Reference- Indian Standards

IS 1893 (Part 1) : 2002

IS 4326 : 2013

IS 13920 : 1993

IS 13935 : 2009

IS 15988 : 2013

(Reaffirmed 2007)

भारतीय मानक

संरचनाओं के भूकम्परोधी डिजाइन के मानदंड

भाग 1 सामान्य प्रावधान और भवन

(पाँचवाँ पुनरीक्षण)

Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 1 GENERAL PROVISIONS AND BUILDINGS

(Fifth Revision)

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BUREAU OF INDIAN STANDARDS
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Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 1 GENERAL PROVISIONS AND BUILDINGS

(*Fifth Revision*)

FOREWORD

This Indian Standard (Part 1) (Fifth Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

Himalayan-Nagalushai region, Indo-Gangetic Plain, Western India, Kutch and Kathiawar regions are geologically unstable parts of the country, and some devastating earthquakes of the world have occurred there. A major part of the peninsular India has also been visited by strong earthquakes, but these were relatively few in number occurring at much larger time intervals at any site, and had considerably lesser intensity. The earthquake resistant design of structures taking into account seismic data from studies of these Indian earthquakes has become very essential, particularly in view of the intense construction activity all over the country. It is to serve this purpose that IS 1893 : 1962 'Recommendations for earthquake resistant design of structures' was published and revised first time in 1966.

As a result of additional seismic data collected in India and further knowledge and experience gained since the publication of the first revision of this standard, the sectional committee felt the need to revise the standard again incorporating many changes, such as revision of maps showing seismic zones and epicentres, and adding a more rational approach for design of buildings and sub-structures of bridges. These were covered in the second revision of IS 1893 brought out in 1970.

As a result of the increased use of the standard, considerable amount of suggestions were received for modifying some of the provisions of the standard and, therefore, third revision of the standard was brought out in 1975. The following changes were incorporated in the third revision:

a) The standard incorporated seismic zone factors (previously given as multiplying factors in the second revision) on a more rational basis.

b) Importance factors were introduced to account for the varying degrees of importance for various structures.

c) In the clauses for design of multi-storeyed buildings, the coefficient of flexibility was given in the form of a curve with respect to period of buildings.

d) A more rational formula was used to combine modal shear forces.

e) New clauses were introduced for determination of hydrodynamic pressures in elevated tanks.

f) Clauses on concrete and masonry dams were modified, taking into account their dynamic behaviour during earthquakes. Simplified formulae for design forces were introduced based on results of extensive studies carried out since second revision of the standard was published.

The fourth revision, brought out in 1984, was prepared to modify some of the provisions of the standard as a result of experience gained with the use of the standard. In this revision, a number of important basic modifications with respect to load factors, field values of N, base shear and modal analysis were introduced. A new concept of performance factor depending on the structural framing system and on the ductility of construction was incorporated. Figure 2 for average acceleration spectra was also modified and a curve for zero percent damping incorporated.

In the fifth revision, with a view to keep abreast with the rapid development and extensive research that has been carried out in the field of earthquake resistant design of various structures, the committee has decided to cover the provisions for different types of structures in separate parts. Hence, IS 1893 has been split into the following five parts:

Part 1 General provisions and buildings

Part 2 Liquid retaining tanks — Elevated and ground supported

Part 3 Bridges and retaining walls

Part 4 Industrial structures including stack like structures

Part 5 Dams and embankments

Part 1 contains provisions that are general in nature and applicable to all structures. Also, it contains provisions that are specific to buildings only. Unless stated otherwise, the provisions in Parts 2 to 5 shall be read necessarily in conjunction with the general provisions in Part 1.

NOTE — Pending finalization of Parts 2 to 5 of IS 1893, provisions of Part 1 will be read along with the relevant clauses of IS 1893 : 1984 for structures other than buildings.

The following are the major and important modifications made in the fifth revision:

a) The seismic zone map is revised with only four zones, instead of five. Erstwhile Zone I has been merged to Zone II. Hence, Zone I does not appear in the new zoning; only Zones II, III, IV and V do.

b) The values of seismic zone factors have been changed; these now reflect more realistic values of effective peak ground acceleration considering Maximum Considered Earthquake (MCE) and service life of structure in each seismic zone.

c) Response spectra are now specified for three types of founding strata, namely rock and hard soil, medium soil and soft soil.

d) Empirical expression for estimating the fundamental natural period T_a of multi-storeyed buildings with regular moment resisting frames has been revised.

e) This revision adopts the procedure of first calculating the actual force that may be experienced by the structure during the probable maximum earthquake, if it were to remain elastic. Then, the concept of response reduction due to ductile deformation or frictional energy dissipation in the cracks is brought into the code explicitly, by introducing the 'response reduction factor' in place of the earlier performance factor.

f) A lower bound is specified for the design base shear of buildings, based on empirical estimate of the fundamental natural period T_a.

g) The soil-foundation system factor is dropped. Instead, a clause is introduced to restrict the use of foundations vulnerable to differential settlements in severe seismic zones.

h) Torsional eccentricity values have been revised upwards in view of serious damages observed in buildings with irregular plans.

j) Modal combination rule in dynamic analysis of buildings has been revised.

k) Other clauses have been redrafted where necessary for more effective implementation.

It is not intended in this standard to lay down regulation so that no structure shall suffer any damage during earthquake of all magnitudes. It has been endeavoured to ensure that, as far as possible, structures are able to respond, without structural damage to shocks of moderate intensities and without total collapse to shocks of heavy intensities. While this standard is intended for the earthquake resistant design of normal structures, it has to be emphasized that in the case of special structures, such as large and tall dams, long-span bridges, major industrial projects, etc, site-specific detailed investigation should be undertaken, unless otherwise specified in the relevant clauses.

Though the basis for the design of different types of structures is covered in this standard, it is not implied that detailed dynamic analysis should be made in every case. In highly seismic areas, construction of a type which entails heavy debris and consequent loss of life and property, such as masonry, particularly mud masonry and rubble masonry, should preferably be avoided. For guidance on precautions to be observed in the construction of buildings, reference may be made to IS 4326, IS 13827 and IS 13828.

Earthquake can cause damage not only on account of the shaking which results from them but also due to other chain effects like landslides, floods, fires and disruption to communication. It is, therefore, important to take necessary precautions in the siting, planning and design of structures so that they are safe against such secondary effects also.

The Sectional Committee has appreciated that there cannot be an entirely scientific basis for zoning in view of the scanty data available. Though the magnitudes of different earthquakes which have occurred in the past are known to a reasonable degree of accuracy, the intensities of the shocks caused by these earthquakes have so far been mostly estimated by damage surveys and there is little instrumental evidence to corroborate the conclusions arrived at. Maximum intensity at different places can be fixed on a scale only on the basis of the observations made and recorded after the earthquake and thus a zoning map which is based on the maximum intensities arrived at, is likely to lead in some cases to an incorrect conclusion in view of (a) incorrectness in the assessment of intensities, (b) human error in judgment during the damage survey, and (c) variation in quality and design of structures causing variation in type and extent of damage to the structures for the same intensity of shock. The Sectional Committee has therefore, considered that a rational approach to the problem would be to arrive at a zoning map based on known magnitudes and the known epicentres (*see* Annex A) assuming all other conditions as being average and to modify such an idealized isoseismal map in light of tectonics (*see* Annex B), lithology (*see* Annex C) and the maximum intensities as recorded from damage surveys. The Committee has also reviewed such a map in the light of the past history and future possibilities and also attempted to draw the lines demarcating the different zones so as to be clear of important towns, cities and industrial areas, after making special examination of such cases, as a little modification in the zonal demarcations may mean considerable difference to the economics of a project in that area. Maps shown in Fig. 1 and Annexes A, B and C are prepared based on information available upto 1993.

In the seismic zoning map, Zone I and II of the contemporary map have been merged and assigned the level of Zone II. The Killari area has been included in Zone III and necessary modifications made, keeping in view the probabilistic hazard evaluation. The Bellary isolated zone has been removed. The parts of eastern coast areas have shown similar hazard to that of the Killari area, the level of Zone II has been enhanced to Zone III and connected with Zone III of Godawari Graben area.

The seismic hazard level with respect to ZPA at 50 percent risk level and 100 years service life goes on progressively increasing from southern peninsular portion to the Himalayan main seismic source, the revised seismic zoning map has given status of Zone III to Narmada Tectonic Domain, Mahanandi Graben and Godawari Graben. This is a logical normalization keeping in view the apprehended higher strain rates in these domains on geological consideration of higher neotectonic activity recorded in these areas.

Attention is particularly drawn to the fact that the intensity of shock due to an earthquake could vary locally at any place due to variation in soil conditions. Earthquake response of systems would be affected by different types of foundation system in addition to variation of ground motion due to various types of soils. Considering the effects in a gross manner, the standard gives guidelines for arriving at design seismic coefficients based on stiffness of base soil.

It is important to note that the seismic coefficient, used in the design of any structure, is dependent on many variable factors and it is an extremely difficult task to determine the exact seismic coefficient in each given case. It is, therefore, necessary to indicate broadly the seismic coefficients that could generally be adopted in different parts or zones of the country though, of course, a rigorous analysis considering all the factors involved has to be made in the case of all important projects in order to arrive at a suitable seismic coefficients for design. The Sectional Committee responsible for the formulation of this standard has attempted to include a seismic zoning map (*see* Fig. 1) for this purpose. The object of this map is to classify the area of the country into a number of zones in which one may reasonably expect earthquake shaking of more or less same maximum intensity in future. The Intensity as per Comprehensive Intensity Scale (MSK64) (*see* Annex D) broadly associated with the various zones is VI (or less), VII, VIII and IX (and above) for Zones II, III, IV and V respectively. The maximum seismic ground acceleration in each zone cannot be presently predicted with accuracy either on a deterministic or on a probabilistic basis. The basic zone factors included herein are reasonable estimates of effective peak ground accelerations for the design of various structures covered in this standard. Zone factors for some important towns are given in Annex E.

Base isolation and energy absorbing devices may be used for earthquake resistant design. Only standard devices having detailed experimental data on the performance should be used. The designer must demonstrate by detailed analyses that these devices provide sufficient protection to the buildings and equipment as envisaged in this standard. Performance of locally assembled isolation and energy absorbing devices should be evaluated experimentally before they are used in practice. Design of buildings and equipment using such device should be reviewed by the competent authority.

Base isolation systems are found useful for short period structures, say less than 0.7 s including soil-structure interaction.

In the formulation of this standard, due weightage has been given to international coordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country. Assistance has particularly been derived from the following publications:

a) UBC 1994, Uniform Building Code, International Conference of Building Officials, Whittier, California, U.S.A. 1994.

b) NEHRP 1991, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, Part 1 : Provisions, Report No. FEMA 222, Federal Emergency Management Agency, Washington, D.C., U.S.A., January 1992.

c) NEHRP 1991, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, Part 2 : Commentary, Report No. FEMA 223, Federal Emergency Management Agency, Washington, D.C., U.S.A., January 1992.

d) NZS 4203 : 1992, Code of Practice for General Structural Design and Design Loadings for Buildings, Standards Association of New Zealand, Wellington, New Zealand, 1992.

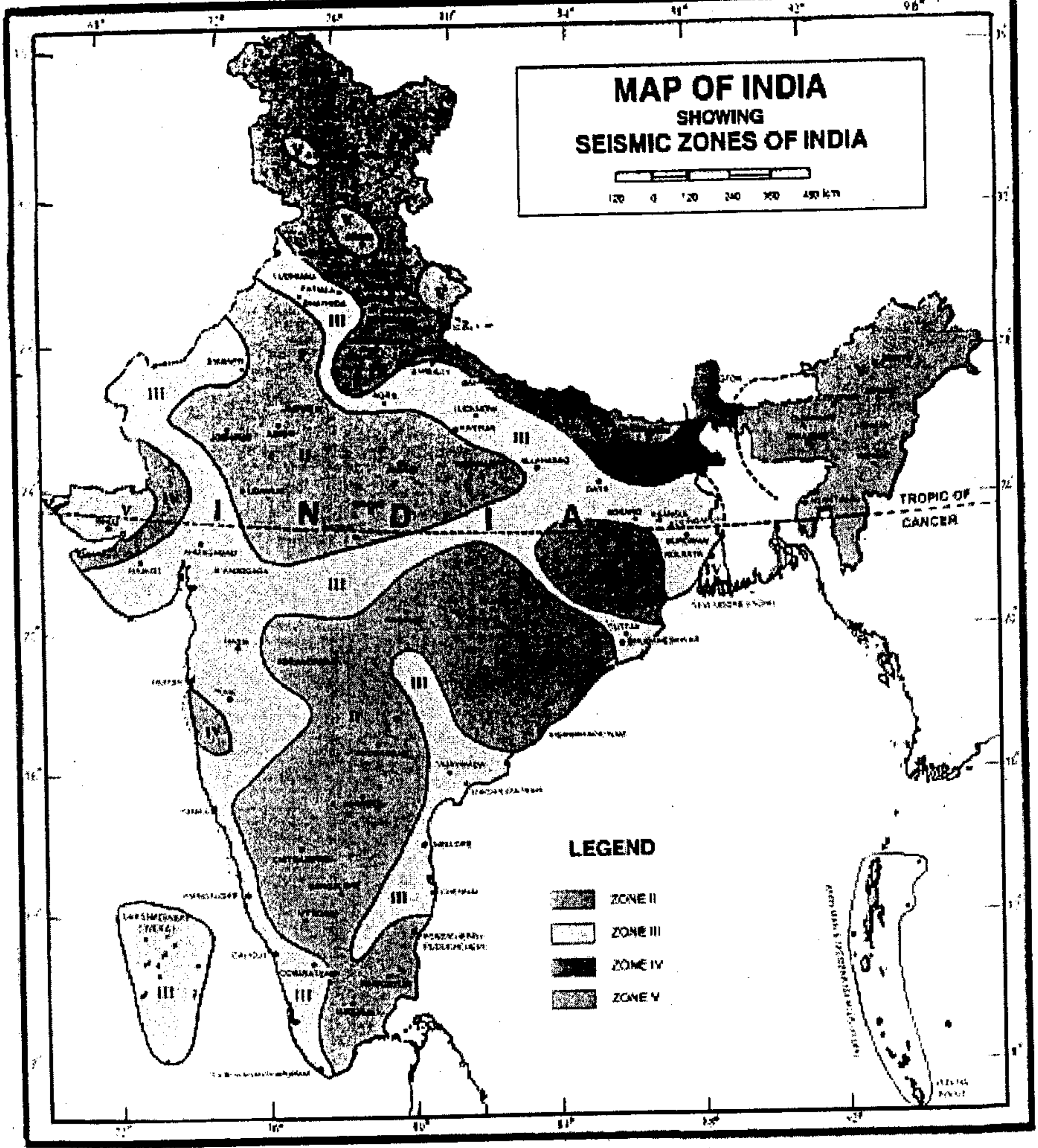
In the preparation of this standard considerable assistance has been given by the Department of Earthquake Engineering, University of Roorkee; Indian Institute of Technology, Kanpur; IIT Bombay, Mumbai; Geological Survey of India; India Meteorological Department, and several other organizations.

The units used with the items covered by the symbols shall be consistent throughout this standard, unless specifically noted otherwise.

The composition of the Committee responsible for the formulation of this standard is given in Annex F.

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 '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.

(Earthquake Engineering Sectional Committee, CED 39)



NOTE : Towns falling at the boundary of zones demarcation line between two zones shall be considered in High Zone.

- Based upon Survey of India map with the permission of the Surveyor General of India.
- The responsibility for the correctness of internal details rests with the publisher.
- The territorial waters of India extend into the sea to distance of twelve nautical miles measured from the appropriate base line.
- The administrative headquarters of Chandigarh, Haryana and Punjab are at Chandigarh.
- The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.
- The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.

FIG. 1

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Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 1 GENERAL PROVISIONS AND BUILDINGS

(Fifth Revision)

1 SCOPE

1.1 This standard (Part 1) deals with assessment of seismic loads on various structures and earthquake resistant design of buildings. Its basic provisions are applicable to buildings; elevated structures; industrial and stack like structures; bridges; concrete masonry and earth dams; embankments and retaining walls and other structures.

1.2 Temporary elements such as scaffolding, temporary excavations need not be designed for earthquake forces.

1.3 This standard does not deal with the construction features relating to earthquake resistant design in buildings and other structures. For guidance on earthquake resistant construction of buildings, reference may be made to the following Indian Standards:

IS 4326, IS 13827, IS 13828, IS 13920 and IS 13935.

2 REFERENCES

2.1 The following Indian Standards are necessary adjuncts to this standard:

IS No.	*Title*
456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
800 : 1984	Code of practice for general construction in steel (*second revision*)
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 (*second revision*)
(Part 2) : 1987	Imposed loads (*second revision*)
(Part 3) : 1987	Wind loads (*second revision*)
(Part 4) : 1987	Snow loads (*second revision*)
(Part 5) : 1987	Special loads and load combinations (*second revision*)
1343 : 1980	Code of practice for pre-stressed concrete (*first revision*)
1498 : 1970	Classification and identification of soils for general engineering purposes (*first revision*)
1888 : 1982	Method of load test on soils (*second revision*)
1893 (Part 4)	Criteria for earthquake resistant design of structures: Part 4 Industrial structures including stack like structures
2131 : 1981	Method of standard penetration test for soils (*first revision*)
2809 : 1972	Glossary of terms and symbols relating to soil engineering (*first revision*)
2810 : 1979	Glossary of terms relating to soil dynamics (*first revision*)
4326 : 1993	Earthquake resistant design and construction of buildings — Code of practice (*second revision*)
6403 : 1981	Code of practice for determination of bearing capacity of shallow foundations (*first revision*)
13827 : 1993	Improving earthquake resistance of earthen buildings — Guidelines
13828 : 1993	Improving earthquake resistance of low strength masonry buildings — Guidelines
13920 : 1993	Ductile detailing of reinforced concrete structures subjected to seismic forces — Code of practice
13935 : 1993	Repair and seismic strengthening of buildings — Guidelines
SP 6 (6) : 1972	Handbook for structural engineers: Application of plastic theory in design of steel structures

3 TERMINOLOGY FOR EARTHQUAKE ENGINEERING

3.1 For the purpose of this standard, the following definitions shall apply which are applicable generally to all structures.

NOTE — For the definitions of terms pertaining to soil mechanics and soil dynamics references may be made to IS 2809 and IS 2810.

3.2 Closely-Spaced Modes

Closely-spaced modes of a structure are those of its natural modes of vibration whose natural frequencies differ from each other by 10 percent or less of the lower frequency.

3.3 Critical Damping

The damping beyond which the free vibration motion will not be oscillatory.

3.4 Damping

The effect of internal friction, imperfect elasticity of material, slipping, sliding, etc in reducing the amplitude of vibration and is expressed as a percentage of critical damping.

3.5 Design Acceleration Spectrum

Design acceleration spectrum refers to an average smoothened plot of maximum acceleration as a function of frequency or time period of vibration for a specified damping ratio for earthquake excitations at the base of a single degree of freedom system.

3.6 Design Basis Earthquake (DBE)

It is the earthquake which can reasonably be expected to occur at least once during the design life of the structure.

3.7 Design Horizontal Acceleration Coefficient (A_h)

It is a horizontal acceleration coefficient that shall be used for design of structures.

3.8 Design Lateral Force

It is the horizontal seismic force prescribed by this standard, that shall be used to design a structure.

3.9 Ductility

Ductility of a structure, or its members, is the capacity to undergo large inelastic deformations without significant loss of strength or stiffness.

3.10 Epicentre

The geographical point on the surface of earth vertically above the focus of the earthquake.

3.11 Effective Peak Ground Acceleration (EPGA)

It is 0.4 times the 5 percent damped average spectral acceleration between period 0.1 to 0.3 s. This shall be taken as Zero Period Acceleration (ZPA).

3.12 Floor Response Spectra

Floor response spectra is the response spectra for a time history motion of a floor. This floor motion time history is obtained by an analysis of multi-storey building for appropriate material damping values subjected to a specified earthquake motion at the base of structure.

3.13 Focus

The originating earthquake source of the elastic waves inside the earth which cause shaking of ground due to earthquake.

3.14 Importance Factor (I)

It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterised by hazardous consequences of its failure, its post-earthquake functional need, historic value, or economic importance.

3.15 Intensity of Earthquake

The intensity of an earthquake at a place is a measure of the strength of shaking during the earthquake, and is indicated by a number according to the modified Mercalli Scale or M.S.K. Scale of seismic intensities (*see* Annex D).

3.16 Liquefaction

Liquefaction is a state in saturated cohesionless soil wherein the effective shear strength is reduced to negligible value for all engineering purpose due to pore pressure caused by vibrations during an earthquake when they approach the total confining pressure. In this condition the soil tends to behave like a fluid mass.

3.17 Lithological Features

The nature of the geological formation of the earths crust above bed rock on the basis of such characteristics as colour, structure, mineralogical composition and grain size.

3.18 Magnitude of Earthquake (Richter's Magnitude)

The magnitude of earthquake is a number, which is a measure of energy released in an earthquake. It is defined as logarithm to the base 10 of the maximum trace amplitude, expressed in microns, which the standard short-period torsion seismometer (with a period of 0.8 s, magnification 2 800 and damping nearly critical) would register due to the earthquake at an epicentral distance of 100 km.

3.19 Maximum Considered Earthquake (MCE)

The most severe earthquake effects considered by this standard.

