against direct entrance of water. In case of open joints, the width of the gap shall be sufficiently wide to allow movement of the panels due to temperature changes. Filled joints require flexible sealing compounds or plastic seals capable of absorbing long-term and cyclic movements. Polysulphide compounds or other sealants which take up movement of the panel and prevent water penetration are preferred (*see* Fig. 11). Continuity of protective membrane in the joints shall also be ensured (suggested scheme given in Fig. 12).

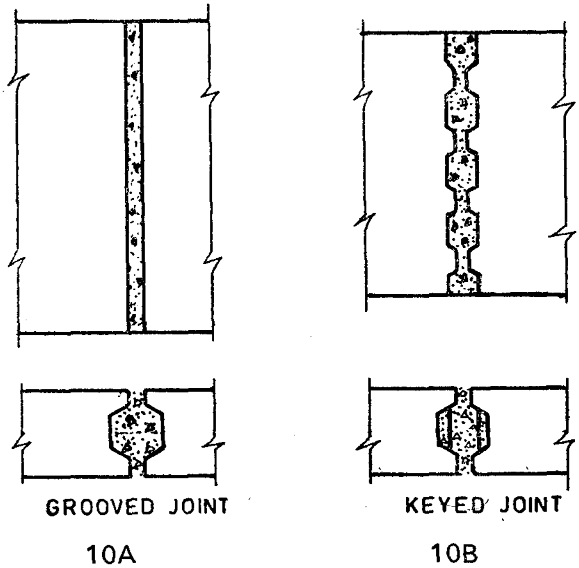


FIG. 10 VERTICAL JOINT BETWEEN WALL PANELS

4.7 Tie-Beams

4.7.1 The tie-beams shall be provided at each floor level along all structural walls and along the perimeter of the building to obtain a monolithic action of walls and floors, and to limit the possibility of progressive collapse. Tie-beams may be designed as monolithic ones constructed at site during assembly or hidden ones constructed by connecting the bars placed on the floor panels (*see* Fig. 13). The reinforcement in the tie-beam shall be at least 2·5 cm^2 when the distance between the internal longitudinal structural walls is not more than 4·5 m. In case the distance between the longitudinal structural walls is between 4·5 and 6 m, the reinforcement shall be increased to 4·0 cm^2. The effective cross-sectional area of the tie-beam shall not be less than 75 cm^2. Concrete of minimum grade M20 shall be used for the tie-beam.

4.7.2 When hidden tie-beams are used, the tie-beam reinforcement shall be provided as near to the support or longitudinal edge as possible and shall be provided within a distance of 500 mm from the ends. The reinforcement of the tie-beam shall be continuously connected (*see* Fig. 14).

4.8 Erection and Assembly — The erection and assembly of wall panels and floor panels shall be done as per design. The sequence of erection and assembly shall be planned in advance taking into consideration the stability

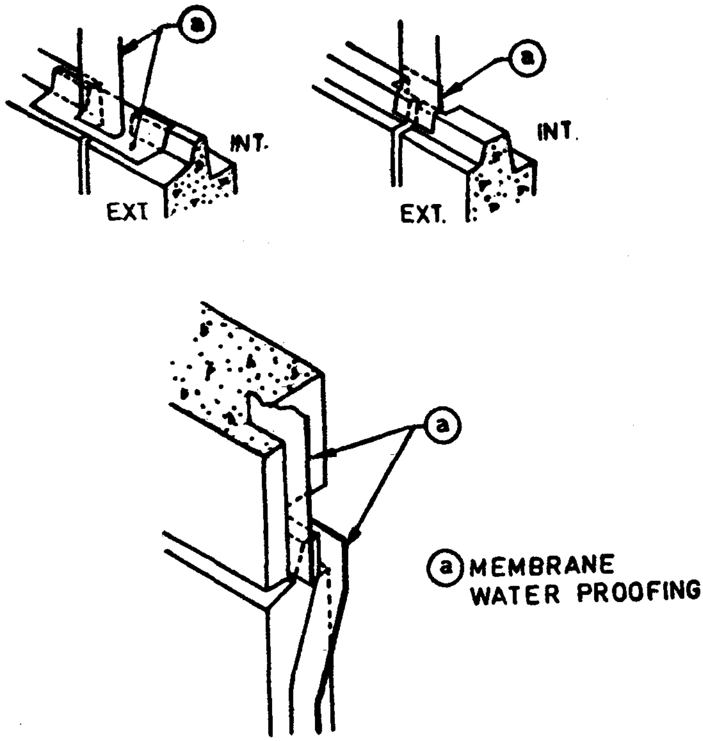


FIG. 11 TYPICAL DETAILS OF FILLED JOINTS WITH SEALANTS

of the panels during erection and shall have the approval of the engineer-in-charge. Adjustable steel props are recommended for supporting the wall panels after erection till the joints attain enough strength to withstand the erection stresses. The adjustable in plumb of wall panels are done with the help of wall props. For the purpose of lifting and erection, inserts or other suitable devices have to be buried inside the panels at the time of casting. Mild steel inserts and hair pin loops are normally used. Some typical inserts are shown in Fig. 15. The location of such inserts also needs to be carefully and optimally designed so that the stresses in the panels during handling will be minimum. Inserts required for services can be planned and fixed wherever necessary. The stresses induced in the panels due to erection shall be within the permissible limits. Joints for external walls, in addition to their structural efficiency, shall be watertight

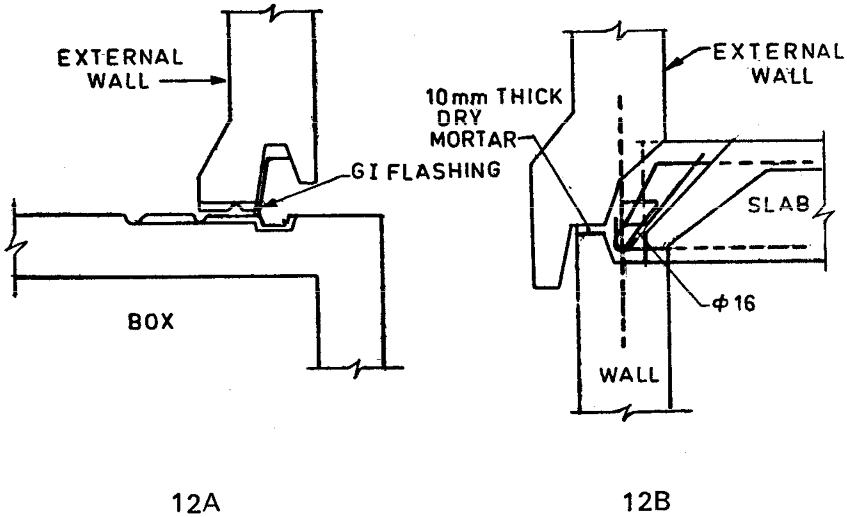


FIG. 12 TYPICAL DETAILS OF JOINTS

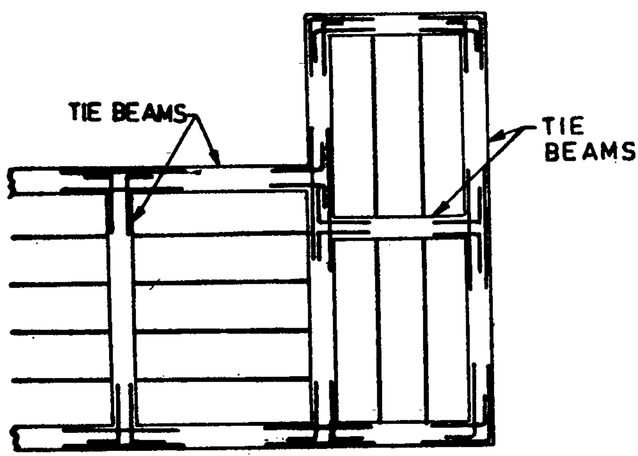


FIG. 13 TIE-BEAMS AT FLOOR LEVEL

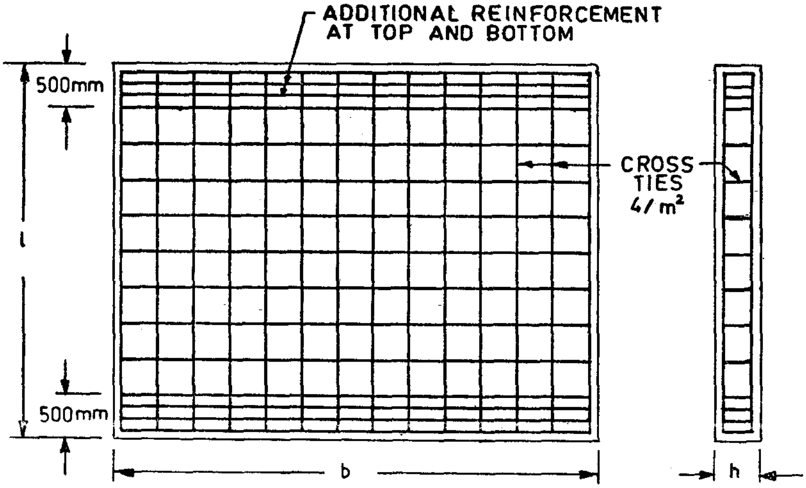
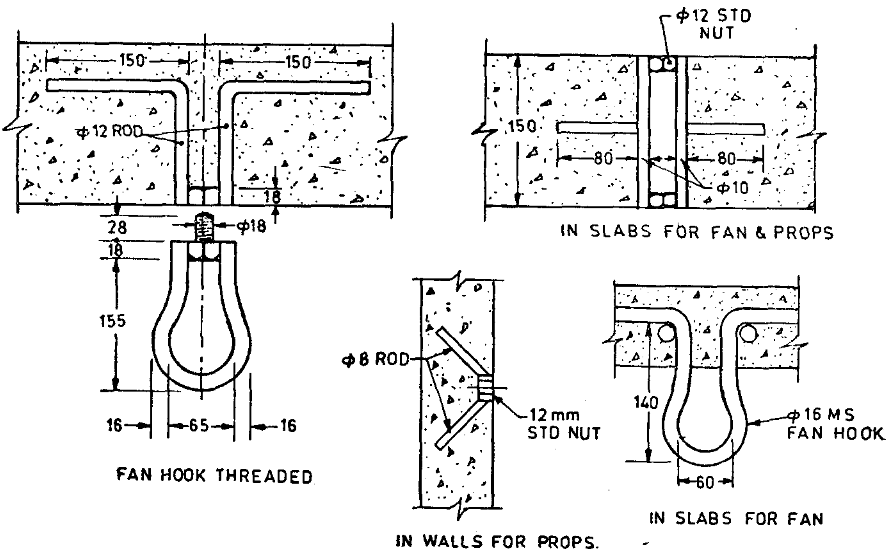


FIG. 14 DETAILS OF REINFORCEMENT IN HIDDEN TIE-BEAMS



All dimensions in millimetres.

FIG. 15 TYPICAL SKETCH FOR INSERTS FOR SUPPORTING PROPS AND FANS and shall allow free movement of the walls due to temperature changes. This can be achieved by suitably shaping the joint or by sealing compounds. The sealing compound shall be approved by the engineer-in-charge and shall have a long life to avoid frequent repair of the joints.

The load factor for evaluating of structural strength during erection can be assumed as 1·2. However, in case of inserts and fixtures left in precast element for handling and erection purposes dynamic effects due to erection have to be taken into consideration in the design.