3.20 Modal Mass (M_k)

Modal mass of a structure subjected to horizontal or vertical, as the case may be, ground motion is a part of the total seismic mass of the structure that is effective in mode *k* of vibration. The modal mass for a given mode has a unique value irrespective of scaling of the mode shape.

3.21 Modal Participation Factor (P_k)

Modal participation factor of mode *k* of vibration is the amount by which mode *k* contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions. Since the amplitudes of 95 percent mode shapes can be scaled arbitrarily, the value of this factor depends on the scaling used for mode shapes.

3.22 Modes of Vibration (*see* Normal Mode)

3.23 Mode Shape Coefficient (ϕ_{ik})

When a system is vibrating in normal mode *k*, at any particular instant of time, the amplitude of mass *i* expressed as a ratio of the amplitude of one of the masses of the system, is known as mode shape coefficient (ϕ_{ik}).

3.24 Natural Period (*T*)

Natural period of a structure is its time period of undamped free vibration.

3.24.1 *Fundamental Natural Period* (T_1)

It is the first (longest) modal time period of vibration.

3.24.2 *Modal Natural Period* (T_k)

The modal natural period of mode *k* is the time period of vibration in mode *k*.

3.25 Normal Mode

A system is said to be vibrating in a normal mode when all its masses attain maximum values of displacements and rotations simultaneously, and pass through equilibrium positions simultaneously.

3.26 Response Reduction Factor (*R*)

It is the factor by which the actual base shear force, that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall be reduced to obtain the design lateral force.

3.27 Response Spectrum

The representation of the maximum response of idealized single degree freedom systems having certain period and damping, during earthquake ground motion. The maximum response is plotted against the undamped natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

3.28 Seismic Mass

It is the seismic weight divided by acceleration due to gravity.

3.29 Seismic Weight (*W*)

It is the total dead load plus appropriate amounts of specified imposed load.

3.30 Structural Response Factors (S_a/g)

It is a factor denoting the acceleration response spectrum of the structure subjected to earthquake ground vibrations, and depends on natural period of vibration and damping of the structure.

3.31 Tectonic Features

The nature of geological formation of the bed rock in the earth's crust revealing regions characterized by structural features, such as dislocation, distortion, faults, folding, thrusts, volcanoes with their age of formation, which are directly involved in the earth movement or quake resulting in the above consequences.

3.32 Time History Analysis

It is an analysis of the dynamic response of the structure at each increment of time, when its base is subjected to a specific ground motion time history.

3.33 Zone Factor (*Z*)

It is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake (MCE) in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimate of effective peak ground acceleration.

3.34 Zero Period Acceleration (ZPA)

It is the value of acceleration response spectrum for period below 0.03 s (frequencies above 33 Hz).

4 TERMINOLOGY FOR EARTHQUAKE ENGINEERING OF BUILDINGS

4.1 For the purpose of earthquake resistant design of buildings in this standard, the following definitions shall apply.

4.2 Base

It is the level at which inertia forces generated in the structure are transferred to the foundation, which then transfers these forces to the ground.

4.3 Base Dimensions (d)

Base dimension of the building along a direction is the dimension at its base, in metre, along that direction.

4.4 Centre of Mass

The point through which the resultant of the masses of a system acts. This point corresponds to the centre of gravity of masses of system.

4.5 Centre of Stiffness

The point through which the resultant of the restoring forces of a system acts.

4.6 Design Eccentricity (e_{di})

It is the value of eccentricity to be used at floor i in torsion calculations for design.

4.7 Design Seismic Base Shear (V_B)

It is the total design lateral force at the base of a structure.

4.8 Diaphragm

It is a horizontal, or nearly horizontal system, which transmits lateral forces to the vertical resisting elements, for example, reinforced concrete floors and horizontal bracing systems.

4.9 Dual System

Buildings with dual system consist of shear walls (or braced frames) and moment resisting frames such that:

a) The two systems are designed to resist the total design lateral force in proportion to their lateral stiffness considering the interaction of the dual system at all floor levels; and

b) The moment resisting frames are designed to independently resist at least 25 percent of the design base shear.

4.10 Height of Floor (h_i)

It is the difference in levels between the base of the building and that of floor i.

4.11 Height of Structure (h)

It is the difference in levels, in metres, between its base and its highest level.

4.12 Horizontal Bracing System

It is a horizontal truss system that serves the same function as a diaphragm.

4.13 Joint

It is the portion of the column that is common to other members, for example, beams, framing into it.

4.14 Lateral Force Resisting Element

It is part of the structural system assigned to resist lateral forces.

4.15 Moment-Resisting Frame

It is a frame in which members and joints are capable of resisting forces primarily by flexure.

4.15.1 *Ordinary Moment-Resisting Frame*

It is a moment-resisting frame not meeting special detailing requirements for ductile behaviour.

4.15.2 *Special Moment-Resisting Frame*

It is a moment-resisting frame specially detailed to provide ductile behaviour and comply with the requirements given in IS 4326 or IS 13920 or SP 6 (6).

4.16 Number of Storeys (n)

Number of storeys of a building is the number of levels above the base. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

4.17 Principal Axes

Principal axes of a building are generally two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented.

4.18 P-Δ Effect

It is the secondary effect on shears and moments of frame members due to action of the vertical loads, interacting with the lateral displacement of building resulting from seismic forces.

4.19 Shear Wall

It is a wall designed to resist lateral forces acting in its own plane.

4.20 Soft Storey

It is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.

4.21 Static Eccentricity (e_{si})

It is the distance between centre of mass and centre of rigidity of floor i.

4.22 Storey

It is the space between two adjacent floors.

4.23 Storey Drift

It is the displacement of one level relative to the other level above or below.

4.24 Storey Shear (V_i)

It is the sum of design lateral forces at all levels above the storey under consideration.

4.25 Weak Storey

It is one in which the storey lateral strength is less than 80 percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

5 SYMBOLS

The symbols and notations given below apply to the provisions of this standard:

A_h	Design horizontal seismic coefficient
A_k	Design horizontal acceleration spectrum value for mode k of vibration
b_i	i^{th} Floor plan dimension of the building perpendicular to the direction of force
c	Index for the closely-spaced modes
d	Base dimension of the building, in metres, in the direction in which the seismic force is considered.
DL	Response quantity due to dead load
e_{di}	Design eccentricity to be used at floor i calculated as per **7.8.2**
e_{si}	Static eccentricity at floor i defined as the distance between centre of mass and centre of rigidity
EL_x	Response quantity due to earthquake load for horizontal shaking along x-direction
EL_y	Response quantity due to earthquake load for horizontal shaking along y-direction
EL_z	Response quantity due to earthquake load for vertical shaking along z-direction
F_{roof}	Design lateral forces at the roof due to all modes considered
F_i	Design lateral forces at the floor i due to all modes considered
g	Acceleration due to gravity
h	Height of structure, in metres
h_i	Height measured from the base of the building to floor i
I	Importance factor
IL	Response quantity due to imposed load
M_k	Modal mass of mode k
n	Number of storeys
N	SPT value for soil
P_k	Modal participation factor of mode k
Q_i	Lateral force at floor i
Q_{ik}	Design lateral force at floor i in mode k
r	Number of modes to be considered as per **7.8.4.2**
R	Response reduction factor
S_a/g	Average response acceleration coefficient for rock or soil sites as given by Fig. 2 and Table 3 based on appropriate natural periods and damping of the structure
T	Undamped natural period of vibration of the structure (in second)
T_a	Approximate fundamental period (in seconds)
T_k	Undamped natural period of mode k of vibration (in second)
T_1	Fundamental natural period of vibration (in second)
V_B	Design seismic base shear
$\bar{V}_B$	Design base shear calculated using the approximate fundamental period T_a
V_i	Peak storey shear force in storey i due to all modes considered
V_{ik}	Shear force in storey i in mode k
V_{roof}	Peak storey shear force at the roof due to all modes considered
W	Seismic weight of the structure
W_i	Seismic weight of floor i
Z	Zone factor
ϕ_{ik}	Mode shape coefficient at floor i in mode k
λ	Peak response (for example member forces, displacements, storey forces, storey shears or base reactions) due to all modes considered
λ_k	Absolute value of maximum response in mode k
λ_c	Absolute value of maximum response in mode c, where mode c is a closely-spaced mode.
λ^*	Peak response due to the closely-spaced modes only

ρ_{ij} Coefficient used in the Complete Quadratic Combination (CQC) method while combining responses of modes i and j

ω_i Circular frequency in rad/second in the i^{th} mode

6 GENERAL PRINCIPLES AND DESIGN CRITERIA

6.1 General Principles

6.1.1 *Ground Motion*

The characteristics (intensity, duration, etc) of seismic ground vibrations expected at any location depends upon the magnitude of earthquake, its depth of focus, distance from the epicentre, characteristics of the path through which the seismic waves travel, and the soil strata on which the structure stands. The random earthquake ground motions, which cause the structure to vibrate, can be resolved in any three mutually perpendicular directions. The predominant direction of ground vibration is usually horizontal.

Earthquake-generated vertical inertia forces are to be considered in design unless checked and proven by specimen calculations to be not significant. Vertical acceleration should be considered in structures with large spans, those in which stability is a criterion for design, or for overall stability analysis of structures. Reduction in gravity force due to vertical component of ground motions can be particularly detrimental in cases of prestressed horizontal members and of cantilevered members. Hence, special attention should be paid to the effect of vertical component of the ground motion on prestressed or cantilevered beams, girders and slabs.

6.1.2 The response of a structure to ground vibrations is a function of the nature of foundation soil; materials, form, size and mode of construction of structures; and the duration and characteristics of ground motion. This standard specifies design forces for structures standing on rocks or soils which do not settle, liquefy or slide due to loss of strength during ground vibrations.

6.1.3 The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (<DBE), which occur frequently, without damage; resist moderate earthquakes (DBE) without significant structural damage though some non-structural damage may occur; and aims that structures withstand a major earthquake (MCE) without collapse. Actual forces that appear on structures during earthquakes are much greater than the design forces specified in this standard. However, ductility, arising from inelastic material behaviour and detailing, and overstrength, arising from the additional reserve strength in structures over and above the design strength, are relied upon to account for this difference in actual and design lateral loads.

Reinforced and prestressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur, subject to the provisions of IS 456 and IS 1343. Provisions for appropriate ductile detailing of reinforced concrete members are given in IS 13920.

In steel structures, members and their connections should be so proportioned that high ductility is obtained, vide SP 6 (Part 6), avoiding premature failure due to elastic or inelastic buckling of any type.

The specified earthquake loads are based upon post-elastic energy dissipation in the structure and because of this fact, the provision of this standard for design, detailing and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.

6.1.4 *Soil-Structure Interaction*

The soil-structure interaction refers to the effects of the supporting foundation medium on the motion of structure. The soil-structure interaction may not be considered in the seismic analysis for structures supported on rock or rock-like material.

6.1.5 The design lateral force specified in this standard shall be considered in each of the two orthogonal horizontal directions of the structure. For structures which have lateral force resisting elements in the two orthogonal directions only, the design lateral force shall be considered along one direction at a time, and not in both directions simultaneously. Structures, having lateral force resisting elements (for example frames, shear walls) in directions other than the two orthogonal directions, shall be analysed considering the load combinations specified in **6.3.2**.

Where both horizontal and vertical seismic forces are taken into account, load combinations specified in **6.3.3** shall be considered.

6.1.6 Equipment and other systems, which are supported at various floor levels of the structure, will be subjected to motions corresponding to vibration at their support points. In important cases, it may be necessary to obtain floor response spectra for design of equipment supports. For detail reference be made to IS 1893 (Part 4).

6.1.7 *Additions to Existing Structures*

Additions shall be made to existing structures only as follows:

a) An addition that is structurally independent from an existing structures shall be designed and constructed in accordance with the seismic requirements for new structures.

b) An addition that is not structurally independent from an existing structure shall be designed and constructed such that the entire structure conforms to the seismic force resistance requirements for new structures unless the following three conditions are complied with:

1) The addition shall comply with the requirements for new structures,

2) The addition shall not increase the seismic forces in any structural elements of the existing structure by more than 5 percent unless the capacity of the element subject to the increased force is still in compliance with this standard, and

3) The addition shall not decrease the seismic resistance of any structural element of the existing structure unless reduced resistance is equal to or greater than that required for new structures.

6.1.8 *Change in Occupancy*

When a change of occupancy results in a structure being re-classified to a higher importance factor (I), the structure shall conform to the seismic requirements for a new structure with the higher importance factor.

6.2 Assumptions

The following assumptions shall be made in the earthquake resistant design of structures:

a) Earthquake causes impulsive ground motions, which are complex and irregular in character, changing in period and amplitude each lasting for a small duration. Therefore, resonance of the type as visualized under steady-state sinusoidal excitations, will not occur as it would need time to build up such amplitudes.

NOTE — However, there are exceptions where resonance-like conditions have been seen to occur between long distance waves and tall structures founded on deep soft soils.

b) Earthquake is not likely to occur simultaneously with wind or maximum flood or maximum sea waves.

c) The value of elastic modulus of materials, wherever required, may be taken as for static analysis unless a more definite value is available for use in such condition (*see* IS 456, IS 1343 and IS 800)

6.3 Load Combination and Increase in Permissible Stresses

6.3.1 *Load Combinations*

When earthquake forces are considered on a structure, these shall be combined as per **6.3.1.1** and **6.3.1.2** where the terms DL, IL and EL stand for the response quantities due to dead load, imposed load and designated earthquake load respectively.

6.3.1.1 *Load factors for plastic design of steel structures*

In the plastic design of steel structures, the following load combinations shall be accounted for:

1) $1.7 (DL + IL)$

2) $1.7 (DL \pm EL)$

3) $1.3 (DL + IL \pm EL)$

6.3.1.2 *Partial safety factors for limit state design of reinforced concrete and prestressed concrete structures*

In the limit state design of reinforced and prestressed concrete structures, the following load combinations shall be accounted for:

1) $1.5 (DL + IL)$

2) $1.2 (DL + IL \pm EL)$

3) $1.5 (DL \pm EL)$

4) $0.9 DL \pm 1.5 EL$

6.3.2 *Design Horizontal Earthquake Load*

6.3.2.1 When the lateral load resisting elements are oriented along orthogonal horizontal direction, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction at time.

6.3.2.2 When the lateral load resisting elements are not oriented along the orthogonal horizontal directions, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction plus 30 percent of the design earthquake load in the other direction.

NOTE — For instance, the building should be designed for ($\pm ELx \pm 0.3 ELy$) as well as ($\pm 0.3 ELx \pm ELy$), where x and y are two orthogonal horizontal directions. EL in **6.3.1.1** and **6.3.1.2** shall be replaced by ($ELx \pm 0.3 ELy$) or ($ELy \pm 0.3 ELx$).

6.3.3 *Design Vertical Earthquake Load*

When effects due to vertical earthquake loads are to be considered, the design vertical force shall be calculated in accordance with **6.4.5**.

6.3.4 *Combination for Two or Three Component Motion*

6.3.4.1 When responses from the three earthquake components are to be considered, the responses due to each component may be combined using the assumption that when the maximum response from one component occurs, the responses from the other two component are 30 percent of their maximum. All possible combinations of the three components (*ELx*, *ELy* and *ELz*) including variations in sign (plus or minus) shall be considered. Thus, the response due earthquake force (*EL*) is the maximum of the following three cases:

1) $\pm ELx \pm 0.3\, ELy \pm 0.3\, ELz$
2) $\pm ELy \pm 0.3\, ELx \pm 0.3\, ELz$
3) $\pm ELz \pm 0.3\, ELx \pm 0.3\, ELy$

where *x* and *y* are two orthogonal directions and *z* is vertical direction.

6.3.4.2 As an alternative to the procedure in **6.3.4.1**, the response (*EL*) due to the combined effect of the three components can be obtained on the basis of 'square root of the sum of the square (SRSS)' that is

$$EL = \sqrt{(ELx)^2 + (ELy)^2 + (ELz)^2}$$

NOTE — The combination procedure of **6.3.4.1** and **6.3.4.2** apply to the same response quantity (say, moment in a column about its major axis, or storey shear in a frame) due to different components of the ground motion.

6.3.4.3 When two component motions (say one horizontal and one vertical, or only two horizontal) are combined, the equations in **6.3.4.1** and **6.3.4.2** should be modified by deleting the term representing the response due to the component of motion not being considered.

6.3.5 *Increase in Permissible Stresses*

6.3.5.1 *Increase in permissible stresses in materials*

When earthquake forces are considered along with other normal design forces, the permissible stresses in material, in the elastic method of design, may be increased by one-third. However, for steels having a definite yield stress, the stress be limited to the yield stress; for steels without a definite yield point, the stress will be limited to 80 percent of the ultimate strength or 0.2 percent proof stress, whichever is smaller; and that in prestressed concrete members, the tensile stress in the extreme fibers of the concrete may be permitted so as not to exceed two-thirds of the modulus of rupture of concrete.

6.3.5.2 *Increase in allowable pressure in soils*

When earthquake forces are included, the allowable bearing pressure in soils shall be increased as per Table 1, depending upon type of foundation of the structure and the type of soil.

In soil deposits consisting of submerged loose sands and soils falling under classification SP with standard penetration *N*-values less than 15 in seismic Zones III, IV, V and less than 10 in seismic Zone II, the vibration caused by earthquake may cause liquefaction or excessive total and differential settlements. Such sites should preferably be avoided while locating new settlements or important projects. Otherwise, this aspect of the problem needs to be investigated and appropriate methods of compaction or stabilization adopted to achieve suitable *N*-values as indicated in Note 3 under Table 1. Alternatively, deep pile foundation may be provided and taken to depths well into the layer which is not likely to liquefy. Marine clays and other sensitive clays are also known to liquefy due to collapse of soil structure and will need special treatment according to site condition.

NOTE — Specialist literature may be referred for determining liquefaction potential of a site.

6.4 Design Spectrum

6.4.1 For the purpose of determining seismic forces, the country is classified into four seismic zones as shown in Fig. 1.

6.4.2 The design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$$A_h = \frac{Z\, I\, S_a}{2\, R\, g}$$

Provided that for any structure with $T \leq 0.1$ s, the value of A_h will not be taken less than $Z/2$ whatever be the value of I/R

where

Z = Zone factor given in Table 2, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE).

I = Importance factor, depending upon the functional use of the structures, characterised by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance (Table 6).

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterised by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0 (Table 7). The values of R for buildings are given in Table 7.

S_a/g = Average response acceleration coefficient

Table 1 Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils

(*Clause* 6.3.5.2)

Sl No.	Foundation	Type of Soil Mainly Constituting the Foundation		
		Type I Rock or Hard Soil : Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC)[1] having N[2] above 30, where N is the standard penetration value	*Type II Medium Soils :* All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines (SP[1]) with $N > 15$	*Type III Soft Soils :* All soils other than SP[1] with $N < 10$
(1)	(2)	(3)	(4)	(5)
i)	Piles passing through any soil but resting on soil type I	50	50	50
ii)	Piles not covered under item i	—	25	25
iii)	Raft foundations	50	50	50
iv)	Combined isolated RCC footing with tie beams	50	25	25
v)	Isolated RCC footing without tie beams, or unreinforced strip foundations	50	25	—
vi)	Well foundations	50	25	25

NOTES

1 The allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888.

2 If any increase in bearing pressure has already been permitted for forces other than seismic forces, the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits specified above.

3 Desirable minimum field values of N — If soils of smaller N-values are met, compacting may be adopted to achieve these values or deep pile foundations going to stronger strata should be used.

4 The values of N (corrected values) are at the founding level and the allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888.

Seismic Zone level (in metres)	*Depth Below Ground*	*N-Values*	*Remark*
III, IV and V	≤ 5	15	For values of depths between 5 m and 10 m, linear interpolation is recommended
	≥ 10	25	
II (for important structures only)	≤ 5 ≥ 10	15 20	

5 The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy.

6 IS 1498 and IS 2131 may also be referred.

7 Isolated R.C.C. footing without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with $N < 10$.

[1] *See* IS 1498.

[2] *See* IS 2131.

for rock or soil sites as given by Fig. 2 and Table 3 based on appropriate natural periods and damping of the structure. These curves represent free field ground motion.

NOTE — For various types of structures, the values of Importance Factor I, Response Reduction Factor R, and damping values are given in the respective parts of this standard. The method (empirical or otherwise) to calculate the natural periods of the structure to be adopted for evaluating S_a/g is also given in the respective parts of this standard.

Table 2 Zone Factor, Z

(*Clause* 6.4.2)

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

6.4.3 Where a number of modes are to be considered for dynamic analysis, the value of A_h as defined in **6.4.2** for each mode shall be determined using the natural period of vibration of that mode.

6.4.4 For underground structures and foundations at depths of 30 m or below, the design horizontal acceleration spectrum value shall be taken as half the value obtained from **6.4.2**. For structures and foundations placed between the ground level and 30 m depth, the design horizontal acceleration spectrum value shall be linearly interpolated between A_h and $0.5\,A_h$, where A_h is as specified in **6.4.2**.

6.4.5 The design acceleration spectrum for vertical motions, when required, may be taken as two-thirds of the design horizontal acceleration spectrum specified in **6.4.2**.

Figure 2 shows the proposed 5 percent spectra for rocky and soils sites and Table 3 gives the multiplying factors for obtaining spectral values for various other dampings.

For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.40 \\ 1.00/T & 0.40 \le T \le 4.00 \end{cases}$$

For medium soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.55 \\ 1.36/T & 0.55 \le T \le 4.00 \end{cases}$$

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15\,T; & 0.00 \le T \le 0.10 \\ 2.50 & 0.10 \le T \le 0.67 \\ 1.67/T & 0.67 \le T \le 4.00 \end{cases}$$

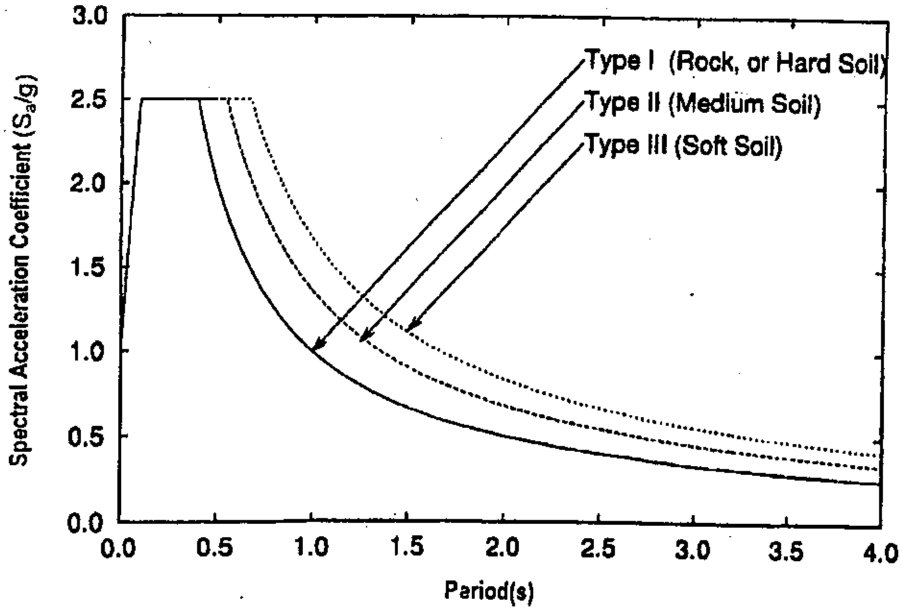


FIG. 2 RESPONSE SPECTRA FOR ROCK AND SOIL SITES FOR 5 PERCENT DAMPING

6.4.6 In case design spectrum is specifically prepared for a structure at a particular project site, the same may be used for design at the discretion of the project authorities

7 BUILDINGS

7.1 Regular and Irregular Configuration

To perform well in an earthquake, a building should possess four main attributes, namely simple and regular configuration, and adequate lateral strength, stiffness and ductility. Buildings having simple regular geometry and uniformly distributed mass and stiffness in plan as well as in elevation, suffer much less damage than buildings with irregular configurations. A building shall be considered as irregular for the purposes of this standard, if at least one of the conditions given in Tables 4 and 5 is applicable.

7.2 Importance Factor *I* and Response Reduction Factor *R*

The minimum value of importance factor, *I*, for different building systems shall be as given in Table 6. The response reduction factor, *R*, for different building systems shall be as given in Table 7.

7.3 Design Imposed Loads for Earthquakes Force Calculation

7.3.1 For various loading classes as specified in IS 875 (Part 2), the earthquake force shall be calculated for the full dead load plus the percentage of imposed load as given in Table 8.

7.3.2 For calculating the design seismic forces of the structure, the imposed load on roof need not be considered.

7.3.3 The percentage of imposed loads given in 7.3.1 and 7.3.2 shall also be used for 'Whole frame loaded' condition in the load combinations specified in 6.3.1.1 and 6.3.1.2 where the gravity loads are combined with the earthquake loads [that is, in load combinations (3) in 6.3.1.1, and (2) in 6.3.1.2]. No further reduction in the imposed load will be used as envisaged in IS 875 (Part 2) for number of storeys above the one under consideration or for large spans of beams or floors.

7.3.4 The proportions of imposed load indicated above for calculating the lateral design forces for earthquakes are applicable to average conditions. Where the probable loads at the time of earthquake are more accurately assessed, the designer may alter the proportions indicated or even replace the entire imposed load proportions by the actual assessed load. In such cases, where the imposed load is not assessed as per 7.3.1 and 7.3.2 only that part of imposed load, which possesses mass, shall be considered. Lateral design force for earthquakes shall not be calculated on contribution of impact effects from imposed loads.

7.3.5 Other loads apart from those given above (for example snow and permanent equipment) shall be considered as appropriate.

7.4 Seismic Weight

7.4.1 *Seismic Weight of Floors*

The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in 7.3.1 and 7.3.2. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above and below the storey.

7.4.2 *Seismic Weight of Building*

The seismic weight of the whole building is the sum of the seismic weights of all the floors.

7.4.3 Any weight supported in between storeys shall be distributed to the floors above and below in inverse proportion to its distance from the floors.

Table 3 Multiplying Factors for Obtaining Values for Other Damping

(*Clause* 6.4.2)

Damping, percent	0	2	5	7	10	15	20	25	30
Factors	3.20	1.40	1.00	0.90	0.80	0.70	0.60	0.55	0.50

Table 4 Definitions of Irregular Buildings — Plan Irregularities (Fig. 3)

(*Clause* 7.1)

Sl No.	Irregularity Type and Description
(1)	(2)
i)	*Torsion Irregularity* To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure
ii)	*Re-entrant Corners* Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction
iii)	*Diaphragm Discontinuity* Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next
iv)	*Out-of-Plane Offsets* Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements
v)	*Non-parallel Systems* The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements

Table 5 Definition of Irregular Buildings — Vertical Irregularities (Fig. 4)

(*Clause* 7.1)

Sl No.	Irregularity Type and Description
(1)	(2)
i)	a) *Stiffness Irregularity — Soft Storey* A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above b) *Stiffness Irregularity — Extreme Soft Storey* A extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent of the average stiffness of the three storeys above. For example, buildings on STILTS will fall under this category.

Table 5 — *Concluded*

Sl No.	Irregularity Type and Description
(1)	(2)
ii)	*Mass Irregularity* Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs
iii)	*Vertical Geometric Irregularity* Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey
iv)	*In-Plane Discontinuity in Vertical Elements Resisting Lateral Force* A in-plane offset of the lateral force resisting elements greater than the length of those elements
v)	*Discontinuity in Capacity — Weak Strorey* A weak storey is one in which the storey lateral strength is less than 80 percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

Table 6 Importance Factors, *I*

(*Clause* 6.4.2)

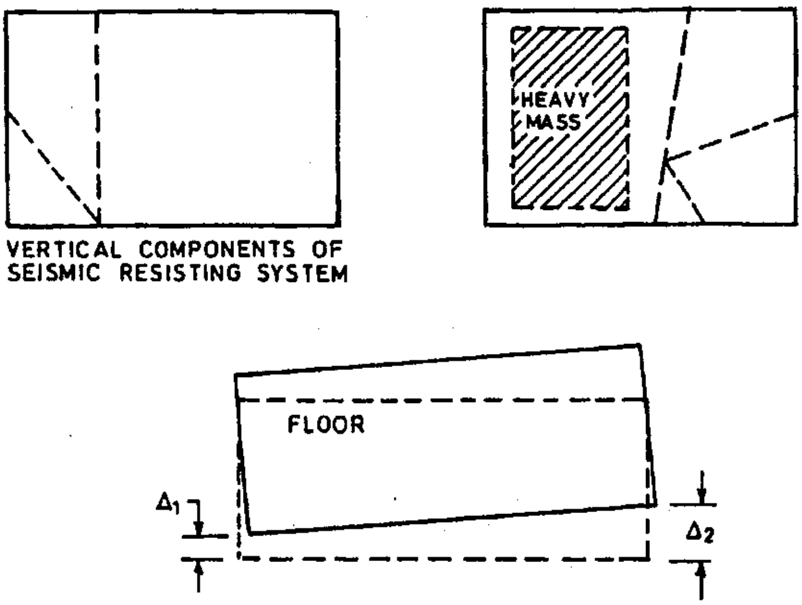
Sl No.	Structure	Importance Factor
(1)	(2)	(3)
i)	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations, power stations	1.5
ii)	All other buildings	1.0

NOTES

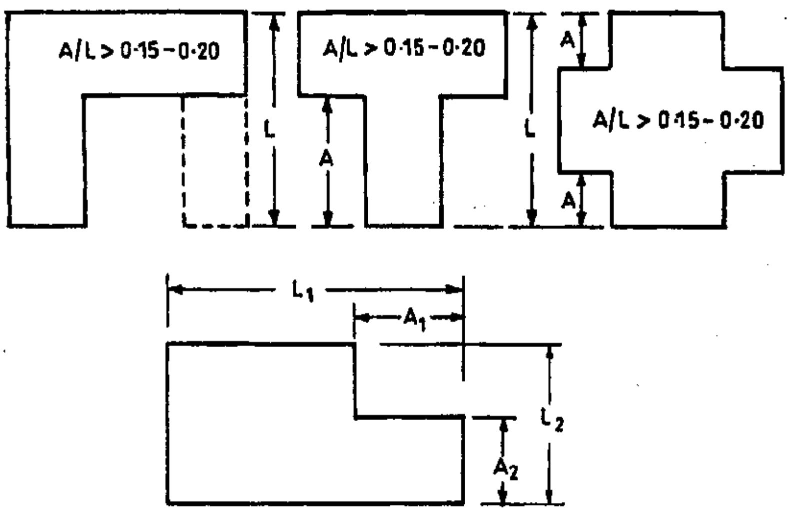
1 The design engineer may choose values of importance factor *I* greater than those mentioned above.

2 Buildings not covered in Sl No. (i) and (ii) above may be designed for higher value of *I*, depending on economy, strategy considerations like multi-storey buildings having several residential units.

3 This does not apply to temporary structures like excavations, scaffolding etc of short duration.

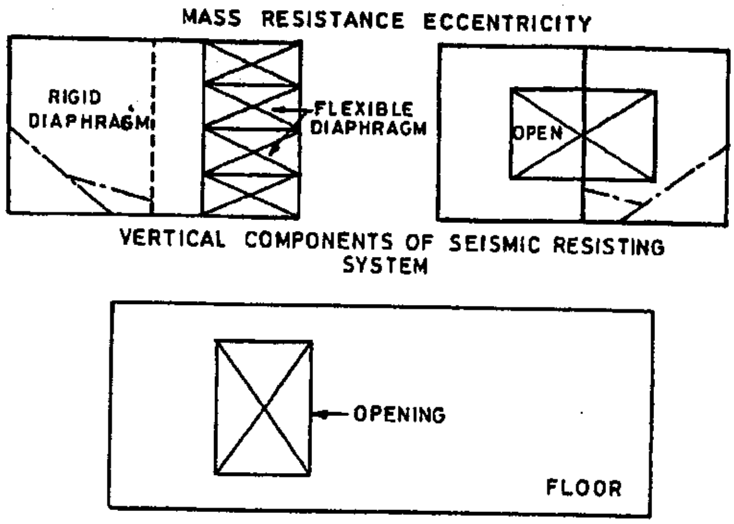


3 A Torsional Irregularity

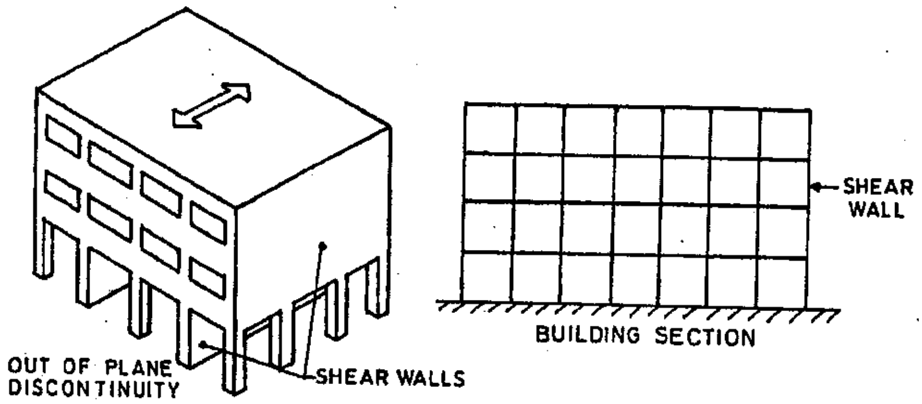


3 B Re-entrant Corner

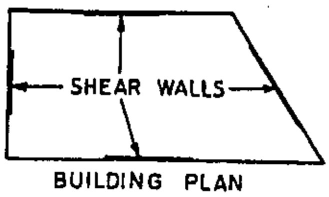
FIG. 3 PLAN IRREGULARITIES — *Continued*



3 C Diaphragm Discontinuity

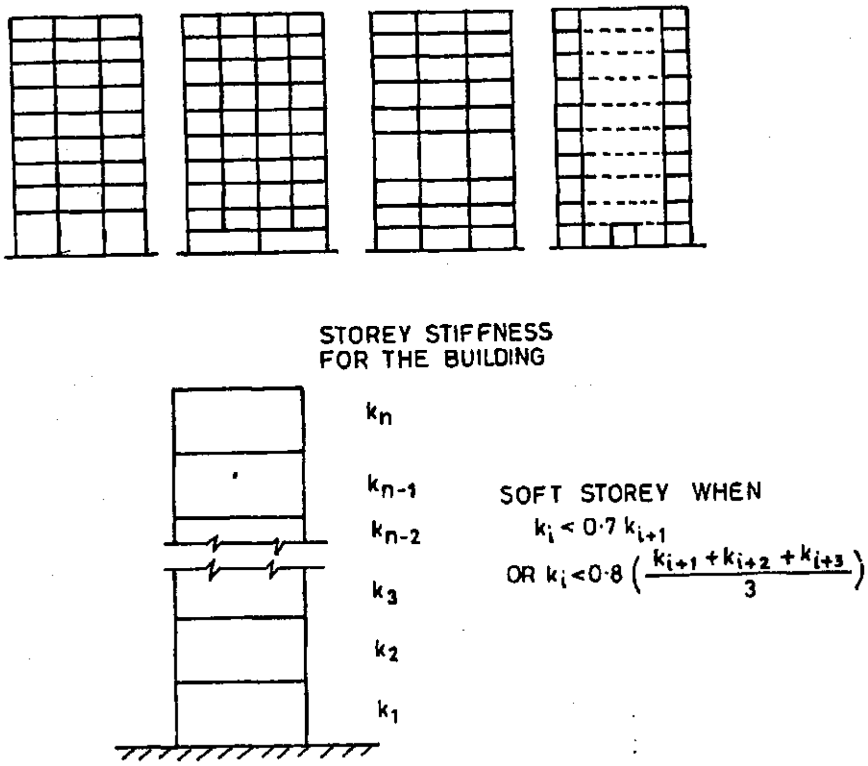


3 D Out-of-Plane Offsets

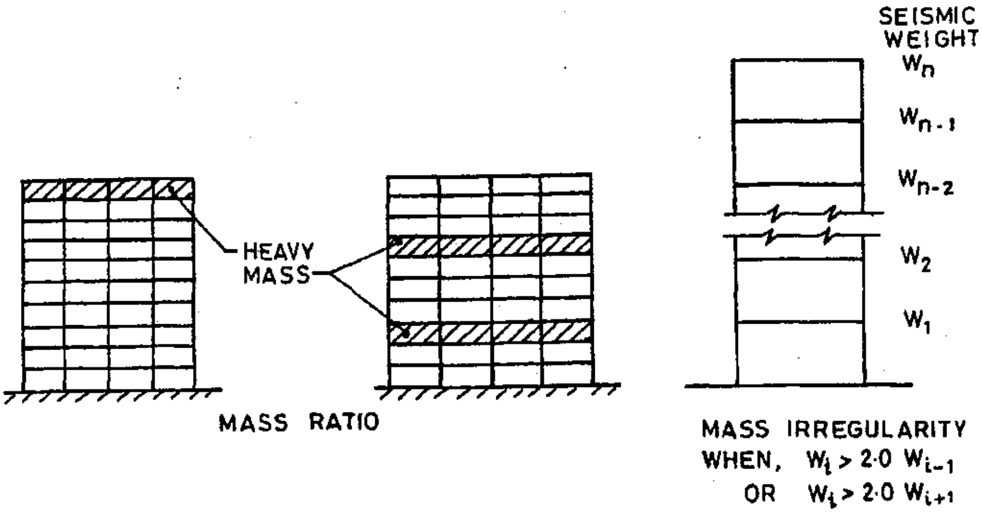


3 E Non-Parallel System

FIG. 3 PLAN IRREGULARITIES

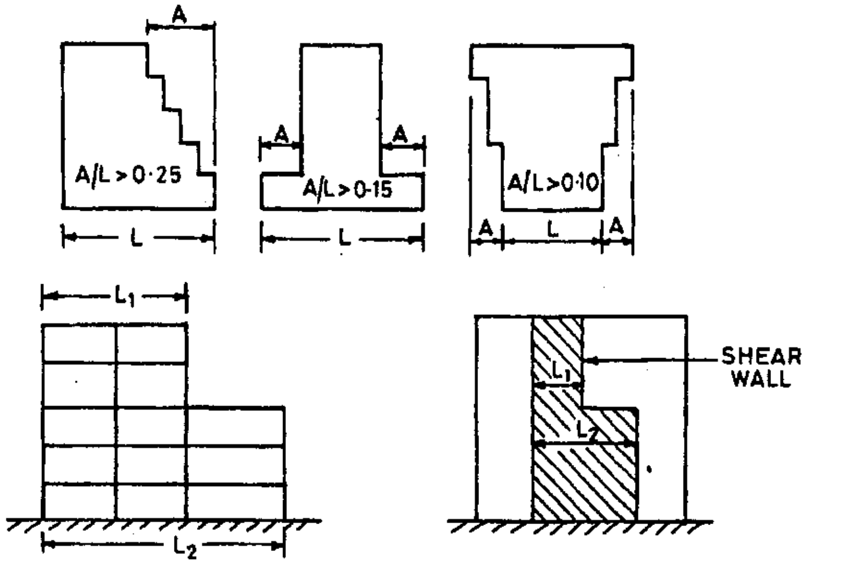


4 A Stiffness Irregularity

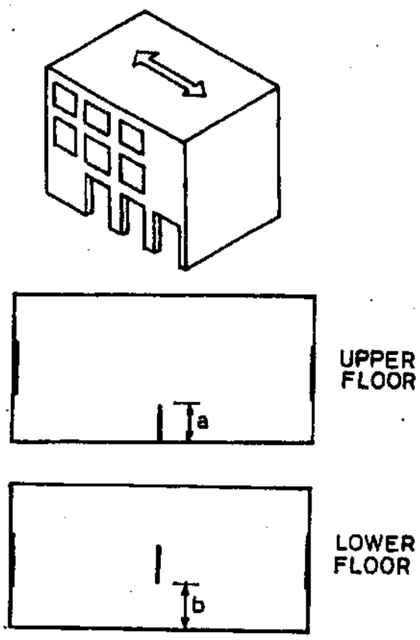


4 B Mass Irregularity

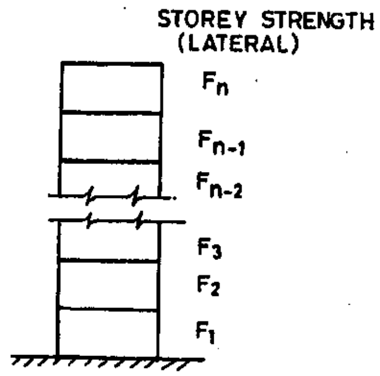
FIG. 4 VERTICAL IRREGULARITIES — *Continued*



4 C Vertical Geometric Irregularity when $L_2 > 1.5\ L_1$



4 D In-Plane Discontinuity in Vertical Elements Resisting Lateral Force when $b > a$



4 E Weak Storey when $F_i < 0.8\ F_{i+1}$

FIG. 4 VERTICAL IRREGULARITIES

Table 7 Response Reduction Factor [1], *R*, for Building Systems

(*Clause* 6.4.2)

Sl No.	Lateral Load Resisting System	*R*
(1)	(2)	(3)
	Building Frame Systems	
i)	Ordinary RC moment-resisting frame (OMRF) [2]	3.0
ii)	Special RC moment-resisting frame (SMRF) [3]	5.0
iii)	Steel frame with	
	a) Concentric braces	4.0
	b) Eccentric braces	5.0
iv)	Steel moment resisting frame designed as per SP 6 (6)	5.0
	Building with Shear Walls [4]	
v)	Load bearing masonry wall buildings [5]	
	a) Unreinforced	1.5
	b) Reinforced with horizontal RC bands	2.5
	c) Reinforced with horizontal RC bands and vertical bars at corners of rooms and jambs of openings	3.0
vi)	Ordinary reinforced concrete shear walls [6]	3.0
vii)	Ductile shear walls [7]	4.0
	Buildings with Dual Systems [8]	
viii)	Ordinary shear wall with OMRF	3.0
ix)	Ordinary shear wall with SMRF	4.0
x)	Ductile shear wall with OMRF	4.5
xi)	Ductile shear wall with SMRF	5.0

[1] The values of response reduction factors are to be used for buildings with lateral load resisting elements, and not just for the lateral load resisting elements built in isolation.

[2] OMRF are those designed and detailed as per IS 456 or IS 800 but not meeting ductile detailing requirement as per IS 13920 or SP 6 (6) respectively.

[3] SMRF defined in 4.15.2.

[4] Buildings with shear walls also include buildings having shear walls and frames, but where:

a) frames are not designed to carry lateral loads, or

b) frames are designed to carry lateral loads but do not fulfil the requirements of 'dual systems'.

[5] Reinforcement should be as per IS 4326.

[6] Prohibited in zones IV and V.

[7] Ductile shear walls are those designed and detailed as per IS 13920.