4.9 Special Requirements for Buildings in High Seismic Zones

4.9.1 Large panel buildings in high seismic zones are recommended to be constructed with closed symmetrical layout. The verandahs shall be designed, as an integral part of the building or as separate construction with pliable connections with the remaining part of the building. The height of the building shall generally be restricted to 10 storeys in Zone IV and 7 storeys in Zone V. Buildings up to 5 storeys in Zone IV with a close symmetrical layout and load-bearing cross walls at not more than 4 m spacings and with floor panels supported on the longitudinal walls (in addition to the support on the cross walls) may be designed as per the structural requirements for high-rise buildings in low seismic zones.

NOTE — For taller buildings more than specified in **4.9.1** model analysis shall be made.

4.9.2 It is recommended to use solid concrete panels for floors for buildings in high seismic zones. In case of ribbed panels the thickness of flange shall not be less than 50 mm and they shall be connected together by joints in between the ribs only. The floor panels shall be designed with continuous connection over supports. The continuous reinforcement shall consist of projected bars at top and bottom with a minimum cross-sectional area of 1·5 cm^2/m length of the panel. The projected bars may be welded or in the form of loops. The edges of the floor panels shall also have castellations to provide keyed joint.

4.9.3 Concrete walls shall be reinforced to resist the shear and tensile forces developed by earthquake.

4.9.4 The vertical and horizontal joints for wall panels shall be designed as keyed joints. The vertical joint shall be reinforced with projected bars welded or overlapped or loops and with vertical bar. The minimum area of transverse reinforcement in the vertical joint shall be 2·0 cm^2 for one storey height. The joint shall have access for connecting the reinforcement. For the horizontal joint, the wall panels shall be connected by a minimum vertical reinforcement of 0·7 cm^2/m length of the horizontal joint.

4.9.5 The building shall have tie-beams along the external edges in vertical and horizontal directions to ensure interaction of prefabricates. The floor and wall panels shall have keys along their edges and shall be connected by continuous reinforcement. A minimum reinforcement of 4·0 cm^2 shall be provided in the tie-beams and the connections shall be provided by loops or by welding and not by overlapping.

5. STRUCTURAL ANALYSIS

5.1 General — The design of prefabricated components should be undertaken primarily to ensure an adequate margin of safety against the limit state of collapse being reached. Generally, this is achieved by ensuring that the design strength of member is greater than or equal to the design load. Characteristic and design values and partial safety factors shall be in accordance with IS : 456-1978*. In addition the following complimentary safety factor shall be applied:

$\gamma_s = \gamma_{s1}, \gamma_{s2}, \gamma_{s3}$

— For wall panels with a horizontal cross-sectional area

$\leqslant 0{\cdot}06$ sqm, $\gamma_{s1} = 1{\cdot}1$

— For wall panels with percentage of vertical reinforcement in the range of $0{\cdot}14 \leqslant P - 0{\cdot}4$, $\gamma_{s2} = 1 - 0{\cdot}7\,P$

— For the stages of construction including verification of stability during construction and during accidental damages

$\gamma_{s3} = 0{\cdot}8$

— In all other cases

$\gamma_{s1} = \gamma_{s2} = \gamma_{s3} = 1{\cdot}0$

The designer responsible for the overall stability of the structure should ensure the compatibility of design and details of parts and components. To ensure a robust and stable design, it will be necessary to consider the layout of structure on plan, interaction between intersecting walls and rigidity of joints.

5.2 Loads — Analysis for the stability of large panel buildings covers mainly two types of lateral forces, namely, wind and earthquake forces. The magnitude and distribution of the wind and earthquake loads shall be

*Code of practice for plain and reinforced concrete (*third revision*).

taken in accordance with IS : 875-1964* and IS : 1893-1975† respectively. In the case of earthquake forces the base shear calculated in accordance with IS : 1893-1975†, shall be increased by 5 percent for each storey beyond the fifth storey subject to a maximum value of 25 percent.

5.2.1 *Distribution of Lateral Forces* — The distribution of lateral forces among individual shear walls is based on the following assumptions:

a) The direction of lateral forces is parallel to the shear walls considered,

b) The shear walls are connected by infinitely rigid horizontal diaphragms provided by the floors, and

c) The shear walls behave elastically under the action of lateral forces and the distribution of lateral forces occurs according to the rigidity of the shear walls.

The distribution of the loads shared by the shear walls on the assumptions made above is valid so long as the rigidity of the shear wall under consideration is not reduced below 70 percent of its rigidity before cracking.

5.2.2 For buildings with closed-layout the load shared by wall is given as

$$w_i = \frac{W}{\alpha_i \sum_{j=1}^{n} \frac{1}{\alpha_i}} \qquad \dots (1)$$

where

W = total lateral forces on the building, and

α_i = flexibility coefficient defining the magnitude of deflection of wall i due to uniformly distributed load of $w = 1$.

5.2.3 For building with open-layout the load shared by wall i is given as

$$w_i = w \left\{ \frac{\bar{\alpha}_i}{\sum_{j=1}^{n} \alpha_j} \pm \frac{r\, \bar{\alpha}_i\, r_i}{\sum_{j=1}^{n} r_j^2\, \bar{\alpha}_j} \right\} \qquad \dots (2)$$

where

r = distance of the point of action of W from the centre of rotation, and

*Code of practice for structural safety of buildings: Loading standards (*revised*).
†Criteria for earthquake resistant design of structures (*third revision*).

r_i = distance of the shear wall i from the centre of rotation.

$$\overline{\alpha_i} = \frac{\alpha_o}{\alpha_i}$$

where

α_o = flexibility coefficient of the least rigid wall, and

α_i = as defined in Eq. (1).

The cartesian coordinates of the centre of rotation are given by

$$\alpha_o = \frac{\sum_{j=1}^{n} \frac{1}{\alpha_j} x_j}{\sum_{j=1}^{n} \frac{1}{\alpha_j}} \qquad \text{... (4)}$$

where

x_j = distance of the wall j from the y-axis, and

α_j = flexibility coefficient defining the magnitude of deflection of wall j under a unit force $w = 1$ parallel to y-axis

$$y_o = \frac{\sum_{j=1}^{n} \frac{1}{\alpha_j} y_j}{\sum_{j=1}^{n} \frac{1}{\alpha_j}} \qquad \text{... (5)}$$

where y_j = distance of the wall j from the x-axis

j = flexibility coefficient defining the magnitude of deflection of wall j under a unit force $w = 1$ parallel to x-axis

For buildings over 50 m in height, a more precise analysis preferably coupled with dynamic and model analysis for distribution of lateral forces is recommended.

5.3 Analysis of Shear Walls — In the computation of stiffness and structural response of shear walls it is permitted to neglect the influence of:

a) deformability of the wall in its own plane;

b) deformability of horizontal joints; and

c) deformability of lintels if their height to clear span ratio (hl/s) is greater than 0·8. In this case, the statical model for analysis shall be assumed as a solid cantilever.

When hl/s $<$ 0·8, any one of the following statical models for shear walls may be assumed:

a) banded cantilever model with discrete lintels represented by a continuous laminae,

b) multi-storey frame model, and

c) interconnected mesh using finite element technique.

The banded cantiliver method is recommended for the analysis of shear walls in large panel buildings.

The interaction of mutually connected walls can be considered in the analysis of shear walls. The segments of intersecting walls perpendicular to the considered ones measured in both directions from the axis of vertical joint shall not be greater than the width of the perpendicular wall to the nearest row of opening or 0·2 H whichever is less.

In the case of composite shear walls, their equivalent cross-sectional dimensions based on their respective modulii of elasticity shall be considered.

The statical model for a shear wall with plastic joints shall be a banded cantilever with the plastic joints represented by a continuous laminae subjected to tangential shear forces.

5.3.1 Usually the joints in wall panels are consider to be rigid. However, when the magnitude of shear forces caused by the lateral loads is such as to cause plastic deformations of the joints, the analysis of walls shall be based on the assumption of plastic joint.

5.4 Floors

5.4.1 In structural analysis floors are to be checked at the serviceability as well as ultimate states. A check on deflection is very essential at the serviceability state since excessive deflections are likely to cause cracking in the partition walls, cladding, finishes, etc. Allowable deflections and span to depth ratios of floor slabs shall conform to IS : 456-1978*.

5.4.2 *Ultimate State of Floor Panels* — The values of bending moments in floor panels supported along two edges shall be calculated for the ultimate state as follows:

a) Floor panels made continuous by providing top reinforcement

*Code of practice for plain and reinforced concrete (*third revision*).

— interior spans

middle span moment $= \frac{q_1 l^2}{8} + \frac{q_2 l^2}{16}$ ... (6)

support moment $= \frac{q_2 l^2}{15}$ ... (7)

— end spans

middle span moment $= \frac{q_1 l^2}{8} + \frac{q_2 l^2}{10}$ ... (8)

support moment on the continuous end $= \frac{q_2 l^2}{15}$ ... (9)

where

l = span of floor panel,

q_1 = load acting on floor panels before connecting the reinforcement over supports, and

q_2 = load on floor panels after connecting the reinforcement over supports.

When the floor spans in a continuous floor do not differ by more than 15 percent the longest span shall be taken as l in the above calculations.

b) Simply supported floors

span moment $= \frac{q\,l^2}{8}$ ... (10)

where

q = total load acting on floor panels ($q_1 + q_2$)

For floor panels supported on three or more edges it is recommended that the design moments are to be calculated as for freely supported members.

5.4.3 *Fixity Due to Walls at the Serviceability State* — The floor panels should be provided with top reinforcement to meet fixing moments created by walls. The fixing moments created by walls at the serviceability state for the end support of continuous floors and simply supported floors shall be taken as given below:

— for floors with a bearing less than 100 mm

fixing moment at support $= \frac{q_1 l^2}{30}$ ... (11)

— for floors with bearing more than 100 mm

$$\text{fixing moment at support} = \frac{q_2\, l^2}{16} \qquad \ldots (12)$$

where

q_2 = load acting on floor panels after the walls have been built without enhancing by safety factors.

5.4.4 *Distribution of Loads in the Direction Perpendicular to the Plane of the Floor* — When the load applied to one of the floor panels is greater than the loads applied to the neighbouring ones, the interaction of the floor panels may be considered provided they are connected.

a) *Ultimate state* — Additional loads acting on individual floor panels at ultimate state shall be assumed to be distributed to the adjacent panels at the ultimate state as given in Fig. 16.

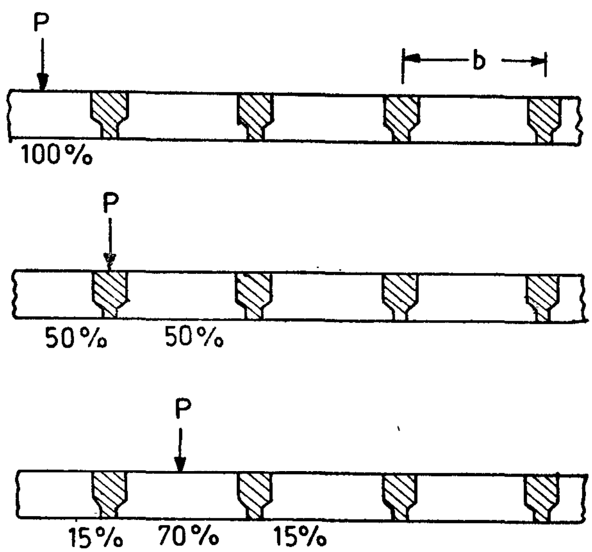


FIG. 16 DISTRIBUTION OF LOADS ON FLOOR PANELS ULTIMATE STATE

b) *Serviceability state* — For the purpose of checking deflection, the distribution of load may be assumed as shown in Fig. 17. When the number of floor panels is less than that given in Fig. 17, load