[8] Buildings with dual systems consist of shear walls (or braced frames) and moment resisting frames such that:

a) the two systems are designed to resist the total design force in proportion to their lateral stiffness considering the interaction of the dual system at all floor levels; and

b) the moment resisting frames are designed to independently resist at least 25 percent of the design seismic base shear.

Table 8 Percentage of Imposed Load to be Considered in Seismic Weight Calculation

(*Clause* 7.3.1)

Imposed Uniformity Distributed Floor Loads (kN/m^2)	Percentage of Imposed Load
(1)	(2)
Upto and including 3.0	25
Above 3.0	50

7.5 Design Lateral Force

7.5.1 Buildings and portions thereof shall be designed and constructed, to resist the effects of design lateral force specified in **7.5.3** as a minimum.

7.5.2 The design lateral force shall first be computed for the building as a whole. This design lateral force shall then be distributed to the various floor levels. The overall design seismic force thus obtained at each floor level, shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

7.5.3 *Design Seismic Base Shear*

The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expression:

$$V_B = A_h W$$

where

A_h = Design horizontal acceleration spectrum value as per **6.4.2**, using the fundamental natural period T_a as per **7.6** in the considered direction of vibration; and

W = Seismic weight of the building as per **7.4.2**.

7.6 Fundamental Natural Period

7.6.1 The approximate fundamental natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick infil panels may be estimated by the empirical expression:

$$T_a = 0.075\,h^{0.75} \quad \text{for RC frame building}$$
$$= 0.085\,h^{0.75} \quad \text{for steel frame building}$$

where

h = Height of building, in m. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

7.6.2 The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with brick infil panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09}{\sqrt{d}}$$

where

h = Height of building, in m, as defined in **7.6.1**; and

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

7.7 Distribution of Design Force

7.7.1 *Vertical Distribution of Base Shear to Different Floor Levels*

The design base shear (V_B) computed in **7.5.3** shall be distributed along the height of the building as per the following expression:

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$$

where

Q_i = Design lateral force at floor i,

W_i = Seismic weight of floor i,

h_i = Height of floor i measured from base, and

n = Number of storeys in the building is the number of levels at which the masses are located.

7.7.2 *Distribution of Horizontal Design Lateral Force to Different Lateral Force Resisting Elements*

7.7.2.1 In case of buildings whose floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane shall be distributed to the various vertical elements of lateral force resisting system, assuming the floors to be infinitely rigid in the horizontal plane.

7.7.2.2 In case of building whose floor diaphragms can not be treated as infinitely rigid in their own plane, the lateral shear at each floor shall be distributed to the vertical elements resisting the lateral forces, considering the in-plane flexibility of the diaphragms.

NOTES

1 A floor diaphragm shall be considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.5 times the average displacement of the entire diaphragm.

2 Reinforced concrete monolithic slab-beam floors or those consisting of prefabricated/precast elements with topping reinforced screed can be taken a rigid diaphragms.

7.8 Dynamic Analysis

7.8.1 Dynamic analysis shall be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

a) *Regular buildings* — Those greater than 40 m in height in Zones IV and V, and those greater than 90 m in height in Zones II and III. Modelling as per 7.8.4.5 can be used.

b) *Irregular buildings* (*as defined in* 7.1) — All framed buildings higher than 12 m in Zones IV and V, and those greater than 40 m in height in Zones II and III.

The analytical model for dynamic analysis of buildings with unusual configuration should be such that it adequately models the types of irregularities present in the building configuration. Buildings with plan irregularities, as defined in Table 4 (as per 7.1), cannot be modelled for dynamic analysis by the method given in 7.8.4.5.

NOTE — For irregular buildings, lesser than 40 m in height in Zones II and III, dynamic analysis, even though not mandatory, is recommended.

7.8.2 Dynamic analysis may be performed either by the Time History Method or by the Response Spectrum Method. However, in either method, the design base shear (V_B) shall be compared with a base shear ($\bar{V}_B$) calculated using a fundamental period T_a, where T_a is as per 7.6. Where V_B is less than $\bar{V}_B$, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by $\bar{V}_B / V_B$.

7.8.2.1 The value of damping for buildings may be taken as 2 and 5 percent of the critical, for the purposes of dynamic analysis of steel and reinforced concrete buildings, respectively.

7.8.3 *Time History Method*

Time history method of analysis, when used, shall be based on an appropriate ground motion and shall be performed using accepted principles of dynamics.

7.8.4 *Response Spectrum Method*

Response spectrum method of analysis shall be performed using the design spectrum specified in 6.4.2, or by a site-specific design spectrum mentioned in 6.4.6.

7.8.4.1 *Free Vibration Analysis*

Undamped free vibration analysis of the entire building shall be performed as per established methods of mechanics using the appropriate masses and elastic stiffness of the structural system, to obtain natural periods (T) and mode shapes $\{\phi\}$ of those of its modes of vibration that need to be considered as per 7.8.4.2.

7.8.4.2 *Modes to be considered*

The number of modes to be used in the analysis should be such that the sum total of modal masses of all modes considered is at least 90 percent of the total seismic mass and missing mass correction beyond 33 percent. If modes with natural frequency beyond 33 Hz are to be considered, modal combination shall be carried out only for modes upto 33 Hz. The effect of higher modes shall be included by considering missing mass correction following well established procedures.

7.8.4.3 *Analysis of building subjected to design forces*

The building may be analyzed by accepted principles of mechanics for the design forces considered as static forces.

7.8.4.4 *Modal combination*

The peak response quantities (for example, member forces, displacements, storey forces, storey shears and base reactions) shall be combined as per Complete Quadratic Combination (CQC) method.

$$\lambda = \sqrt{\sum_{i=1}^{r} \sum_{j=1}^{r} \lambda_i \rho_{ij} \lambda_j}$$

where

r = Number of modes being considered,

ρ_{ij} = Cross-modal coefficient,

λ_i = Response quantity in mode i (including sign),

λ_j = Response quantity in mode j (including sign),

$$\rho_{ij} = \frac{8\varsigma^2 (1+\beta)\beta^{1.5}}{(1-\beta^2)^2 + 4\varsigma^2\beta(1+\beta)^2}$$

ς = Modal damping ratio (in fraction) as specified in 7.8.2.1,

β = Frequency ratio = ω_j/ω_i,

ω_i = Circular frequency in ith mode, and

ω_j = Circular frequency in jth mode.

Alternatively, the peak response quantities may be combined as follows:

a) If the building does not have closely-spaced modes, then the peak response quantity (λ) due to all modes considered shall be obtained as

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2}$$

where

λ_k = Absolute value of quantity in mode k, and

r = Number of modes being considered.

b) If the building has a few closely-spaced modes (*see* **3.2**), then the peak response quantity (λ^*) due to these modes shall be obtained as

$$\lambda^* = \sum_{c}^{r} \lambda_c$$

where the summation is for the closely-spaced modes only. This peak response quantity due to the closely spaced modes (λ^*) is then combined with those of the remaining well-separated modes by the method described in **7.8.4.4** (a).

7.8.4.5 Buildings with regular, or nominally irregular plan configurations may be modelled as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration. In such a case, the following expressions shall hold in the computation of the various quantities :

a) *Modal Mass* — The modal mass (M_k) of mode k is given by

$$M_k = \frac{\left[\sum_{i=1}^{n} W_i \phi_{ik}\right]^2}{g \sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

where

g = Acceleration due to gravity,

ϕ_{ik} = Mode shape coefficient at floor i in mode k, and

W_i = Seismic weight of floor i.

b) *Modal Participation Factors* — The modal participation factor (P_k) of mode k is given by:

$$P_k = \frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

c) *Design Lateral Force at Each Floor in Each Mode* — The peak lateral force (Q_{ik}) at floor i in mode k is given by

$$Q_{ik} = A_k \phi_{ik} P_k W_i$$

where

A_k = Design horizontal acceleration spectrum value as per **6.4.2** using the natural period of vibration (T_k) of mode k.

d) *Storey Shear Forces in Each Mode* — The peak shear force (V_{ik}) acting in storey i in mode k is given by

$$V_{ik} = \sum_{j=i+1}^{n} Q_{ik}$$

e) *Storey Shear Forces due to All Modes Considered* — The peak storey shear force (V_i) in storey i due to all modes considered is obtained by combining those due to each mode in accordance with **7.8.4.4**.

f) *Lateral Forces at Each Storey Due to All Modes Considered* — The design lateral forces, F_{roof} and F_i, at roof and at floor i :

$$F_{roof} = V_{roof}, \text{ and}$$

$$F_i = V_i - V_{i+1}$$

7.9 Torsion

7.9.1 Provision shall be made in all buildings for increase in shear forces on the lateral force resisting elements resulting from the horizontal torsional moment arising due to eccentricity between the centre of mass and centre of rigidity. The design forces calculated as in **7.8.4.5** are to be applied at the centre of mass appropriately displaced so as to cause design eccentricity (**7.9.2**) between the displaced centre of mass and centre of rigidity. However, negative torsional shear shall be neglected.

7.9.2 The design eccentricity, e_{di} to be used at floor i shall be taken as:

$$e_{di} = \begin{cases} 1.5 e_{si} + 0.05\, b_i \\ \text{or} \quad e_{si} - 0.05\, b_i \end{cases}$$

whichever of these gives the more severe effect in the shear of any frame where

e_{di} = Static eccentricity at floor i defined as the distance between centre of mass and centre of rigidity, and

b_i = Floor plan dimension of floor i, perpendicular to the direction of force.

NOTE — The factor 1.5 represents dynamic amplification factor, while the factor 0.05 represents the extent of accidental eccentricity.

7.9.3 In case of highly irregular buildings analyzed according to **7.8.4.5**, additive shears will be superimposed for a statically applied eccentricity of $\pm 0.05 b_i$ with respect to the centre of rigidity.

7.10 Buildings with Soft Storey

7.10.1 In case buildings with a flexible storey, such as the ground storey consisting of open spaces for parking that is Stilt buildings, special arrangement needs to be made to increase the lateral strength and stiffness of the soft/open storey.

7.10.2 Dynamic analysis of building is carried out including the strength and stiffness effects of infills and inelastic deformations in the members, particularly, those in the soft storey, and the members designed accordingly.

7.10.3 Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys:

a) the columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads specified in the other relevant clauses; or,

b) besides the columns designed and detailed for the calculated storey shears and moments, shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible; to be designed exclusively for 1.5 times the lateral storey shear force calculated as before.

7.11 Deformations

7.11.1 *Storey Drift Limitation*

The storey drift in any storey due to the minimum specified design lateral force, with partial load factor of 1.0, shall not exceed 0.004 times the storey height.

For the purposes of displacement requirements only (*see* **7.11.1**, **7.11.2** and **7.11.3** only), it is permissible to use seismic force obtained from the computed fundamental period (T) of the building without the lower bound limit on design seismic force specified in **7.8.2**.

There shall be no drift limit for single storey building which has been designed to accommodate storey drift.

7.11.2 *Deformation Compatibility of Non-Seismic Members*

For building located in seismic Zones IV and V, it shall be ensured that the structural components, that are not a part of the seismic force resisting system in the direction under consideration, do not lose their vertical load-carrying capacity under the induced moments resulting from storey deformations equal to R times the storey displacements calculated as per **7.11.1**, where R is specified in Table 7.

NOTE — For instance, consider a flat-slab building in which lateral load resistance is provided by shear walls. Since the lateral load resistance of the slab-column system is small, these are often designed only for the gravity loads, while all the seismic force is resisted by the shear walls. Even though the slabs and columns are not required to share the lateral forces, these deform with rest of the structure under seismic force. The concern is that under such deformations, the slab-column system should not lose its vertical load capacity.

7.11.3 *Separation Between Adjacent Units*

Two adjacent buildings, or two adjacent units of the same building with separation joint in between shall be separated by a distance equal to the amount R times the sum of the calculated storey displacements as per **7.11.1** of each of them, to avoid damaging contact when the two units deflect towards each other. When floor levels of two similar adjacent units or buildings are at the same elevation levels, factor R in this requirement may be replaced by $R/2$.

7.12 Miscellaneous

7.12.1 *Foundations*

The use of foundations vulnerable to significant differential settlement due to ground shaking shall be avoided for structures in seismic Zones III, IV and V. In seismic Zones IV and V, individual spread footings or pile caps shall be interconnected with ties, (*see* **5.3.4.1** of IS 4326) except when individual spread footings are directly supported on rock. All ties shall be capable of carrying, in tension and in compression, an axial force equal to $A_h/4$ times the larger of the column or pile cap load, in addition to the otherwise computed forces. Here, A_h is as per **6.4.2**.

7.12.2 *Cantilever Projections*

7.12.2.1 *Vertical projections*

Tower, tanks, parapets, smoke stacks (chimneys) and other vertical cantilever projections attached to buildings and projecting above the roof, shall be designed and checked for stability for five times the design horizontal seismic coefficient A_h specified in **6.4.2**. In the analysis of the building, the weight of these projecting elements will be lumped with the roof weight.

7.12.2.2 *Horizontal projection*

All horizontal projections like cornices and balconies shall be designed and checked for stability for five times the design vertical coefficient specified in 6.4.5 (that is = 10/3 A_h).

7.12.2.3 The increased design forces specified in 7.12.2.1 and 7.12.2.2 are only for designing the projecting parts and their connections with the main structures. For the design of the main structure, such increase need not be considered.

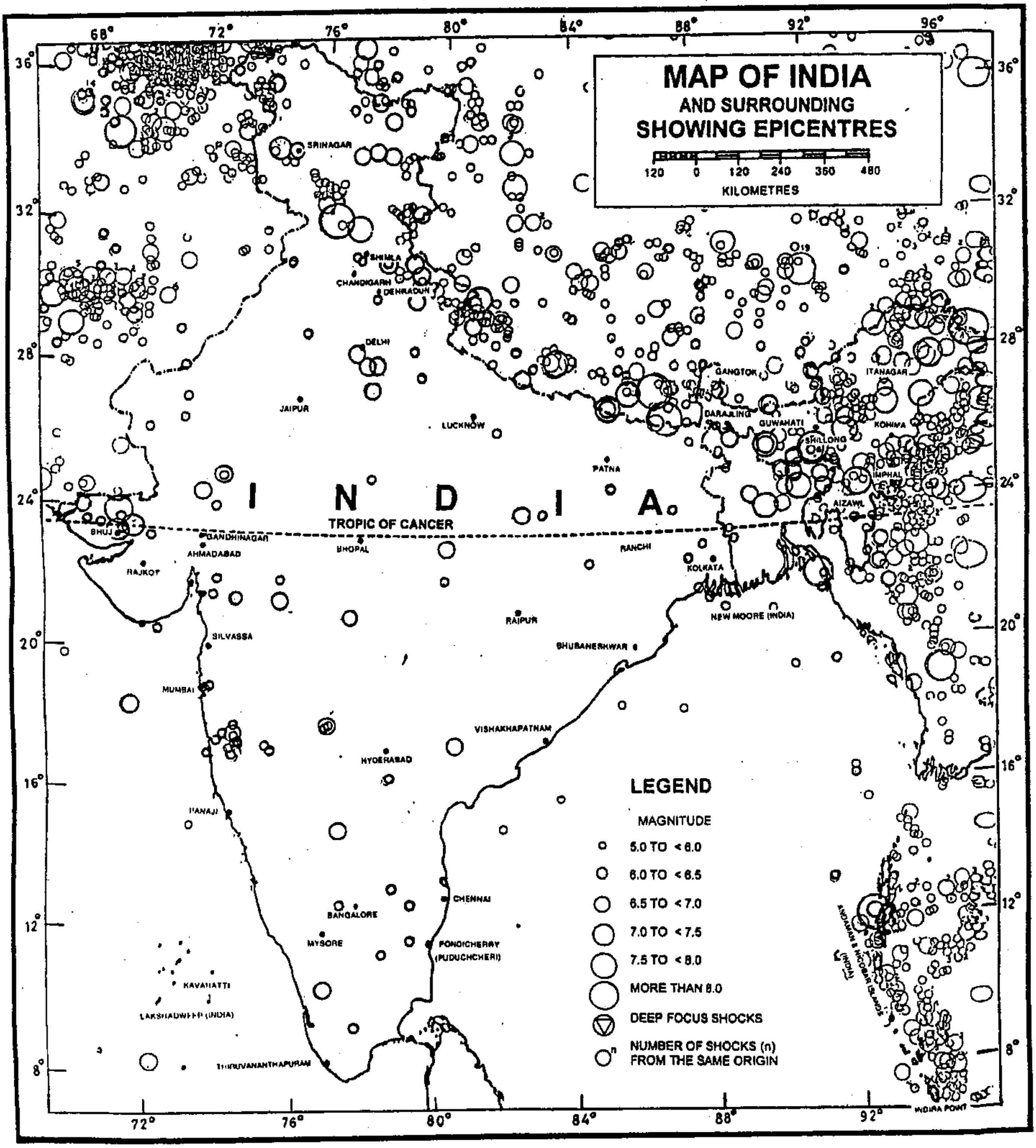
7.12.3 *Compound Walls*

Compound walls shall be designed for the design horizontal coefficient A_h with importance factor I = 1.0 specified in 6.4.2.

7.12.4 *Connections Between Parts*

All parts of the building, except between the separation sections, shall be tied together to act as integrated single unit. All connections between different parts, such as beams to columns and columns to their footings, should be made capable of transmitting a force, in all possible directions, of magnitude (Q_i/W_i) times but not less than 0.05 times the weight of the smaller part or the total of dead and imposed load reaction. Frictional resistance shall not be relied upon for fulfilling these requirements.

ANNEX A
(Foreword)



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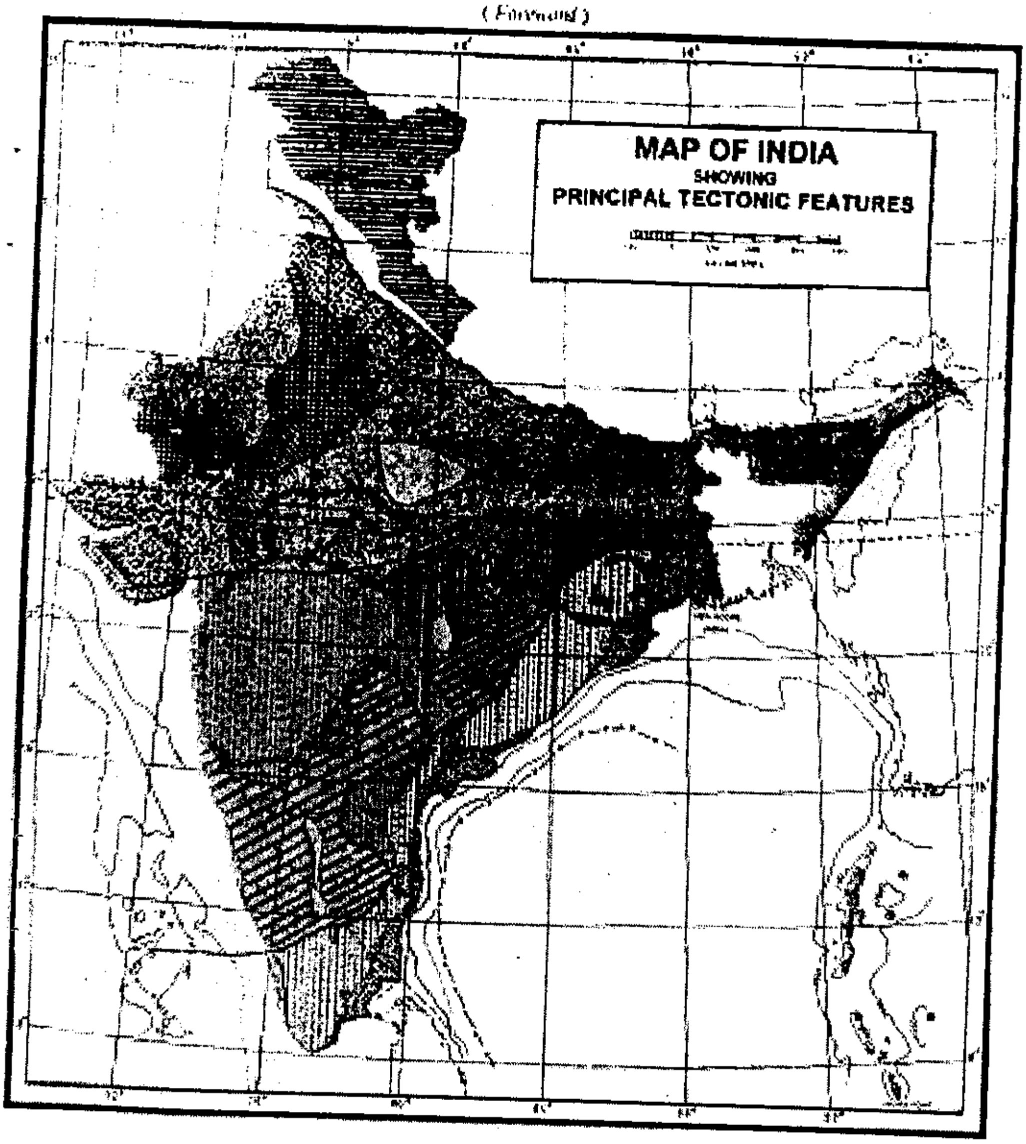
The territorial waters of India extend into the sea to distance of twelve nautical miles measured from the appropriate base line.