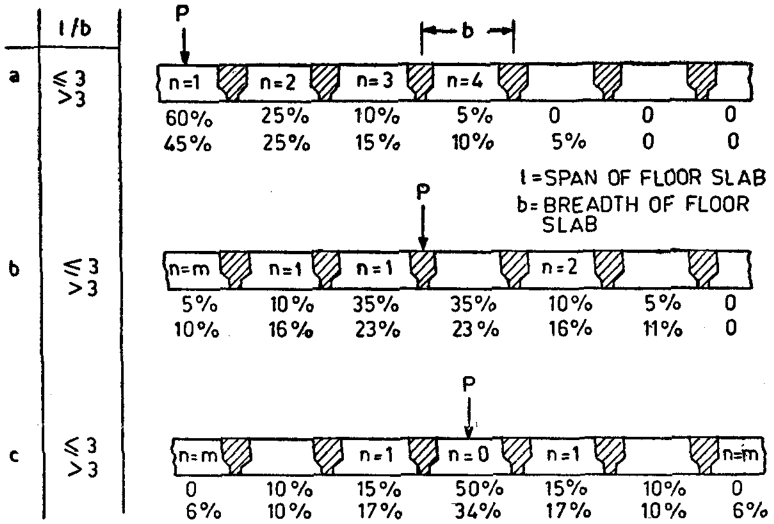


FIG. 17 DISTRIBUTION OF LOADS ON FLOOR PANELS SERVICEABILITY STATE

W_i carried by the considered floor panel shall be computed from the formula:

$$W_i = p \frac{x_i}{\sum_{i=1}^{k} x_i} \qquad \text{.. (13)}$$

p = load other than the dead load of floor panels.

x_i = number expressing the percentage of load transferred by a considered floor panel, assuming the number of floor panels as in Fig. 17.

$\sum_{i=1}^{k} x_i$ = sum of x_i values for given number of floor panels k.

5.5 Walls

5.5.1 *Scope of Analysis* — While analyzing the structural walls the safety of the following portions of the wall shall be checked:

a) in zone close to the mid-height of the wall (Zone 1),

b) at supports of floors (Zone 2),

c) lintels, and

d) vertical and horizontal joints.

While calculating the structural response of load-bearing walls, the interaction of self-supporting walls placed in a perpendicular direction is usually neglected but it can also be considered when the exact analysis is required.

5.5.2 *Zone 1 of the Wall* — The structural response in the Zone 1 of the wall is a function of the eccentricity, slenderness and support conditions. Walls can be supported on horizontal edges and on one or both of its vertical edges as shown in Fig. 18. The most common case is a wall supported along its horizontal edges only.

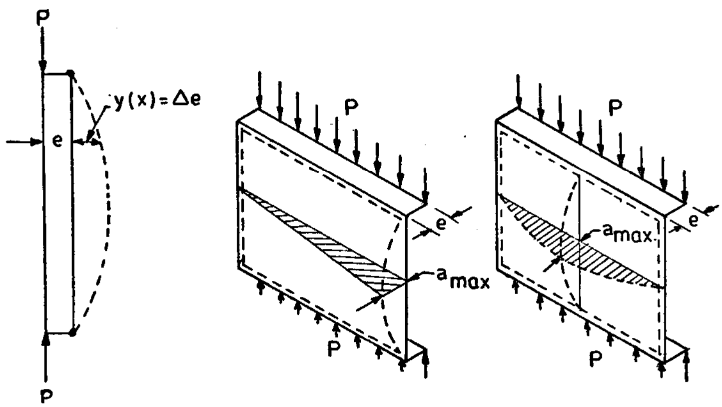


FIG. 18 SUPPORT CONDITIONS OF WALL

a) *Eccentricities* — A simplified approach for calculating the response of the wall in Zone 1 is based on the assumption that the wall is a bar loaded with equal eccentricities from above and below. The total eccentricities shall be assumed as given below:

for external walls $e = 0{\cdot}20\ t_w$ subject to a maximum of 30 mm ... (14)

for internal walls $e = 0{\cdot}15\ t_w$ subject to a maximum of 20 mm ... (15)

NOTE — The provision given in **5.5.2**(a) are the minimum design eccentricities for external and internal wall panel. The actual eccentricities due to gravity, superimposed seismic or wind load, have to be calculated for the actual conditions of loading.

b) *Stiffening along vertical edges* — A wall panel is said to be stiffened along a vertical edge if it is connected along that edge to another wall component or stiffener which can prevent any displacement of that edge perpendicular to the plane of the wall. For such fixing to be effective, the stiffening panel shall be at right angle to the considered panel and should have a width at least equal to quarter of its depth.

The influence of boundary conditions on the effective height of wall is expressed by the following formula:

$$l_{wd} = K_1 \, l_w \qquad \text{... (16)}$$

where

l_{wd} = effective height of wall

l_w = actual height of wall

K_1 = reduction factor which is computed as follows:

for panels unstiffened along vertical edges

$K_1 = 1{\cdot}0$

for panels stiffened along one vertical edge

when $l_w < b_w$, $K_1 = 1{\cdot}0$

when $b_w < l_w \leqslant 2b_w$

$$K_1 = 1{\cdot}4 - 0{\cdot}4 \frac{l_w}{b_w} \qquad \text{... (17)}$$

when $l_w > 2b_w$

$$K_1 = \frac{1}{\sqrt{1 + 0{\cdot}5 \left(\frac{l_w}{b_w} \right)^2}} \qquad \text{... (18)}$$

in which b_w = breadth of wall panel

for panels stiffened along both vertical edges

when $l_w < 0{\cdot}5 \, b_w$, $K_1 = 1{\cdot}0$

when $0{\cdot}5 \, b_w < l_w \leqslant b_w$

$$\therefore K_1 = 1{\cdot}5 - \frac{l_w}{b_w} \qquad \text{... (19)}$$

when $l_w > b_w$

$$K_1 = \frac{1}{1 + \left(\frac{l_w}{b_w} \right)^2} \qquad \dots (20)$$

c) *Load bearing capacity* — The load bearing capacity of Zone 1 in wall panels with longitudinal main reinforcement $\leqslant$ 0·40 percent shall be calculated according to the following formula:

1) Plain concrete walls

$$R = \frac{\sigma_{kc}}{\gamma_m} A_w \varphi \qquad \dots (21)$$

where

R = load-bearing capacity of wall

σ_{kc} = characteristic strength of concrete in wall

A_w = horizontal cross-sectional area of wall

φ = reduction factor

γ_m = strength reduction factor. In case of composite walls the equivalent characteristic strengths of the composite material based on tests is to be taken

When load is non-uniformly distributed (for example, when wind load is taken into account), it can be assumed that adequate uniformly distributed load acts along the width b_w. The value of b_w calculated from the edge of panel or from the edge of strips separated by openings, shall be assumed as follows:

$b_w = l_w$, when the panel is stiffened along the vertical edge

b_w = 1·0 m, when the panel is unstiffened along the vertical edge

In both cases, b_w shall be less than the width of strips between openings. The values of coefficient for panels with rectangular cross-sections are given in Table 1.

2) *Reinforced concrete wall* — The load-bearing capacity of reinforced concrete walls shall be calculated in accordance with IS : 456-1978* taking into account the appropriate strength reduction factors.

5.5.3 *Zone* 2 *of the Wall* — Load-bearing capacity of Zone 2 of the wall shall be calculated according to the following formula neglecting the

*Code of practice for plain and reinforced concrete (*third revision*).

TABLE 1 REDUCTION COEFFICIENT

[*Clause* 5.5.2 (c)]

$\frac{l_{wd}}{t_w}$	e_o/t_w		
	0·10	0·15	0·20
8	0·69	0·59	0·50
10	0·57	0·67	0·48
12	0·65	0·55	0·46
14	0·63	0·53	0·43
16	0·60	0·50	0·39
18	0·57	0·47	0·34
20	0·54	0·42	0·31
22	0·50	0·38	0·28
24	0·46	0·35	0·26

influence of transverse reinforcement

$$R = \frac{\sigma_{kc}}{\gamma_m} A_w \psi_1 \psi_2 \qquad \dots (22)$$

where

R = load-bearing capacity of the wall

σ_{kc} = characteristic strength of concrete

γ_m = strength reduction factor

A_w = horizontal cross-sectional area of the wall considered

ψ_1 = coefficient depending on the type of joint

ψ_2 = coefficient depending on the reinforcement

$= 1 + 0·7 \sqrt{\frac{n\,d}{s}}$

where

n = number of meshes in the zone of the joint (subject to a minimum of 3)

d = dia of transverse reinforcement

s = distance between transverse bars

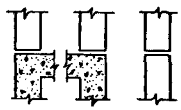
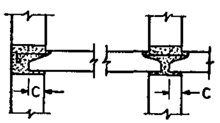
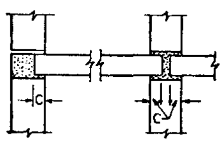
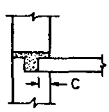
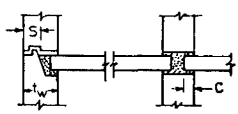
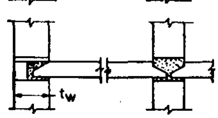
If in a composite wall, the thickness of top concrete rib is not less than 1/3 of the distance between vertical ribs then values of l as given for normal concrete in Table 2 can be adopted.

In the case of non-linear distribution of vertical loads on the wall panel caused due to wind load, it can be assumed that adequate uniformly distributed load acts on the length of the joint b_i. In that case the value of b_j shall not be greater than 2·0 m or the distance between the rows of openings.

The transverse reinforcement is normally placed at the top of the wall panel in the form of closely spaced meshes arranged in at least three horizontal layers.

TABLE 2 VALUES OF COEFFICIENT ψ_1

(*Clause* 5.5.3)

Kind of Joint	$l_{ef} < 4.8$ m External Wall	$l_{ef} < 4.8$ m Internal Wall	$4.8 < l_{ef} \leqslant 6.0$ m External Wall	$4.8 < l_{ef} \leqslant 6.0$ m Internal Wall	Remarks
	0·80	0·80	0·80	0·80	Wall panels resting on homogeneous cast *in-situ* structures
	0·75	0·75	0·70	0·75	Walls of normal concrete
	0·50	0·60	0·45	0·50	—
	0·60	—	0·50	—	Walls of normal concrete $c \ngtr 4$ cm
	0·40	0·55	0·35	0·50	Walls of light-weight concrete and composite walls
	0·45	0·65	0·40	0·65	$S \leqslant 0.25\, t_w$

For wall panels produced in battery moulds, when concrete is compacted by means of external vibrators the value of R in Eq. 22 shall be reduced by 10 percent.

In case when the horizontal joint is designed otherwise than what is given in Table 2, the load-bearing capacity of the wall in the zone of the joint shall be determined empirically.

NOTE — Calculation of the strength of the wall panels in Zone 2 is based on the results of empirical formulae derived from test results. The analysis is more complex since the behaviour of the zone is predominantly dependent on the characteristics of the joint, its shape and the strength of mortar used in the joint.

5.5.4 *Lintels* — Load-bearing capacity of lintels shall be calculated for the shear and bending moment taking into account the influence of both vertical and horizontal loads. Load-bearing capacity shall be computed according to simple bending theory applicable to reinforced concrete members taking into account the appropriate strength reduction factors.

5.5.5 *Vertical Joints* — Shear resistance of vertical joints shall be checked by the following formulae:

a) Keyed joints

1) Unreinforced keyed joint with a minimum steel of 250 mm² in the tie-beam.

$$R_j = 0{\cdot}1 \frac{\sigma_{kc}}{\gamma_m} (A_k + A_{ct}) \qquad \dots (23)$$

where

R_j = ultimate shear capacity of the joint

σ_{kc} = characteristic strength of concrete

γ_m = strength reduction factor

A_k, A_{ct} = total area of cross-section of shear-keys in the joint and area of cross-section of tie-beam respectively.