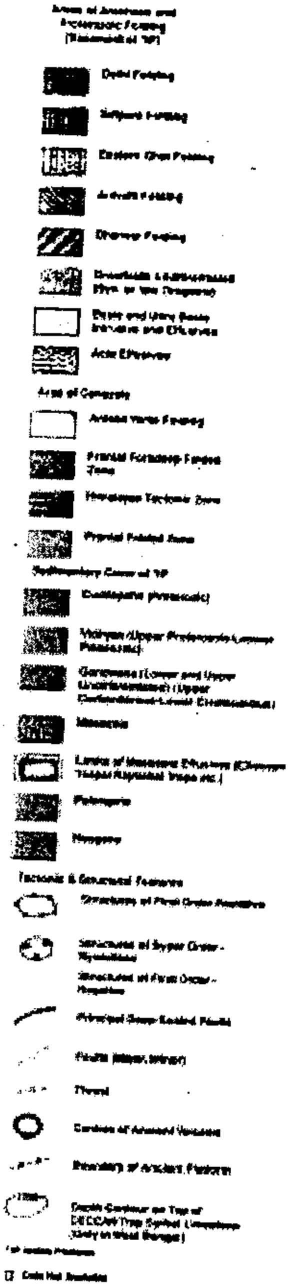
The administrative headquarters of Chandigarh, Haryana and Punjab are at Chandigarh.

The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.

The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.

ANNEX B
(Foreword)





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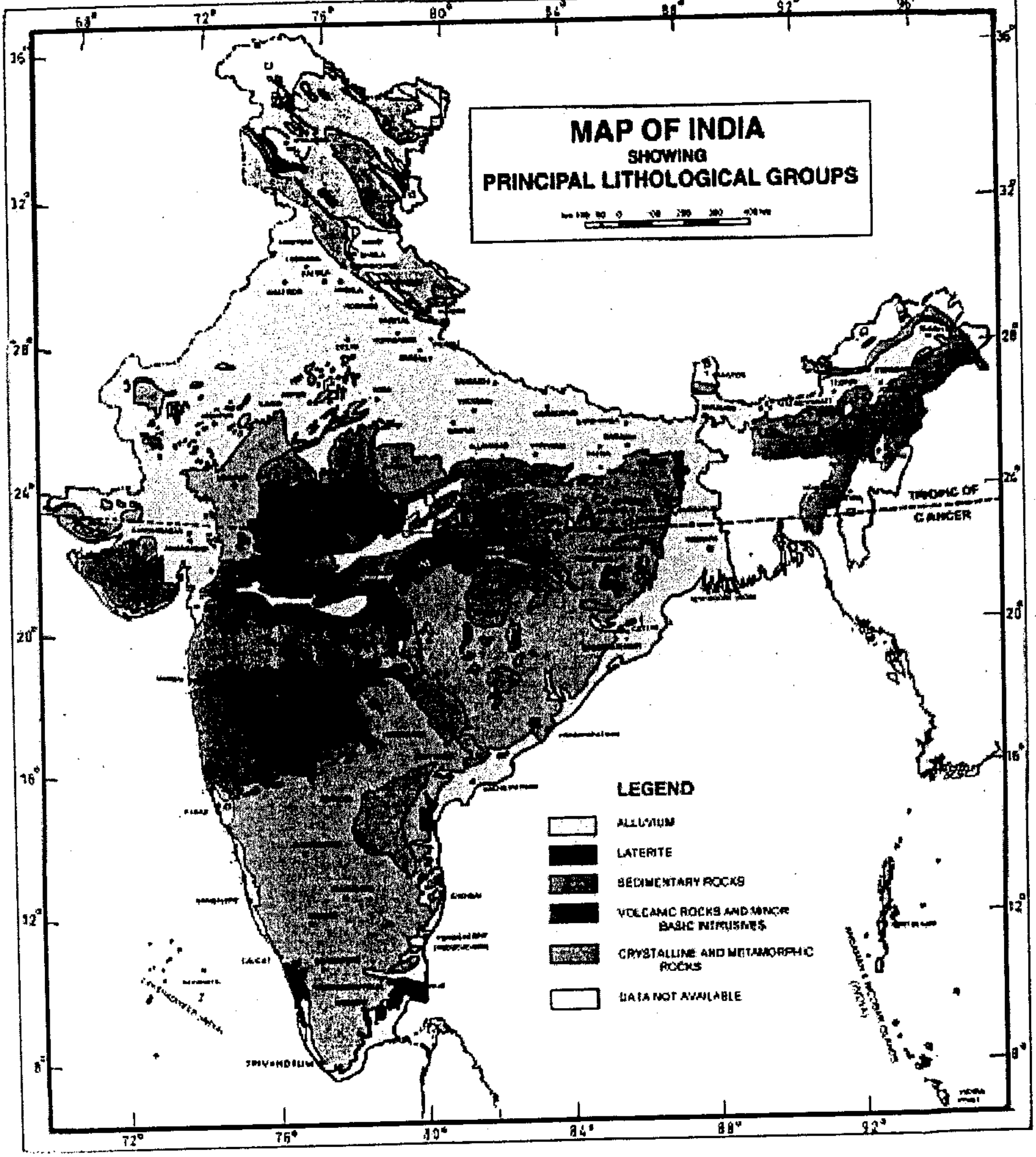
ANNEX C

(Foreword)



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ANNEX D

(*Foreword* and *Clause* 3.15)

COMPREHENSIVE INTENSITY SCALE (MSK 64)

The scale was discussed generally at the inter-governmental meeting convened by UNESCO in April 1964. Though not finally approved the scale is more comprehensive and describes the intensity of earthquake more precisely. The main definitions used are followings;

a) *Type of Structures (Buildings)*

Type A — Building in field-stone, rural structures, unburnt-brick houses, clay houses.

Type B — Ordinary brick buildings, buildings of large block and prefabricated type, half timbered structures, buildings in natural hewn stone.

Type C — Reinforced buildings, well built wooden structures.

b) *Definition of Quantity:*

Single, few	About 5 percent
Many	About 50 percent
Most	About 75 percent

c) *Classification of Damage to Buildings*

Grade 1	Slight damage	Fine cracks in plaster; fall of small pieces of plaster.
Grade 2	Moderate damage	Small cracks in plaster; fall of fairly large pieces of plaster: pantiles slip off: cracks in chimneys parts of chimney fall down.
Grade 3	Heavy damage	Large and deep cracks in plaster: fall of chimneys.
Grade 4	Destruction	Gaps in walls: parts of buildings may collapse: separate parts of the buildings lose their cohesion: and inner walls collapse.
Grade 5	Total damage	Total collapse of the buildings.

d) *Intensity Scale*

1. *Not noticeable* — The intensity of the vibration is below the limits of sensibility: the tremor is detected and recorded by seismograph only.

2. *Scarcely noticeable (very slight)* — Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.

3. *Weak, partially observed only* — The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects, somewhat more heavily on upper floors.

4. *Largely observed* — The earthquake is felt indoors by many people, outdoors by few. Here and there people awake, but no one is frightened. The vibration is like that due to the passing of a heavily loaded truck. Windows, doors, and dishes rattle. Floors and walls crack. Furniture begins to shake. Hanging objects swing slightly. Liquid in open vessels are slightly disturbed. In standing motor cars the shock is noticeable.

5. *Awakening*

i) The earthquake is felt indoors by all, outdoors by many. Many people awake. A few run outdoors. Animals become uneasy. Building tremble throughout. Hanging objects swing considerably. Pictures knock against walls or swing out of place. Occasionally pendulum clocks stop. Unstable objects overturn or shift. Open doors and windows are thrust open and slam back again. Liquids spill in small amounts from well-filled open containers. The sensation of vibration is like that due to heavy objects falling inside the buildings.

ii) Slight damages in buildings of Type A are possible.

iii) Sometimes changes in flow of springs.

6. *Frightening*

i) Felt by most indoors and outdoors. Many people in buildings are frightened and run outdoors. A few persons loose their balance. Domestic animals run out of their stalls. In few instances, dishes and glassware may break, and books fall down. Heavy furniture may possibly move and small steeple bells may ring.

ii) Damage of Grade 1 is sustained in single buildings of Type B and in many of Type A. Damage in few buildings of Type A is of Grade 2.

iii) In few cases, cracks up to widths of 1 cm possible in wet ground; in mountains occasional landslips; change in flow of springs and in level of well water are observed.

7. *Damage of buildings*

i) Most people are frightened and run outdoors. Many find it difficult to stand. The vibration is noticed by persons driving motor cars. Large bells ring.

ii) In many buildings of Type C damage of Grade 1 is caused; in many buildings of Type B damage is of Grade 2. Most buildings of Type A suffer damage of Grade 3, few of Grade 4. In single instances, landslides of roadway on steep slopes; crack in roads; seams of pipelines damaged; cracks in stone walls.

iii) Waves are formed on water, and is made turbid by mud stirred up. Water levels in wells change, and the flow of springs changes. Some times dry springs have their flow resorted and existing springs stop flowing. In isolated instances parts of sand and gravelly banks slip off.

8. *Destruction of buildings*

i) Fright and panic; also persons driving motor cars are disturbed. Here and there branches of trees break off. Even heavy furniture moves and partly overturns. Hanging lamps are damaged in part.

ii) Most buildings of Type C suffer damage of Grade 2, and few of Grade 3. Most buildings of Type B suffer damage of Grade 3. Most buildings of Type A suffer damage of Grade 4. Occasional breaking of pipe seams. Memorials and monuments move and twist. Tombstones overturn. Stone walls collapse.

iii) Small landslips in hollows and on banked roads on steep slopes; cracks in ground upto widths of several centimetres. Water in lakes become turbid. New reservoirs come into existence. Dry wells refill and existing wells become dry. In many cases, change in flow and level of water is observed.

9. *General damage of buildings*

i) General panic; considerable damage to furniture. Animals run to and fro in confusion, and cry.

ii) Many buildings of Type C suffer damage of Grade 3, and a few of Grade 4. Many buildings of Type B show a damage of Grade 4 and a few of Grade 5. Many buildings of Type A suffer damage of Grade 5. Monuments and columns fall. Considerable damage to reservoirs; underground pipes partly broken. In individual cases, railway lines are bent and roadway damaged.

iii) On flat land overflow of water, sand and mud is often observed. Ground cracks to widths of up to 10 cm, on slopes and river banks more than 10 cm. Further more, a large number of slight cracks in ground; falls of rock, many land slides and earth flows; large waves in water. Dry wells renew their flow and existing wells dry up.

10. *General destruction of buildings*

i) Many buildings of Type C suffer damage of Grade 4, and a few of Grade 5. Many buildings of Type B show damage of Grade 5. Most of Type A have destruction of Grade 5. Critical damage to dykes and dams. Severe damage to bridges. Railway lines are bent slightly. Underground pipes are bent or broken. Road paving and asphalt show waves.

ii) In ground, cracks up to widths of several centimetres, sometimes up to 1 m. Parallel to water courses occur broad fissures. Loose ground slides from steep slopes. From river banks and steep coasts, considerable landslides are possible. In coastal areas, displacement of sand and mud; change of water level in wells; water from canals, lakes, rivers, etc, thrown on land. New lakes occur.

11. *Destruction*

i) Severe damage even to well built buildings, bridges, water dams and railway lines. Highways become useless. Underground pipes destroyed.

ii) Ground considerably distorted by broad cracks and fissures, as well as movement in horizontal and vertical directions. Numerous landslips and falls of rocks. The intensity of the earthquake requires to be investigated specifically.

12. *Landscape changes*

i) Practically all structures above and below ground are greatly damaged or destroyed.

ii) The surface of the ground is radically changed. Considerable ground cracks with extensive vertical and horizontal movements are observed. Falling of rock and slumping of river banks over wide areas, lakes are dammed; waterfalls appear and rivers are deflected. The intensity of the earthquake requires to be investigated specially.

ANNEX E

(Foreword)

ZONE FACTORS FOR SOME IMPORTANT TOWNS

Town	*Zone*	*Zone Factor, Z*	*Town*	*Zone*	*Zone Factor, Z*
Agra	III	0.16	Chitradurga	II	0.10
Ahmedabad	III	0.16	Coimbatore	III	0.16
Ajmer	II	0.10	Cuddalore	III	0.16
Allahabad	II	0.10	Cuttack	III	0.16
Almora	IV	0.24	Darbhanga	V	0.36
Ambala	IV	0.24	Darjeeling	IV	0.24
Amritsar	IV	0.24	Dharwad	III	0.16
Asansol	III	0.16	Dehra Dun	IV	0.24
Aurangabad	II	0.10	Dharampuri	III	0.16
Bahraich	IV	0.24	Delhi	IV	0.24
Bangalore	II	0.10	Durgapur	III	0.16
Barauni	IV	0.24	Gangtok	IV	0.24
Bareilly	III	0.16	Guwahati	V	0.36
Belgaum	III	0.16	Goa	III	0.16
Bhatinda	III	0.16	Gulbarga	II	0.10
Bhilai	II	0.10	Gaya	III	0.16
Bhopal	II	0.10	Gorakhpur	IV	0.24
Bhubaneswar	III	0.16	Hyderabad	II	0.10
Bhuj	V	0.36	Imphal	V	0.36
Bijapur	III	0.16	Jabalpur	III	0.16
Bikaner	III	0.16	Jaipur	II	0.10
Bokaro	III	0.16	Jamshedpur	II	0.10
Bulandshahr	IV	0.24	Jhansi	II	0.10
Burdwan	III	0.16	Jodhpur	II	0.10
Cailcut	III	0.16	Jorhat	V	0.36
Chandigarh	IV	0.24	Kakrapara	III	0.16
Chennai	III	0.16	Kalapakkam	III	0.16

Town	Zone	Zone Factor, Z	Town	Zone	Zone Factor, Z
Kanchipuram	III	0.16	Pondicherry	II	0.10
Kanpur	III	0.16	Pune	III	0.16
Karwar	III	0.16	Raipur	II	0.10
Kohima	V	0.36	Rajkot	III	0.16
Kolkata	III	0.16	Ranchi	II	0.10
Kota	II	0.10	Roorkee	IV	0.24
Kurnool	II	0.10	Rourkela	II	0.10
Lucknow	III	0.16	Sadiya	V	0.36
Ludhiana	IV	0.24	Salem	III	0.16
Madurai	II	0.10	Simla	IV	0.24
Mandi	V	0.36	Sironj	II	0.10
Mangalore	III	0.16	Solapur	III	0.16
Monghyr	IV	0.24	Srinagar	V	0.36
Moradabad	IV	0.24	Surat	III	0.16
Mumbai	III	0.16	Tarapur	III	0.16
Mysore	II	0.10	Tezpur	V	0.36
Nagpur	II	0.10	Thane	III	0.16
Nagarjunasagar	II	0.10	Thanjavur	II	0.10
Nainital	IV	0.24	Thiruvananthapuram	III	0.16
Nasik	III	0.16	Tiruchirappali	II	0.10
Nellore	III	0.16	Tiruvennamalai	III	0.16
Osmanabad	III	0.16	Udaipur	II	0.10
Panjim	III	0.16	Vadodara	III	0.16
Patiala	III	0.16	Varanasi	III	0.16
Patna	IV	0.24	Vellore	III	0.16
Pilibhit	IV	0.24	Vijayawada	III	0.16
			Vishakhapatnam	II	0.10

ANNEX F

(*Foreword*)

COMMITTEE COMPOSITION

Earthquake Engineering Sectional Committee, CED 39

Organization	*Representative(s)*
In personal capcity (*72/6 Civil Lines, Roorkee 247667*)	DR A. S. ARYA (*Chairman*)
Bharat Heavy Electrical Ltd, New Delhi	SHRI N. C. ADDY DR C. KAMESHWARA RAO (*Alternate* I) SHRI A. K. SINGH (*Alternate* II)
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Central Public Works Department, New Delhi	SUPERINTENDING ENGINEER (D) EXECUTIVE ENGINEER (D) III (*Alternate*)
Central Water Commission (ERDD), New Delhi	DIRECTOR CMDD (N&W) DIRECTOR EMBANKMENT (N&W) (*Alternate*)
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Indian Institute of Technology, Kanpur	DR S. K. JAIN DR C. V. R. MURTY (*Alternate*)
Indian Institute of Technology, Mumbai	DR RAVI SINHA DR A. GOYAL (*Alternate*)
Indian Meterological Department, New Delhi	DR S. N. BHATTACHARYA SHRI V. K. MITTAL (*Alternate*)

(*Continued on page* 38)

(Continued from page 37)

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Member-Secretary
SHRI S. CHATURVEDI
Joint Director (Civ Engg), BIS

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(Continued on page 39)

(Continued from page 38)

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AMENDMENT NO. 1 JANUARY 2005
TO
IS 1893 (PART 1) : 2002 CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 1 GENERAL PROVISIONS AND BUILDINGS

(Fifth Revision)

(*Page* 5, *Fig.* 1) — Interchange 'VARANASI' and 'ALLAHABAD' and 'KOLKATA' to be in Zone III.

(*Page* 15, *under Note* 4, *Table* 1) — For Zone II, substitute the following for the existing:

II (for important structures only)	≤5 ≥10	10 20

(*Page* 24, *clause* **7.6.2**) — Substitute the following for the existing expression:

$$T_a = \frac{0.09\,h}{\sqrt{d}}$$

(*Page* 25, *clause* **7.8.4.4**) — Substitute the following for the existing expression:

$$P_{ij} = \frac{8\,\varsigma^2\,(1+\beta)\,\beta^{1.5}}{(1-\beta^2)^2 + 4\,\varsigma^2\,\beta\,(1+\beta)^2}$$

(*Page* 26, *clause* **7.9.1**) — Delete last sentence 'However .... neglected'.

(*Page* 26, *clause* **7.9.2**) — Renumber 'NOTE' as 'NOTE 1' and add the following Note 2 after Note 1:

'NOTE 2 — In case 3D dynamic analysis is carried out, the dynamic amplification factor of 1.5 be replaced with 1.0.'

(*Page* 35, *Annex* E) — Substitute the following for the existing:

' Cuddalore II 0.24'

(CED 39)

Reprography Unit, BIS, New Delhi, India

IS 4326 : 2013

भारतीय मानक

भवनों की भूकम्प प्रतिरोधी डिजाइन और संरचना — रीति संहिता

(तीसरा पुनरीक्षण)

Indian Standard

EARTHQUAKE RESISTANT DESIGN AND CONSTRUCTION OF BUILDINGS — CODE OF PRACTICE

(Third Revision)

ICS 91.120.25

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

August 2013

Price Group 11

Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard (Third Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

Himalayan-Naga Lushai region, Indo-Gangetic Plain, Western India and Kutch and Kathiawar regions are geologically unstable parts of the country and some devastating earthquakes of the world have occurred there. A major part of the peninsular India has also been visited by moderate earthquakes, but these were relatively few in number and had considerably lesser intensity. It has been a long felt need to rationalize the earthquake resistant design and construction of structures taking into account seismic data from studies of these Indian earthquakes, particularly in view of the heavy construction programme at present all over the country. It is to serve this purpose that IS 1893 : 1966 'Criteria for earthquake resistant design of structures' was formulated. It covered the seismic design considerations for various structures. As an adjunct to IS 1893 : 1966, IS 4326 : 1967 'Code of practice for earthquake resistant design and construction of buildings' was formulated and subsequently revised in 1976 to be in line with IS 1893 : 1975. Since 1984 revision of IS 1893 was minor, it did not require a revision of IS 4326. An expansion of IS 4326 was in fact thought of immediately after the Bihar earthquake of August 1988 when greater attention was needed on low-strength brickwork and stone masonry as well as earthen buildings; also repair, restoration and strengthening of earthquake damaged buildings posed a serious issue. After intense deliberations, the Committee decided to issue separate standards to cover these topics. It was further decided to cover detailing of reinforced concrete for achieving ductility in a separate standard to be used with IS 456 : 1978 'Code of practice for plain and reinforced concrete (*third revision*)'. Hence IS 4326 was third revised in 1993.

IS 1893 (Part 1) has been revised in 2002 with a view to keep abreast with the rapid development and extensive research that has been carried out in the field of earthquake resistant design of various structures. IS 456 has been also revised in 2000. Further, four amendments have been issued to IS 4326 : 1993. Therefore, it has been decided to take up the revision of IS 4326 : 1993.

In this standard, it is intended to cover the specified features of design and construction for earthquake resistance of buildings of conventional types. In case of other buildings, detailed analysis of earthquake forces shall be necessary. Recommendations regarding restrictions on openings, provision of steel in various horizontal bands and vertical steel at corners and junctions in walls and at jambs of openings are based on a range of calculations made using steel design seismic coefficient and the ductility of steel reinforcement. Many of the provisions have also been verified experimentally on models by shake table tests.

The Committee responsible for the formulation of this standard has taken into consideration the views of all who are interested in this field and has related the standard to the prevailing practices in the country. Due weightage has also been given to the need for international co-ordination among the standards and practices prevailing in different countries of the world.

The composition of the Committee responsible for the formulation of this standard is given in Annex A.

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 '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.

Indian Standard

EARTHQUAKE RESISTANT DESIGN AND CONSTRUCTION OF BUILDINGS — CODE OF PRACTICE

(*Third Revision*)

1 SCOPE

1.1 This standard deals with the selection of materials, special features of design and construction for earthquake resistant buildings including masonry construction using rectangular masonry units, timber construction and buildings with pre-fabricated flooring/roofing elements.

1.2 Guidelines for earthquake resistant buildings constructed using masonry of low strength and earthen buildings are covered in separate Indian Standards.

2 REFERENCES

The standards listed below 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 edition of the standards indicated below:

IS No.	*Title*
456 : 2000	Plain and reinforced concrete — Code of practice (*third revision*)
883 : 1994	Code of practice for design of structural timber in building (*fourth revision*)
1077 : 1992	Common burnt clay building bricks — Specification (*fifth revision*)
1597 (Part 2) : 1992	Code of practice for construction of stone masonry: Part 2 Ashlar masonry (*first revision*)
1641 : 1988	Code of practice for fire safety of buildings (general): General principles of fire grading and classification (*first revision*)
1642 : 1989	Code of practice for fire safety of buildings (general): Details of construction (*first revision*)
1643 : 1988	Code of practice for fire safety of buildings (general): Exposure hazard (*first revision*)
1644 : 1988	Code of practice for fire safety of buildings (general): Exit requirements and personal hazard (*first revision*)
1646 : 1997	Code of practice for fire safety of buildings (general): Electrical installations (*second revision*)
1893 : 1984	Criteria for earthquake resistant design of structures (*fourth revision*)
1893 (Part 1) : 2002	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)
1904 : 1986	Code of practice for design and construction of foundations in soils: General requirements (*third revision*)
1905 : 1987	Code of practice for structural use of unreinforced masonry (*third revision*)
2185 (Part 1) : 2005	Concrete masonry units — Specification: Part 1 Hollow and solid concrete blocks (*third revision*)
2212 : 1991	Code of practice for brickwork (*first revision*)
2751 : 1979	Code of practice for welding mild steel plain and deformed bars for reinforced construction (*first revision*)
3414 : 1968	Code of practice for design and installation of joints in buildings
9417 : 1989	Recommendations for welding cold worked bars for reinforced steel construction (*first revision*)
13920 : 1993	Ductility detailing of reinforced concrete structures subjected to seismic forces — Code of practice

3 TERMINOLOGY

For the purpose of this standard, the following definitions shall apply.

3.1 Separation Section — A gap of specified width between adjacent buildings or parts of the same building either left uncovered or covered suitably to permit movement in order to avoid pounding due to earthquake.

3.1.1 *Crumple Section* — The separation gap filled with appropriate material that crumples or factures in the event of an earthquake.

3.2 Centre of Rigidity — The point in a structure where application of lateral force produces equal deflections of its components at any level in any particular direction.

3.3 Shear Wall — A wall designed to resist lateral force in the own plane. Braced frames, subjected primarily to axial stresses, shall be considered as shear walls for the purpose of this definition.

3.4 Space Frame — A three-dimensional structural system comprised of interconnected members, without shear or bearing walls, so that to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

3.4.1 *Vertical Load Carrying Frame* — A space frame designed to carry all the vertical loads, the horizontal loads being resisted by shear walls.

3.4.2 *Moment Resistant Frame* — A space frame capable of carrying all vertical and horizontal loads, by developing bending moments in the members and at joints.

3.4.3 *Moment Resistant Frame with Shear Walls* — A space frame with moment resistant joints and strengthened by shear walls to assist in carrying horizontal loads.

3.5 Box System — A bearing wall structure without a space frame, the horizontal forces being resisted by the walls that act as shear walls.

3.6 Band — A reinforced concrete or reinforced brick runner provided in the walls to tie them together and to impart horizontal bending strength in them.

3.7 Seismic Zone and Seismic Coefficient — The seismic zones II to IV as classified in IS 1893 (Part 1) and corresponding basic seismic coefficient α_0 as specified in 3.4 of IS 1893.

3.8 Design Horizontal Seismic Coefficient — The value of horizontal seismic coefficient A_h computed taking into account the soil-foundation system and the importance factor as specified in 6.4 of IS 1893 (Part 1).

3.9 Concrete Grades — 28 day compressive strength of concrete cubes of 150 mm size, in MPa; for example, for Grade M20 of IS 456, the concrete strength equal to 20 MPa.

4 GENERAL PRINCIPLES

4.0 The general principles given in **4.1** to **4.9** shall be observed in construction of earthquake resistant buildings.

4.1 Lightness

Since the earthquake force is a function of mass, the weight of the building shall be as minimum as possible, consistent with structural safety and functional requirements. Roofs and upper storeys of buildings, in particular, should be designed as light as possible.

4.2 Continuity of Construction

4.2.1 As far as possible, the parts of the building should be tied together in such a manner that the building acts as one unit.

4.2.2 For parts of buildings between separation or crumple sections to expansion joints, floor slabs shall be continuous throughout as far as possible. Concrete slabs shall be rigidly connected or integrally cast with the support beams.

4.2.3 Additions to the structures shall be accompanied by the provision of separation or crumple sections between the new and the existing structures as far as possible, unless positive measures are taken to establish continuity between the existing and the new construction.

4.2.4 Alteration to the building structure shall be done by maintaining its structural stability by ensuring proper load path.

4.3 Projecting and Suspended Parts

4.3.1 Projecting parts shall be avoided as far as possible. If the projecting parts cannot be avoided, they shall be properly reinforced and firmly tied to the main structure, and their design shall be in accordance with IS 1893.

4.3.2 Ceiling plaster shall preferably be avoided. When it is unavoidable, the plaster shall be as thin as possible.

4.3.3 Suspended ceiling shall be avoided as far as possible. Where provided they shall be light, adequately framed and secured to which electrical fixtures shall be fully secured.

4.4 Building Configuration

4.4.1 In order to minimize torsion and stress concentration, provisions given in **4.4.2** to **4.4.4** should be complied with as relevant.

4.4.2 The building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity so that the centre of mass and rigidity of the building coincide with each other in which case no separation sections other than expansion joints are necessary. For provision of expansion joints reference may be made to IS 3414.

4.4.3 If symmetry of the structure is not possible in plan, elevation or mass, provision shall be made for torsional and other effects due to earthquake forces in the structural design or the parts of different rigidities may be separated through crumple sections. The length of such building between separation sections shall not preferably exceed three times the width.

NOTE — As an alternative to separation section to reduce torsional moments, the centre of rigidity of the building may be brought close or coincident to the centre of mass by adjusting the locations and/or sizes of columns and walls.

4.4.4 Buildings having plans with shapes like L, T, E and Y shall preferably be separated into rectangular parts by providing separation sections at appropriate places. Typical examples are shown in Fig. 1.

NOTES

1 The buildings with small lengths of projections forming L, T, E or Y shapes need not be provided with separation section. In each cases the length of the projection may not exceed 15 to 20 percent of the total dimension of the building in the direction of the projection (*see* Fig. 2).

2 For buildings with minor asymmetry in plan and elevation separation sections may be omitted.

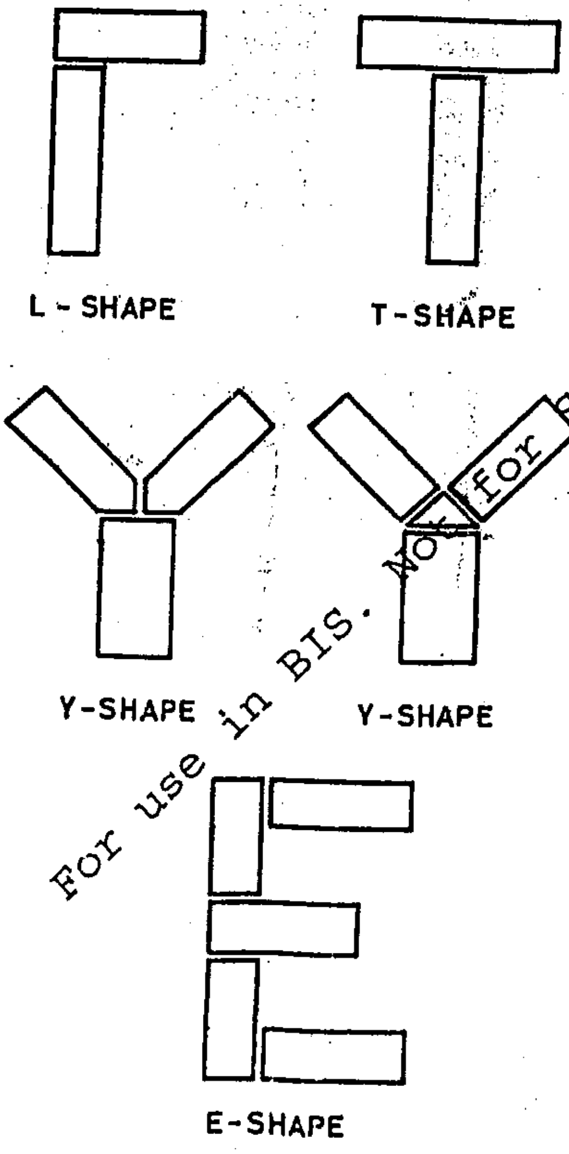


FIG. 1 TYPICAL SHAPES OF BUILDING WITH SEPARATION SECTIONS

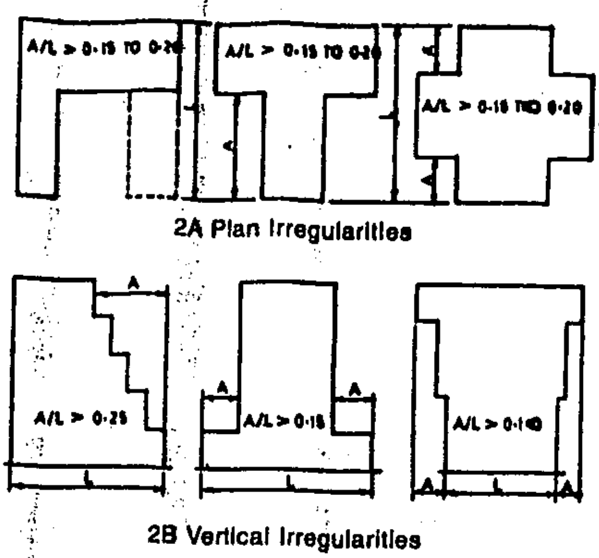


FIG. 2 PLAN AND VERTICAL IRREGULARITIES

4.5 Strength in Various Directions

The structure shall be designed to have adequate strength against earthquake effects along both the horizontal axes. The design shall also be safe considering the reversible nature of earthquake forces.

4.6 Foundations

The structure shall not be founded on such soils which shall subside or liquefy during an earthquake, resulting in large differential settlements (*see also* 5.3.3).

4.7 Ductility

The main structural elements and their connection shall be designed to have a ductile failure. This shall enable the structure to absorb energy during earthquakes to avoid sudden collapse of the structure. Providing reinforcing steel in masonry at critical sections, as specified in this standard shall not only increase strength and stability but also ductility. The details for achieving ductility in reinforced concrete structures is given in IS 13920.

4.8 Damage to Non-structural Parts

Suitable details shall be worked out to connect the non-structural parts with the structural framing so that the deformation of the structural frame leads to minimum damage of the non-structural elements.

4.9 Fire Safety

Fire frequently follows an earthquake and therefore, buildings shall be constructed to make them fire resistant in accordance with the provisions of following Indian Standards for fire safety, as relevant:

IS 1641, IS 1642, IS 1643, IS 1644 and IS 1646.

5 SPECIAL CONSTRUCTION FEATURES

5.1 Separation of Adjoining Structures

5.1.1 Separation of adjoining structures or parts of the same structure is required for structures having different total heights or storey heights and different dynamic characteristics. This is to avoid collision during an earthquake.

5.1.2 Minimum width of separation gaps as mentioned in **5.1.1**, shall be as specified in Table 1. The design seismic coefficient to be used shall be in accordance with IS 1893 (Part 1).

Table 1 Gap Width for Adjoining Structures

Sl No.	Type of Construction	Gap Width/Number of Storey in mm for Design Seismic Coefficient $A_h = 0.12$ mm
(1)	(2)	(3)
i)	Box system or frames with shear walls	15.0
ii)	Moment resistant reinforced concrete frame	20.0
iii)	Moment resistant steel frame	30.0

NOTE — Minimum total gap shall be 25 mm. For any other value of A_h the gap width shall be determined proportionately.

5.1.2.1 For buildings of height greater than 40 m, it shall be desirable to carry out model or dynamic analysis of the structures in order to compute the drift at each storey, and the gap width between the adjoining structures shall not be less than the sum of their dynamic deflection at any level.

5.1.3 Where separation is necessary, a complete separation of the parts shall be made except below the plinth level. The plinth beams, foundation beams and footings may be continuous. Where separation sections are provided in a long building, they shall take care of movement owing to temperature changes also.

5.2 Separation or Crumple Section

5.2.1 In case of framed construction, members shall be duplicated on either side of the separation or crumple section. As an alternative, in certain cases, such duplication may not be provided, if the portions on either side can act as cantilevers to take the weight of the building and other relevant loads.

5.2.2 Typical details of separation and crumple sections are shown in Fig. 3. For other types of joint details, reference may be made to IS 3414.

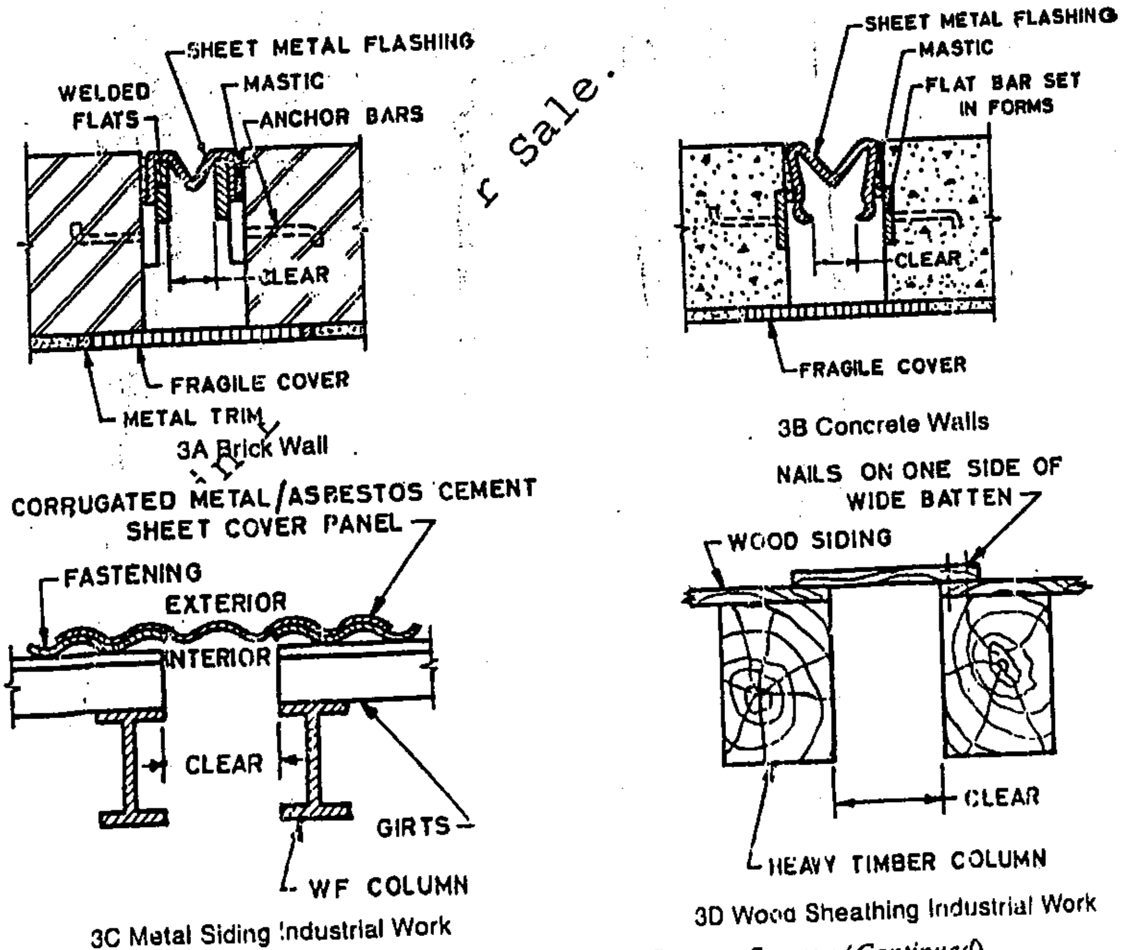
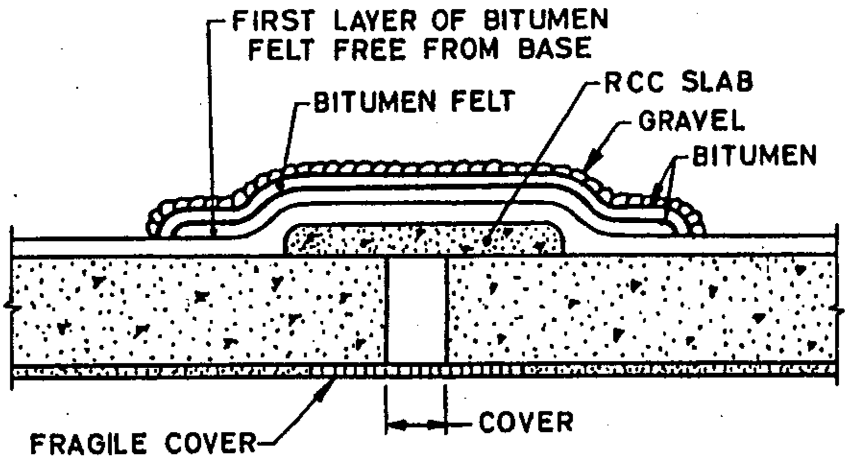
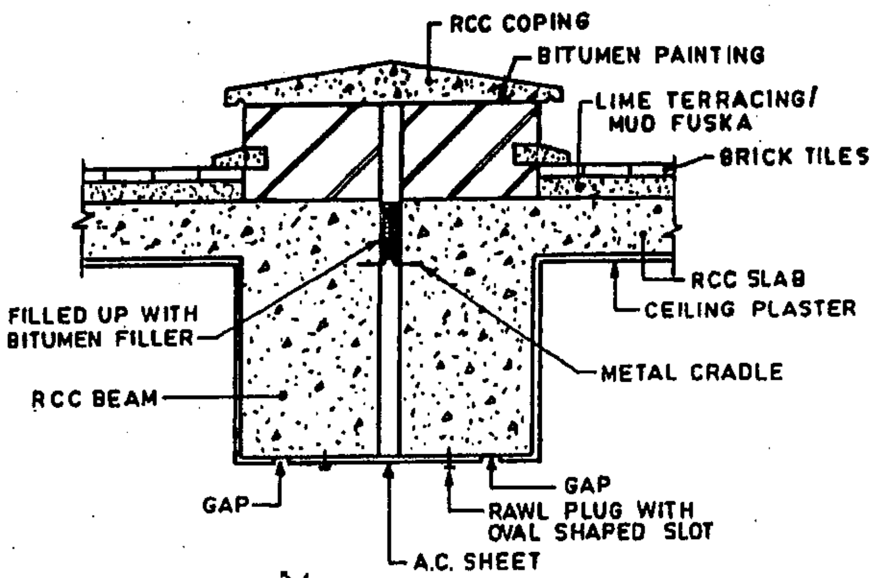


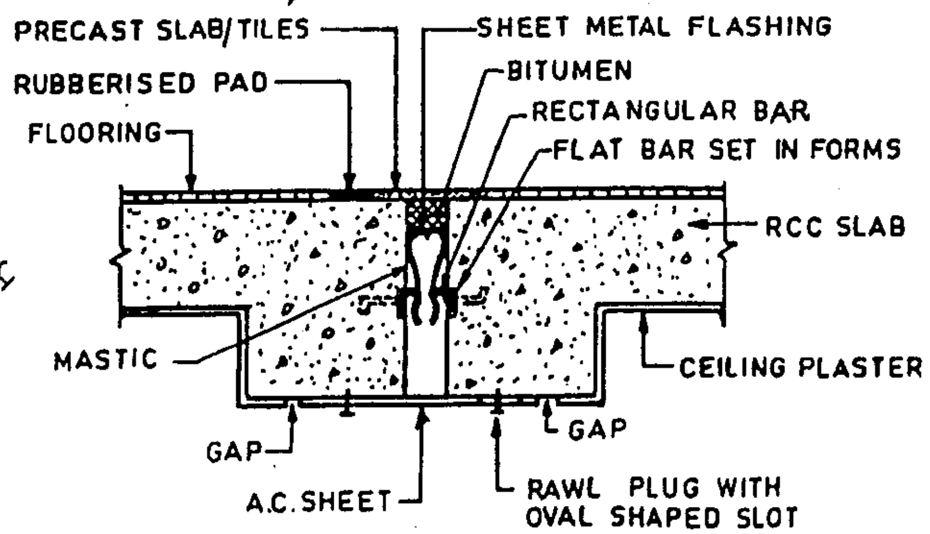
FIG. 3 TYPICAL DETAILS OF SEPARATION OR CRUMPLE SECTION (*Continued*)



3E RCC Slab on Roof Surface



3F Separation Joint Details at Roof



3G Separation at Floor Level

NOTE — Fragile cover may consist of asbestos cement sheet, particle board and like.

FIG. 3 TYPICAL DETAILS OF SEPARATION OR CRUMPLE SECTION

5.3 Foundations

5.3.1 For the design of foundations, the provisions of IS 1904 in conjunction with IS 1893 (Part 1) shall generally be followed.

5.3.2 The subgrade below the entire area of the building shall preferably be of the same type of the soil. Wherever this is not possible, a suitably located separation or crumple section shall be provided.

5.3.3 Loose fine sand, soft silt and expansive clays should be avoided. If unavoidable, the building shall rest either on a rigid raft foundation or on piles taken to a firm stratum. However, for light constructions the following measures may be taken to improve the soil on which the foundation of the building may rest:

a) Sand piling; and
b) Soil stabilization.