2) Shear strength of reinforced keyed joint

$$R_j = 0{\cdot}12 \frac{\sigma_{kc}}{\gamma_m} (A_k + A_{ct}) + 2\,000 \, \triangle A_s \qquad \dots (24)$$

where

R_j, σ_{kc}, γ_m, A_k, A_{ct} are as defined in Eq. 23.

A_s = amount of additional reinforcement over 2·5 cm²

$= (A_s - 2{\cdot}5)$

A_s = steel area provided (the maximum value for this calculation is limited to $9 \cdot 5\ \text{cm}^2$)

b) Grooved joints

$$R_j = 0{\cdot}02 \frac{\sigma_{kc}}{\gamma_m} (A_j + A_{ct}). \qquad \ldots (25)$$

where R_j, σ_{kc}, γ_m, A_{ct} are as given in Eq. 23.

A_j = area of cross-section through the joint, while calculating A, the portion of a joint of width less than 30 mm shall be neglected.

5.5.6 *Horizontal Joints* — In case of joints without keys the shear resistance of horizontal joints shall be checked by the following formula if the entire wall is in compression

$$R_j = 2{\cdot}0\ A_w \qquad \ldots (26)$$

If higher resistance is needed, the horizontal joint is to be provided with keys. The shear resistance of keyed joint shall be checked by the following formulae:

When compressive forces are produced in the key due to the action of lateral forces

$$R_{hl} = R_{cl} + 2{\cdot}5 \frac{N_c}{\sqrt{\sigma_{kc}/\gamma_m}} \leqslant 1{\cdot}4\ R_{cl} \qquad \ldots (27)$$

When tensile forces are produced in the key due to the action of lateral forces

$$R_{hl} + R_{tl} \left[1 - \left(\frac{N_t}{\frac{A_s\ \sigma_{ks}}{\gamma_w}} \right) \right] \qquad \ldots (28)$$

$$\text{in which } R_{tl} = 0{\cdot}06 \frac{(\sigma_{kc})}{\gamma_w} \frac{(\sigma_{ks})^n d}{\gamma_w} \qquad \ldots (29)$$

where

R_{hl} = strength of an individual key in horizontal joint

R_{cl} = strength of the key when acted upon by shear forces parallel to the length of the joint as given by Eq. 30

N_c, N_t = normal compression and tensile forces in the key respectively due to the lateral forces

$\frac{\sigma_{ks}}{\gamma_j}$ = characteristic strength of concrete reduced by the corresponding strength reduction factor in joint or key

$\frac{\sigma_{ks}}{\gamma_w}$ = characteristic strength of transverse reinforcement reduced by the corresponding strength reduction factor in the key

n, d = number and diameter of transverse bars in the key

The value of R_{cl} in Eq. 27 is derived from Eq. 24 which is given as below:

$$R_{cl} = 0{\cdot}12 \frac{\sigma_{kc}}{\gamma_w} A_k' + 2\,000\, A_s' \qquad \text{.. (30)}$$

where

A_k' = area of the key under consideration

A_s' = area of the transverse reinforcement in the key

5.6 Accidental Forces

5.6.1 In addition to designing the structure to support loads arising from normal use, there should be a reasonable probability that it will not collapse under the effect of misuse or accident. No structure can be expected to be resistant to the excessive loads or forces that could arise due to an extreme cause, but it should not be damaged to an extent disproportionate to the original cause. Assessment of residual stability and spread of damage following the removal of a load-bearing element or alternatively, provision within the structure of tensile ties is recommended.

5.6.2 Partial safety factors γ_l for accidental loads

Design dead load = $0{\cdot}95$ (F_A or $1{\cdot}05\ F_d$)

Design imposed load = $0{\cdot}35\ F_i$

= $1{\cdot}05\ F_i$ in case of buildings and predominantly for storage or where the imposed load is of permanent nature

Design wind load = $0{\cdot}35\ F_w$

where

F_d = characteristic dead load

F_i = characteristic imposed load

F_w = characteristic wind load

5.6.3 *Partial Safety Factor γ_m for Accidental Loads* — While considering the probable effects of misuse or accident, the value of γ_m recommended in IS : 456-1978* may be halved except where otherwise required.

5.6.4 *Location of Tie-Beams* — The location of transverse tie-beams for large panel buildings is as shown in Fig. 13. The amount of reinforcement shall be in accordance with **4.7** or **4.9.5**.

*Code of practice for plain and reinforced concrete (*third revision*).

(*Continued from page* 2)

Members	*Representing*
SHRI A. BHARADWAJ	In personal capacity (*207 Golf Links, New Delhi*)
SHRI J. A. D'COSTA	Indian Institute of Architects, Bombay
SHRI ARUN V. OGALE (*Alternate*)	
SHRI R. A. DESHMUKH	United Precast Products, Pune
SHRI A. D. GUPTE (*Alternate*)	
SHRI P. M. DESHPANDE	City and Industrial Development Corporation of Maharashtra Ltd, Bombay
SHRI ZACHARIA GEORGE	Structural Engineering Research Centre (CSIR), Madras
DR A. G. MADHAVA RAO (*Alternate*)	
SHRI N. K. GUPTA	Engineers India Limited, New Delhi
SHRI B. K. JINDAL	Central Building Research Institute (CSIR), Roorkee
SHRI N. N. BHISE (*Alternate*)	
SHRI H. K. JULKA	Cement Research Institute of India, New Delhi
SHRI MAHENDRA RAJ	Engineering Consultants (India), New Delhi
SHRI B. B. CHAUDHARY (*Alternate*)	
SHRI B. G. SHIRKE	B. G. Shirke and Co, Pune
SHRI R. T. PAWAR (*Alternate*)	
SHRI L. R. SASTRI	Tamil Nadu Housing Board, Madras
SHRI S. KUMARESAN (*Alternate*)	
SHRI K. S. SRINIVASAN	National Buildings Organization, New Delhi
SHRI SUNIL BERY (*Alternate*)	
SHRI S. SUBRAMANIAN	Larsen & Toubro Limited (ECC Construction Group), Madras
SHRI A. C. DATTAR (*Alternate*)	
SUPERINTENDING SURVEYOR OF WORKS (NDZ)	Central Public Works Department, New Delhi
SURVEYOR OF WORKS V (NDZ) (*Alternate*)	
SHRI K. VEERARAGHAVACHARY	Bharat Heavey Electricals Ltd, Hyderabad
SHRI V. MALVIYA (*Alternate*)	
REPRESENTATIVE	Hindustan Prefab Ltd, New Delhi
SHRI M. KUNDU (*Alternate*)	