5.3.4 *Isolated Footings for Columns*

All the individual footings or pile caps where used in Type III soft soils [*see* Table 1 of IS 1893 (Part 1)], shall be connected by reinforced concrete ties at least in two directions approximately at right angles to each other. For buildings with no basement, the ties may be placed at or below the plinth level and for buildings with basement they may be placed at the level of basement floor. They shall need to be designed to carry the load of the panel walls also.

NOTE — The ties may not be necessary where structural floor connects the columns at or below the plinth level.

5.3.4.1 Where ties are used, their sections shall be designed to carry in tension as well as in compression an axial load not less than the earthquake force in the direction the tie acting on the heavier of the columns connected, but the sections shall not be less than 200 mm × 200 mm with M20 concrete reinforced with 4 bars of 12 mm diameter plain mild steel bars or 10 mm diameter high strength deformed bars, one at each corner, bound by 8 mm diameter stirrups not more than 150 mm apart.

NOTE — In working out the buckling strength of ties, the lateral support provided by the soil may be taken into account. Calculations show that for such buried ties, lateral buckling is not a problem and the full section of the tie may be taken effective as a short column.

5.3.4.2 In the case of reinforced concrete slab, the thickness shall not be less than 1/50th of the clear distance between the footings, but not less than 100 mm in any case. It shall be reinforced with not less than 0.15 percent mild steel bars or 0.12 percent high strength deformed bars in each direction placed symmetrically at top and bottom.

5.4 Roofs and Floors

5.4.1 Flat roof or floor shall not preferably be made of terrace of ordinary bricks supported on steel, timber or reinforced concrete joists, nor they shall be of a type which in the event of an earthquake is likely to be loosened and parts of all of which may fall. If this type of construction cannot be avoided, the joists should be blocked at ends and bridged at intervals such that their spacing is not altered during an earthquake.

5.4.1.1 For pitched roofs, corrugated iron or asbestos sheets shall be used in preference to country, Allahabad or Mangalore tiles or other loose roofing units. All roofing materials shall be properly tied to the supporting members. Heavy roofing materials shall generally be avoided.

5.4.2 *Pent Roofs*

5.4.2.1 All roof trusses shall be supported on reinforced concrete or reinforced brick band (*see* 8.4.3). The holding down bolts shall have adequate size and length as required for earthquake forces in accordance with IS 1893 (Part 1).

Where a trussed roof adjoins a masonry gable, the ends of the purlins shall be carried on and secured to a plate or bearer which shall be adequately bolted to reinforced concrete or reinforced brick band at the top of gable end masonry (*see* 8.4.4).

NOTE — Hipped roof in general have shown better structural behaviour during earthquakes than gable ended roofs.

5.4.2.2 At tie level all the trusses and the gable end shall be provided with diagonal braces in plan so as to transmit the lateral shear due to earthquake force to the gable walls acting as shear walls at the ends as specified in 8.4.

5.4.3 *Jack Arches*

Jack arched roofs or floors, where used shall be provided with mild steel ties in all spans along with diagonal braces in plan to ensure diaphragms actions.

5.5 Staircases

5.5.1 The inter-connection of the stairs with the adjacent floors should be appropriately treated by providing sliding joints at the stairs to eliminate their bracing effect on the floors. Large stair halls shall preferably be separated from the rest of the building by means of separation or crumple sections.

5.5.2 Three types of stair construction may be adopted as described below:

a) *Separated staircases* — One end of the staircase rests on a wall and the other end is carried by columns and beams which have no connection with the floors. The gap at the vertical joints between the floor and the staircase may be covered either with a tread plate attached to one side of the joint and sliding on the other side, or covered with some appropriate material which could crumple or fracture during an earthquake without causing structural damage. The supporting members, columns or walls, are isolated from the surrounding floors by means of separation or crumple sections. A typical example is shown in Fig. 4.

b) *Built-in staircase* — When stairs are built monolithically with floors, they can be protected against damage by providing rigid walls at the stair opening. An arrangement, in which the staircase is enclosed by two walls, is given in Fig. 5. In such cases, the joints, as mentioned in respect of separated staircases, shall not be necessary.

The two walls mentioned above, enclosing the staircase, shall extend through the entire height of the stairs and to the building foundations.

c) *Staircases with sliding joints* — In case it is not possible to provide rigid walls around stair openings for built-in staircase or to adopt the separated staircases, the staircases shall have sliding joints so that they shall not act as diagonal bracing.

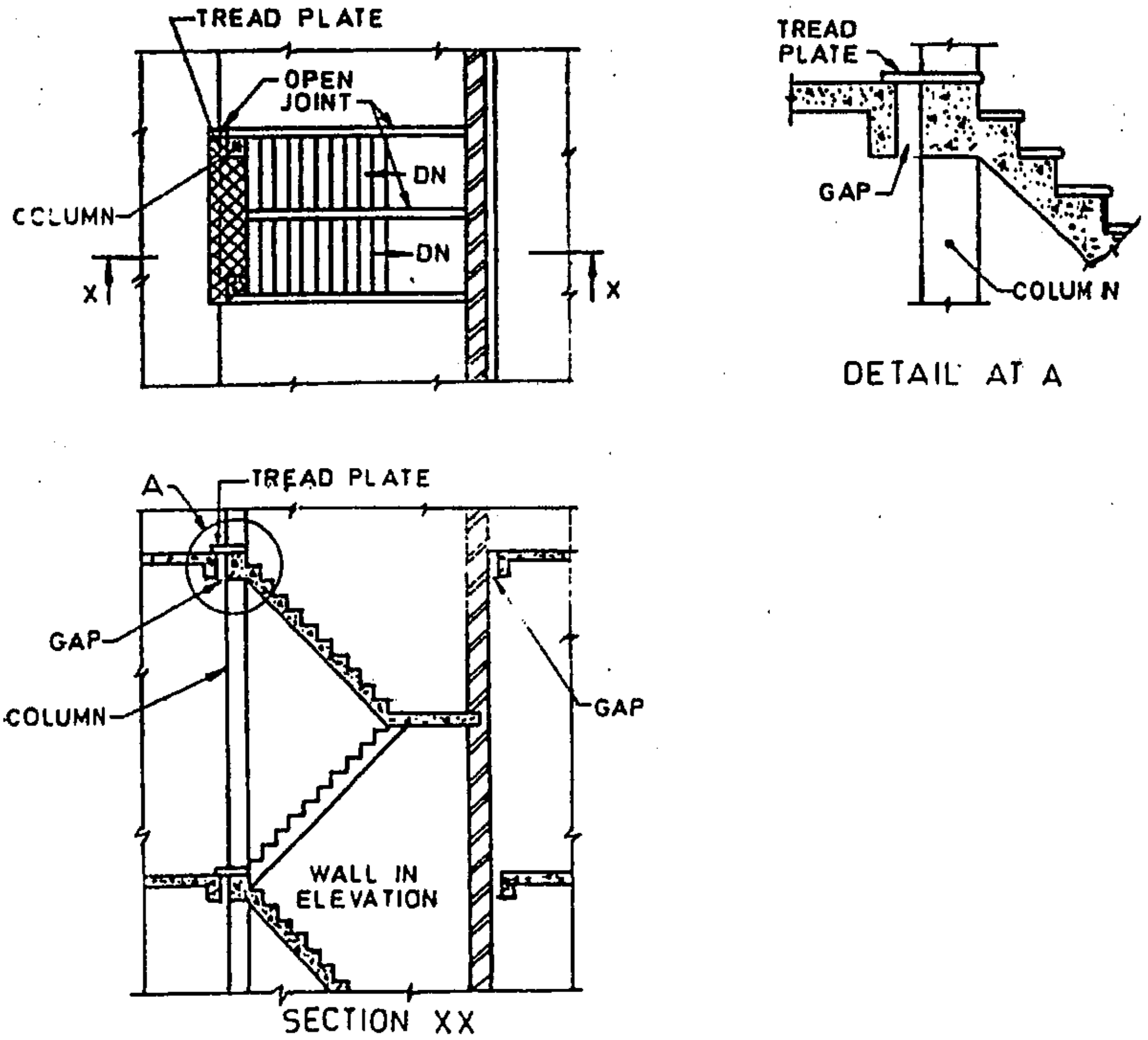
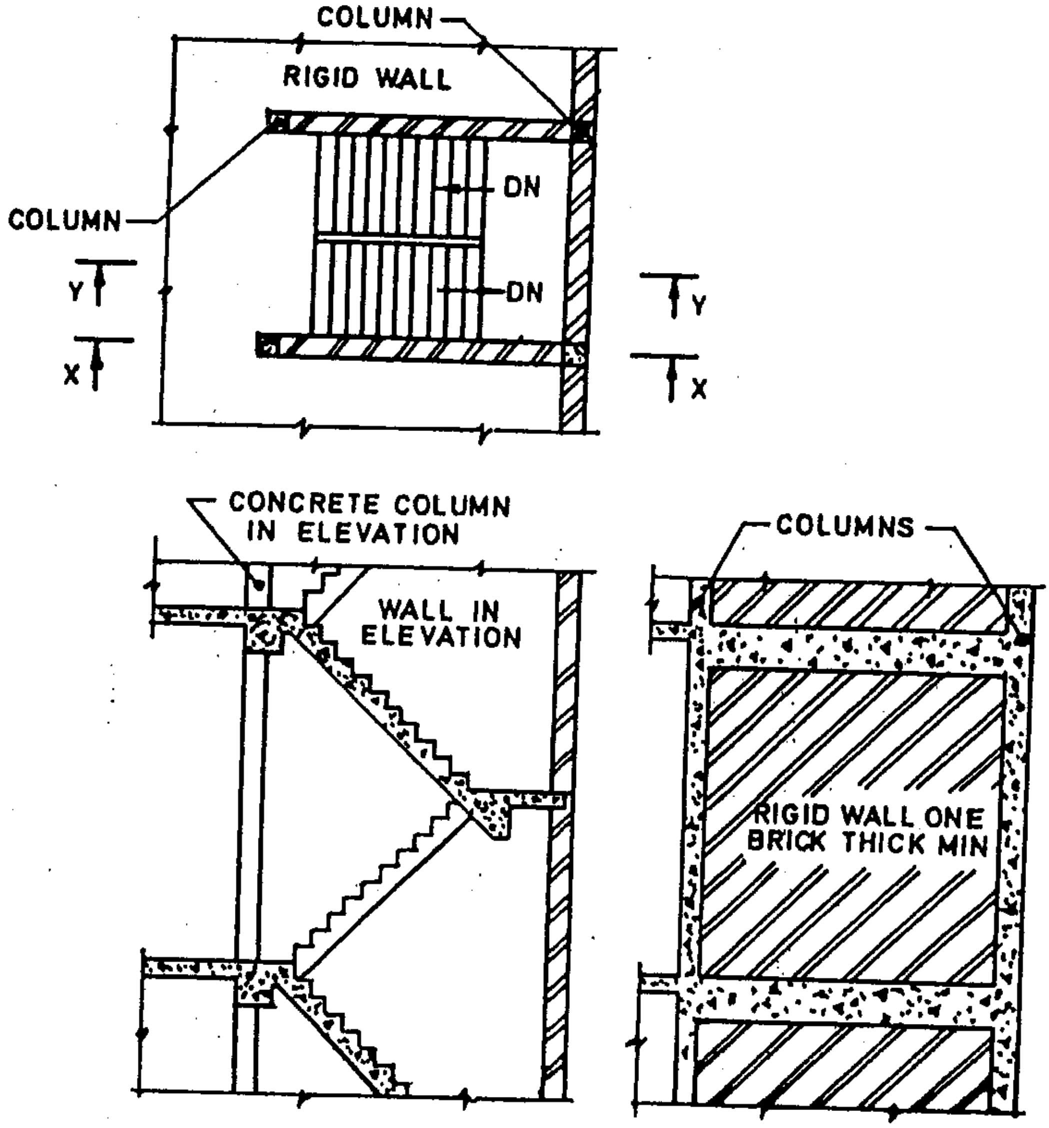


FIG. 4 SEPARATED STAIRCASE



SECTION YY SECTION XX

FIG. 5 RIGIDLY BUILD-IN STAIR CASE

6 TYPES OF CONSTRUCTION

6.1 The types of construction usually adopted in buildings are as follows:

a) Framed construction; and
b) Box type construction.

6.2 Framed Construction

This type of construction is suitable for multi-storied and industrial buildings as described in **6.2.1** and **6.2.2**.

6.2.1 *Vertical Load Carrying Frame Construction*

This type of construction consists of frames with flexible (hinged) joints and bracing members. Steel multi-storied building or industrial frames and timber construction usually are of this type.

6.2.1.1 Such buildings shall be adequately strengthened against lateral forces by shear walls and/or other bracing systems in plan, elevation and sections such that earthquake forces shall be resisted by them in any direction.

6.2.2 *Moment Resistant Frames with Shear Walls*

The frames may be of reinforced concrete or steel with semi-rigid or rigid joints. The walls are rigid capable of acting as shear walls and may be of reinforced concrete or of brickwork reinforced or unreinforced bounded by framing members through shear connectors.

6.2.2.1 The frame and wall combinations shall be designed to carry the total lateral force due to earthquake acting on the building. The frame acting alone shall be designed to resist at least 25 percent of the total lateral force.

6.2.2.2 The shear walls shall preferably be distributed evenly over the whole building. When concentrated at one location, forming what is called a rigid core in the building, the design shall be checked for torsional effects and the shear connection between the core and the floors conservatively designed for the total shear transfer.

6.2.2.3 The shear walls should extend from the foundation either to the top of the building or to a lesser height as required from design consideration. In design, the interaction between frame and the shear walls should be considered properly to satisfy compatibility and equilibrium conditions.

NOTE — Studies show that shear walls of height about 85 percent of total height of building are advantageous.

6.3 Box Type Construction

This type of construction consists of prefabricated or *in-situ* masonry, concrete or reinforced concrete walls along both the axes of the building. The walls support vertical loads and also act as shear walls for horizontal loads acting in any direction. All traditional masonry construction falls under this category. In prefabricated construction, attention shall be paid to the connection between wall panels so that transfer of shear between them is ensured.

7 CATEGORIES OF BUILDINGS

7.1 For the purpose of specifying the earthquake resisting features in masonry and wooden buildings, the buildings have been categorized in four categories B to E based on the seismic zone and the importance of the building *I*, where

I = Importance factor applicable to the building [*see* **6.4.2** and Table 6 of IS 1893 (Part 1)].

7.1.1 The building categories are given in Table 2.

Table 2 Building Categories for Earthquake Resisting Features

Sl No.	Importance Factor	Seismic Zone			
		II	III	IV	V
(1)	(2)	(3)	(4)	(5)	(6)
i)	1.0	B	C	D	E
ii)	1.5	C	D	E	E

NOTE — Category A is now defunct as zone I does not exist any more.

8 MASONRY CONSTRUCTION WITH RECTANGULAR MASONRY UNITS

8.1 The design and construction of masonry walls using rectangular masonry units in general shall be governed by IS 1905 and IS 2212.

8.1.1 *Masonry Units*

8.1.1.1 Well burnt bricks conforming to IS 1077 or solid concrete blocks conforming to IS 2185 (Part 1) and having a crushing strength not less than 3.5 MPa shall be used. The strength of masonry unit required shall depend upon number of storeys and thickness of walls (*see* IS 1905).

8.1.1.2 Squared stone masonry, stone block masonry or hollow concrete block masonry, as specified in IS 1597 (Part 2) of adequate strength, may also be used.

8.1.2 *Mortar*

8.1.2.1 Mortars, such as those given in Table 3 or of equivalent specification, shall preferably be used for masonry construction for various categories of buildings.

8.1.2.2 Where steel reinforcing bars are provided in masonry the bars shall be embedded with adequate cover in cement sand mortar not leaner than 1 : 3 (minimum clear cover 10 mm) or in cement concrete of grade M20 (minimum clear cover 15 mm or bar diameter, whichever more), so as to achieve good bond and corrosion resistance.

8.2 Walls

8.2.1 Masonry bearing walls built in mortar, as specified in **8.1.2.1** unless rationally designed as reinforced masonry shall not be built of greater height than 15 m subject to a maximum of four storeys when measured from the mean ground level to the roof slab or ridge level. The masonry bearing walls shall be reinforced in accordance with **8.4.1**.

8.2.2 The bearing walls in both directions shall be straight and symmetrical in plan as far as possible.

8.2.3 The wall panels formed between cross walls and floors or roof shall be checked for their strength in bending as a plate or as a vertical strip subjected to the earthquake force acting on its own mass.

NOTE — For panel walls of 200 mm or larger thickness having a storey height not more than 3.5 m and laterally supported at the top, this check need not be exercized.

8.2.4 *Masonry Bond*

For achieving full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course. To obtain full bond between perpendicular walls, it is necessary to make a slopping (stepped) joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise, the toothed joint should be made in both the walls alternatively in lifts of about 450 mm (*see* Fig. 6).

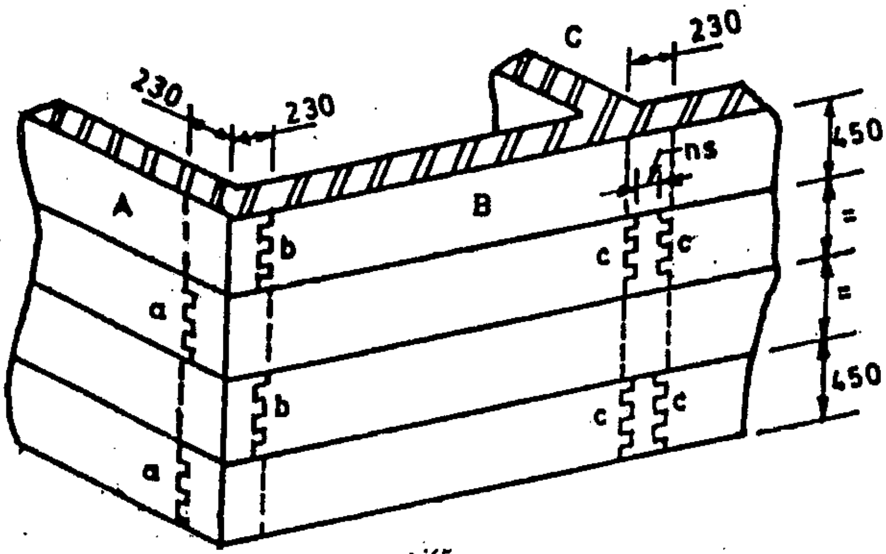
8.2.5 Ignoring tensile strength, free standing walls shall be checked against overturning under the action of design seismic coefficient $_h$ allowing for a factor safety of 1.5.

8.2.6 Panel or filler walls in framed buildings shall be properly bonded to surrounding framing members by means of suitable mortar (*see* Table 3) or connected through dowels. If the walls are so bonded they shall be checked according to **8.2.3** otherwise checked as in **8.2.5**.

8.3 Openings in Bearing Walls

8.3.1 Door and window openings in walls reduce their lateral load resistance and hence, should preferably be small and more centrally located. The guidelines on the size and position of opening are given in Table 4 and Fig. 7.

8.3.2 Openings in any storey shall preferably have their top at the same level so that a continuous band could be provided over them, including the lintels throughout the building.



a, *b*, *c* — toothed joints wall and *A*, *B*, *C*

All dimensions in millimetres.

FIG. 6 ALTERNATING TOOTHED JOINTS IN WALLS AT CORNER AND T-JUNCTION

Table 3 Recommended Mortar Mixes

(*Clauses* 8.1.2.1 *and* 8.2.6)

Sl No.	Building Category	Sl No. as given in Table [illegible] of IS 1905	Grade of Mortar	Mix Proportions (By Loose Volume) Cement	Lime	Sand	Minimum Compressive Strength at 28 Days N/mm²
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	E	2(a)	H2	1	¼ C or B	4	7.5
		2(b)	—	1	½ C or B	4½	6.0
ii)	D	3(a)	M1	1	—	5	5.0
		3(B)	—	1	1 C or B	6	3.0
iii)	C	4(a)	M2	1	—	6	3.0
		4(b)	—	1	2 B	9	2.0
iv)	B	5(a)	M3	1	—	7	1.5
		5(b)	—	1	3 B	12	1.5

NOTES

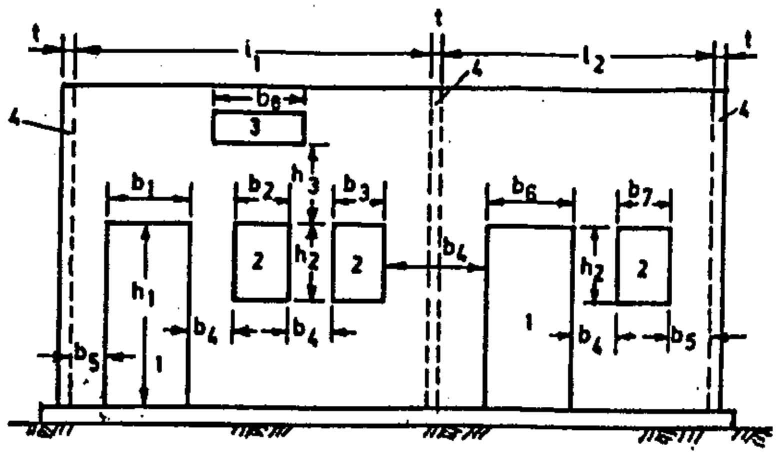
1 Sand for making mortar should be well graded. In case sand is not well graded, its proportion shall be reduced in order to achieve the minimum specified strength.