Panel for Connection in Prefabricated Construction and Large Prefabricates, BDC 32 : P2

Convener	
SHRI A. RAMAKRISHNA	Larsen & Toubro Limited (ECC Construction Group), Madras
Members	
SHRI S. SUBRAMANIAN (*Alternate* to Shri A. Ramakrishna)	
SHRI D. I. DESAI	Gammon India Limited, Bombay
SHRI R. A. DESHMUKH	United Precast Products, Pune

(*Continued on page* 40)

(*Continued from page* 39)

Members	*Representing*
SHRI P. M. DESHPANDE	City and Industrial Development Corporation Ltd (CIDCO), Bombay
SHRI ZACHARIA GEORGE	Structural Engineering Research Centre (CSIR), Madras
DR A. G. MADHAVA RAO (*Alternate*)	
SHRI R. C. MANGAL	Central Building Research Institute (CSIR), Roorkee
SHRI S. C. CHAKRABARTY (*Alternate*)	
SHRI SHIRISH B. PATEL	Shirish Patel & Associates, Bombay
SHRI P. H. SRINIVASACHAR (*Alternate*)	
SHRI R. T. PAWAR	B. G. Shirke and Co, Pune
SHRI K. VEERARAGHAVACHARY	Bharat Heavy Electricals Ltd, Hyderabad
REPRESENTATIVE	Hindustan Prefab Limited, Bombay
SHRI M. KUNDU (*Alternate*)	
REPRESENTATIVE	Ambike Consultants, New Bombay

AMENDMENT NO. 1 DECEMBER 2006
TO
IS 11447 : 1985 CODE OF PRACTICE FOR CONSTRUCTION WITH LARGE PANEL PREFABRICATES

(*Page* 3, *clause* **0.2**) — Insert the following para at the end:

"The various structural schemes are possible using large panel prefabricates. A reference to Part 6 Structural design, Section 7A Prefabricated concrete of SP 7 : 2005 'National Building Code of India 2005' may be made in this regard."

(*Page* 3, *clause* **0.4**) — Insert the following clause at the end:

'**0.5** All standards, whether given herein above or cross-referred to in the main text of this standard, are subject to revision. The parties to agreement based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards.'

(*Page* 7, *clause* **4.1.2**) — Insert the following at the end:

"The structural design of elements shall be required to take care the provisions specified for earthquake resistant design of buildings in IS 1893 (Part 1) : 2002 'Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)'."

(*Page* 9 *clause* **4.1.7**) — Insert the following at the end:

'**4.1.8** The panels shall be designed and fabricated considering the design and laying requirements of various services, such as electrical, public health, mechanical and HVAC for the building.'

(*Page* 13, *clause* **4.5.1**) — Substitute the following for the existing clause:

'**4.5.1** The walls may be designed as unreinforced or reinforced cement concrete. It is recommended to design walls as plain concrete with secondary reinforcement for production and erection purpose only which is not taken into account while checking the safety of wall. For walls, concrete of grade not less than M 20 shall be used.'

4.5.1.1 *External walls*

The external walls, their horizontal and vertical joints/junctions and junctions at doors/windows shall satisfy the following requirements:

a) They should provide resistance to loads either vertical or horizontal, if required; this depends on the layout of the building;
b) The fire resistance, thermal comfort and natural lighting shall be in accordance with the requirements of relevant Indian Standards; and
c) They should provide adequate protection against weather to the building and its occupants.

External walls may be classified as homogenous walls and non-homogenous walls. They may be load bearing as well as non-load bearing walls. The thickness of the walls shall depend on the minimum structural needs as well as the thickness needed at joints to ensure water tightness. Non-homogenous walls may be sandwich panel type, composed of two leaves separated by a layer of insulation, or they may be framed infill type. In sandwich panels, the external and internal walls are both of concrete, appropriately interconnected through the central layer of insulation. All walls shall be designed and detailed to adequately resist the effects of temperature and shrinkage.

In infill walls, where there is a perimeter frame enclosing masonry infills, the frame shall be structurally adequate to carry all imposed loads. Intermediate ribs may be introduced to improve the structural action

For sandwich panels, the minimum thickness of external concrete layer shall be 40 mm when its movement is constrained. When it has free movement, its minimum thickness shall be 60 mm and it should be connected with internal layer by means of suitable clips allowing free movement.

Different shapes of panels are possible for external walls. Windows may be cast into the panel itself, or they may be fixed at site.

4.5.1.2 *Internal walls*

Internal structural walls may be homogenous walls of cement concrete (reinforced or unreinforced) or light weight concrete. In case steel is required across panel joints, the joint should be designed to provide continuity in bars.'

(*Page* 13, *clause* **4.5.2**) — Insert the following at the end:

'However the thickness of walls shall meet the requirements for earthquake resistance in accordance with IS 1893 (Part 1) : 2002.'

(*Page* 24, *clause* **4.9.5**) — Insert the following at the end:

'**4.9.6** While using large panel prefabricates, the structural integrity of the whole structure has to be ensured. For various applications, for ensuring diaphragm action, the provisions specified in IS 1893 (Part 1) : 2002 and IS 4326 : 1993 'Code of practice for earthquake resistant design and construction of buildings (*second revision*)' shall be followed.'

(CED 51)

Reprography Unit, BIS, New Delhi, India