2 For mixes in Sl No. 2(a) and 2(b), use of lime is not essential from consideration of strength as it does not result in increase in strength. However, its use is highly recommended since it improves workability.

3 For mixes in Sl No. 3(a), 4(a) and 5(a), either lime C or B to the extent of 1/4 part of cement (by volume) or some plasticizer should be added for improving workability.

4 For mixes in Sl No. 4(b) and 5(b), lime and sand should first be ground in mortar mill and then cement added to coarse stuff.

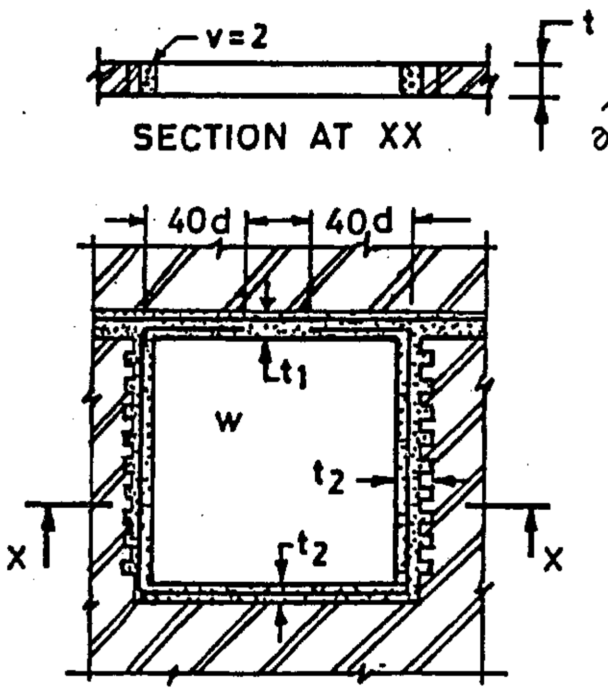
5 A, B and C denote eminently hydraulic lime, semi-hydraulic lime and fat lime respectively as specified in relevant Indian Standards.



1 — door　　3 — ventilator
2 — window　　4 — cross wall

FIG. 7 DIMENSIONS OF OPENINGS AND PIERS FOR RECOMMENDATIONS IN TABLE 4

8.3.3 Where openings do not comply with the guidelines of Table 4, they should be strengthened by providing reinforced concrete or reinforcing the brickwork, as shown in Fig. 8 with high strength deformed (H.S.D.) bars of 8 mm diameter but the quantity of steel shall be increased at the jambs to comply with 8.4.9, if so required.



w — window　　t — wall thickness
t_1 — lintel thickness　t_2 — thickness of concrete in jamb
v — vertical bar　　d — diameter of reinforcing bar

FIG. 8 STRENGTHENING MASONRY AROUND OPENINGS

8.3.4 If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored.

8.3.5 If an opening is tall from bottom to almost top of a storey, thus dividing the wall into two portions, these portions shall be reinforced with horizontal reinforcement of 6 mm diameter bars at not more than 450 mm intervals, one on inner and one on outer face, properly tied to vertical steel at jambs, corners or junction of walls, where used.

8.3.6 The use of arches to span over the openings is a source of weakness and shall be avoided. Otherwise, steel ties should be provided.

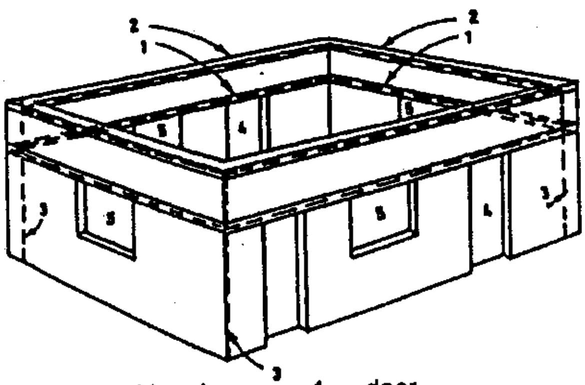
Table 4 Size and Position of Openings in Bearing Walls
(*Clause* 8.3.1)

Sl No.	Position of Opening	Details of Opening for Building Category		
		B	C	D and E
(1)	(2)	(3)	(4)	(5)
i)	Distance b_5 from the inside corner of outside wall, *Min*	0	230	450
ii)	For total length of openings, the ratio $(b_1 + b_2 + b_3)/l_1$ or $(b_6 + b_7)/l_2$ shall not exceed:			
	a) one-storeyed building	0.60	0.55	0.50
	b) two-storeyed building	0.50	0.46	0.42
	c) 3 or 4 storeyed building	0.42	0.37	0.33
iii)	Pier width between consecutive openings b_4, *Min*, in mm	340	450	560
iv)	Vertical distance between two openings one above the other h_3, *Min*, in mm	600	600	600
v)	Width of opening of ventilator b_8, *Max*, in mm	900	900	900

NOTE — Four storeys building not allowed in Category E.

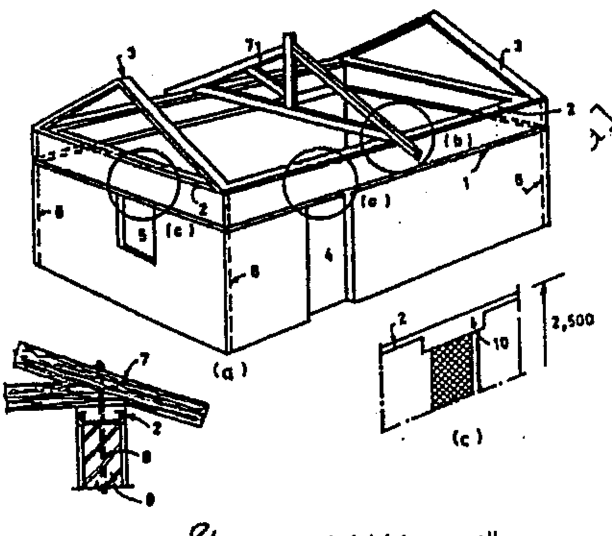
8.4 Seismic Strengthening Arrangements

8.4.1 All masonry buildings shall be strengthened by the methods, as specified for various categories of buildings, as listed in Table 5, and detailed in subsequent clauses. Figures 9 and 10 show, schematically, the overall strengthening arrangements to be adopted for category D and E buildings which consist of horizontal bands of reinforcement at critical levels, vertical reinforcing bars at corners, junctions of walls and jambs of openings.



1 — lintel band
2 — roof/floor band
3 — vertical bar
4 — door
5 — Window

FIG. 9 OVERALL ARRANGEMENT OF REINFORCING LOW STRENGTH MASONRY BUILDINGS



1 — lintel band
2 — eaves level (roof) band
3 — gable band
4 — door
5 — window
6 — vertical steel bar
7 — refter
8 — holding down bolt
9 — brick/stone wall
10 — door lintel integrated with roof band:
a) perspective view
b) details of truss connection with wall
c) detail of integrating door lintel with roof band

FIG. 10 OVERALL ARRANGEMENT OF REINFORCING LOW STRENGTH MASONRY BUILDING HAVING PITCHED ROOF

8.4.2 Lintel band is a band (*see* **3.6**) provided at lintel level on all load bearing internal, external longitudinal and cross walls. The specifications of the band are given in **8.4.5**.

NOTE — Lintel band, if provided in panel or partition walls also shall improve their stability during severe earthquake.

8.4.3 Roof band is a band (*see* **3.6**) provided immediately below the roof or floors. The specifications of the band are given in **8.4.5**. Such a band need not be provided underneath reinforced concrete or brick-work slabs resting on bearing walls, provided that the slabs are continuous over the intermediate wall up to the crumple sections, if any, and cover the width of end walls, fully or at least ¾ of the wall thickness.

Table 5 Strengthening Arrangements Recommended for Masonry Buildings (Rectangular Masonry Units)
(*Clause* 8.4.1)

Sl No. (1)	Building Category (2)	Number of Storeys (3)	Strengthening to be Provided in All Storeys (4)
i)	B	a) 1 to 3	a, b, c, f, g
		b) 4	a, b, c, d, f, g
ii)	C	a) 1 and 2	a, b, c, f, g
		b) 3 and 4	a to g
iii)	D	a) 1 and 2	a to g
		b) 3 and 4	a to h
iv)	E	1 to 3[1)]	a to h

where

a = masonry mortar (*see* **8.1.2**);
b = lintel band (*see* **8.4.2**);
c = roof band and gable band where necessary (*see* **8.4.3** and **8.4.4**);
d = vertical steel at corners and junctions of walls (*see* **8.4.8**);
e = vertical steel at jambs of openings (*see* **8.4.9**);
f = bracing in plan at tie level of roofs (*see* **5.4.2.2**);
g = plinth band where necessary (*see* **8.4.6**); and
h = dowel bars (*see* **8.4.7**).

NOTE — In case of four storey buildings of category B, the requirements of vertical steel may be checked through a seismic analysis using a design seismic co-efficient equal to four times the one given in IS 1893 (Part 1). (This is because the brittle behaviour of masonry in the absence of a vertical steel results in much higher effective seismic force than that envisaged in the seismic coefficient, provided in the code.) If this analysis shows that vertical steel is not required the designer may take the decision accordingly.

1) 4th storey not allowed in category E.

8.4.4 Gable band is a band provided at the top of gable masonry below the purlins. The specifications of the band are given in **8.4.5**. This band shall be made continuous with the roof band at the eaves level.

8.4.5 *Section and Reinforcement of Band*

The band shall be made of reinforced concrete of grade not leaner than M15 or reinforced brickwork in cement mortar not leaner than 1 : 3. The bands shall be of the full width of the wall not less than 75 mm in depth and reinforced with steel, as indicated in Table 6.

NOTE — In coastal areas, the concrete grade shall be M20 concrete and the filling mortar of 1 : 3 (cement-sand with water proofing admixture).

8.4.5.1 In case of reinforced brickwork, the thickness of joints containing steel bars shall be increased so as to have a minimum mortar cover of 10 mm around the bar. In bands of reinforced brickwork the area of steel provided should be equal to that specified above for reinforced concrete bands.

8.4.5.2 For full integrity of walls at corners and junctions of walls and effective horizontal bending resistance of bands continuity of reinforcement is essential. The details as shown in Fig. 11 are recommended.

8.4.6 Plinths band is a band provided at plinth level of walls on top of the foundation wall. This is to be provided where strip footings of masonry (other than reinforced concrete or reinforced masonry) are used and the soil is either soft or uneven in its properties, as frequently happens in hill tracts. Where used, its section may be kept same as in **8.4.5**. This band shall serve as damp proof course as well.

8.4.7 In category D and E buildings, to further enhance the box action of walls, steel dowel bars may be used at corners and T-junctions of walls at the sill level of windows to length of 900 mm from the inside corner in each wall. Such dowel may be in the form of U stirrups 8 mm diameter. Where used, such bars must be laid in 1 : 3 cement-sand-mortar with a minimum clear cover of 10 mm on all sides to minimize corrosion.

Table 6 Recommended Longitudinal Steel in Reinforced Concrete Bands

(*Clause* 8.4.5)

Sl No.	Span	Building Category B		Building Category C		Building Category D		Building Category E	
		No. of Bars	Dia	No. of Bars	Dia	No. of Bars	Dia	No. of Bars	Dia
	m		mm		mm		mm		mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	5 or less	2	8	2	8	2	8	2	10
ii)	6	2	8	2	8	2	10	2	12
iii)	7	2	8	2	10	2	12	4	10
iv)	8	2	10	2	12	4	10	4	12

NOTES

1 Span of wall shall be the distance between centre lines of its cross walls or buttresses. For spans greater than 8 m it shall be desirable to insert pillasters or buttresses to reduce the span or special calculation shall be made to determine the strength of wall and section of band.

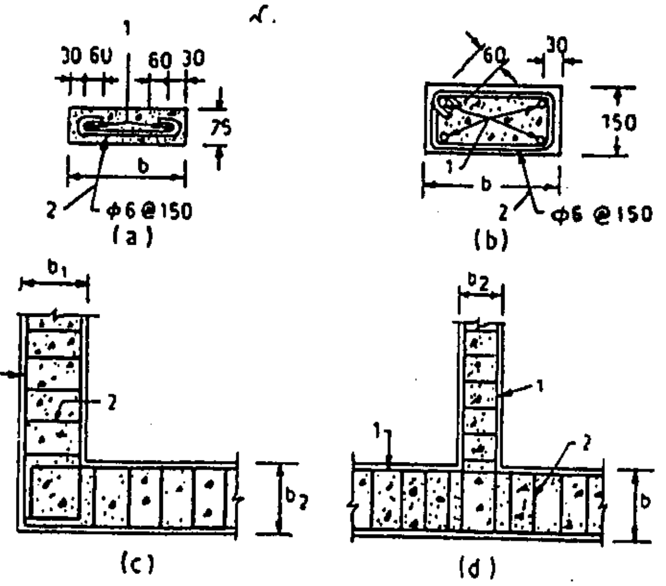
2 The number and diameter of bars given above pertain to high strength deformed bars.

3 Width of R.C. band is assumed same as the thickness of the wall. Wall thickness shall be 200 mm minimum. A clear cover of 20 mm from face of wall shall be maintained.

4 The vertical thickness of R.C. Band be kept 75 mm minimum, where two longitudinal bars are specified, one on each face; and 150 mm, where four bars are specified.

5 Concrete mix shall be of grade M 20 of IS 456 of 1 : 1½ : 3 by volume.

The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm diameter spaced at 150 mm apart.



1 — longitudinal bars
2 — lateral ties
b_1, b_2 — wall thickness

a) Section of band with two bars
b) Section of band with four bars
c) Structural plan at corner junction
d) Section plan at T-junction of walls

All dimensions in millimetres.

FIG. 11 REINFORCEMENT AND BENDING DETAIL IN R. C. BAND

8.4.8 *Vertical Reinforcement*

Vertical steel at corners and junctions of walls, which are up to 340 mm (1½ brick) thick, shall be provided as specified in Table 7. For walls thicker than 340 mm the area of the bars shall be proportionately increased. For earthquake resistant framed wall construction, *see* **8.5**.

Table 7 Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units

Sl No.	No. of Storeys	Storey	Diameter of HSD Single Barat Each Critical Section, mm			
			Category B	Category C	Category D	Category E
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	One	-	Nil	Nil	10	12
ii)	Two	a) Top	Nil	Nil	10	12
		b) Bottom	Nil	Nil	12	16
iii)	Three	a) Top	Nil	10	10	12
		b) Middle	Nil	10	12	16
		c) Bottom	Nil	12	12	16
iv)	Four	a) Top	10	10	10	Four storied building not permitted
		b) Third	10	10	12	
		c) Second	10	12	16	
		d) Bottom	12	12	20	

NOTES

1 The diameters given above are for H.S.D. bars.

2 The vertical bars shall be covered with concrete M 20 or mortar 1:3 grade in suitably created pockets around the bars (*see* Fig. 12). This shall ensure their safety from corrosion and good bond with masonry.

3 In case of floors/roofs with small Precast components, *see also* 9.2.3 for floor/roof band details.

8.4.8.1 The vertical reinforcement shall be properly embedded in the plinth masonry of foundations and roof slab or roof band so as to develop its tensile strength in bond. It shall be passing through the lintel bands and floor slabs or floor level bands in all storeys.

Bars in different storeys may be welded (*see* IS 2751 and IS 9417, as relevant) or suitably lapped.

NOTE — Typical details of providing vertical steel in brickwork masonry with rectangular solid units at corners and T-junctions are shown in Fig. 12.

8.4.9 Vertical reinforcement at jambs of window and door openings shall be provided as per Table 7. It may start from foundation of floor and terminate in lintel band (*see* Fig. 8).

8.5 Framing of Thin Load Bearing Walls (*see* **Fig. 13**)

Load bearing walls can be made thinner than 200 mm say 150 mm inclusive of plastering on both sides. Reinforced concrete framing columns and collar beams shall be necessary to be constructed to have full bond with the walls. Columns are to be located at all corners and junctions of walls and spaced not more than 1.5 m apart but so located as to frame up the doors and windows. The horizontal bands or ring beams are located at all floors roof as well as lintel levels of the openings. The sequence of construction between walls and columns shall be first to build the wall up to 4 to 6 courses height leaving toothed gaps (tooth projection being about 40 mm only) for the columns and second to pour M 20 (1 : 1½ : 3) concrete to fill the columns against the walls using wood forms only on two sides. The columns steel should be accurately held in position all along. The band concrete should be cast on the wall masonry directly so as to develop full bond with it.

Such construction may be limited to only two storeys maximum in view of its vertical load carrying capacity. The horizontal length of walls between cross walls shall be restricted to 7 m and the storey height to 3 m.

8.6 Reinforcing Details for Hollow Block Masonry

The following details may be followed in placing the horizontal and vertical steel in hollow block masonry using cement-sand or cement-concrete blocks.

8.6.1 *Horizontal Band*

U-shaped blocks may be used for construction of horizontal bends at various levels of the storeys as shown in Fig. 14, where the amount of horizontal reinforcement shall be taken 25 percent more than that given in Table 6 and provided by using four bars and 6 mm diameter stirrups. Other continuity details shall be followed, as shown in Fig. 11.

8.6.2 *Vertical Reinforcement*

Bars, as specified in Table 7 shall be located inside the cavities of the hollow blocks, one bar in each cavity (*see* Fig. 15). Where more than one bar is planned these can be located in two or three consecutive cavities. The cavities containing bars are to be filled by using micro-concrete 1 : 2 : 3 or cement coarse sand mortar 1 : 3, and properly rodded for compaction. The vertical bars should be spliced by welding or overlapping for developing full tensile strength. For proper bonding, the overlapped bars should be tied together by winding the binding wire over the lapped length. To reduce the number of overlaps, the blocks may be made U-shaped as shown in Fig. 15 which shall avoid lifting and threading of bars into the hollows.

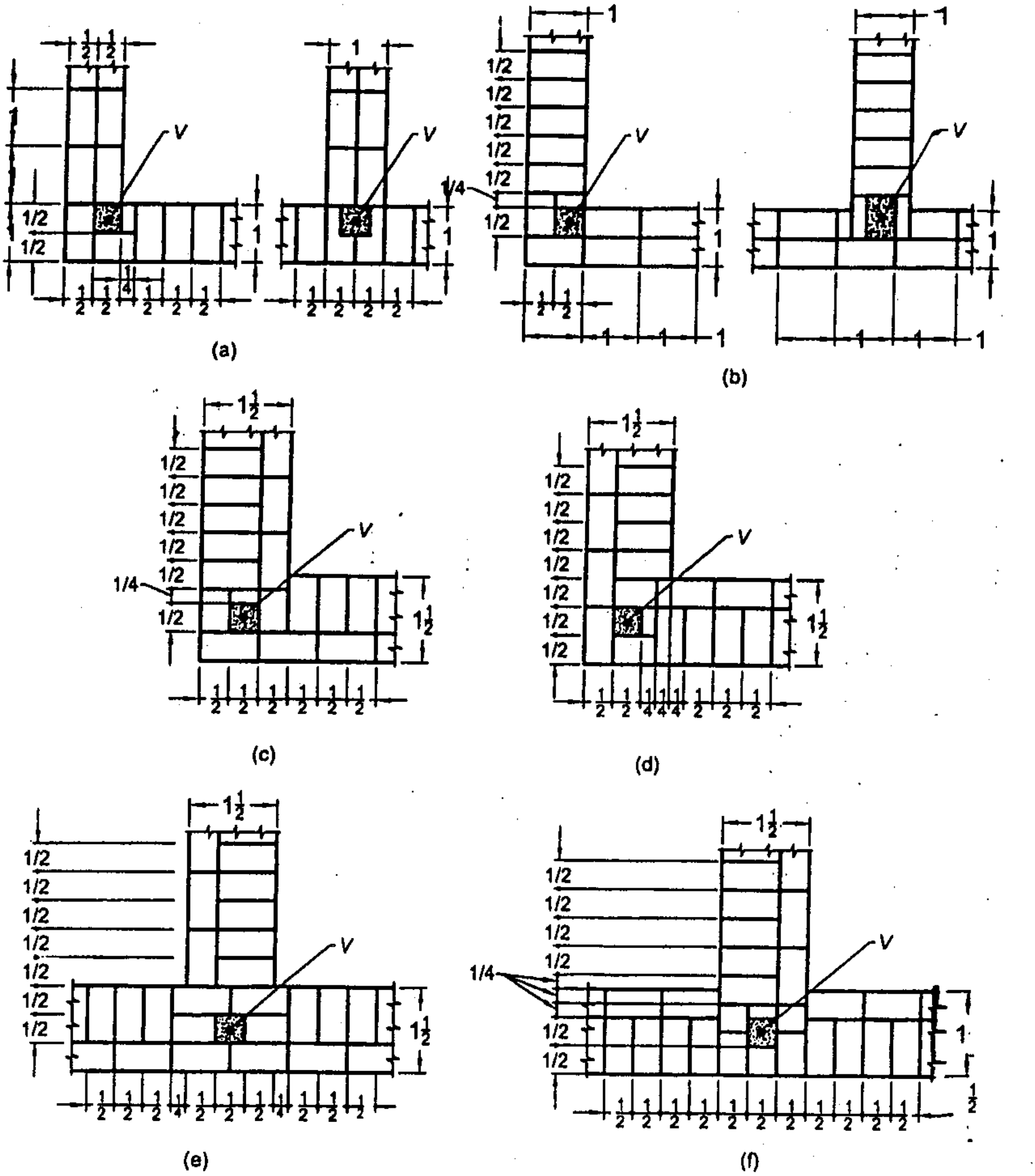
9 FLOORS/ROOFS WITH SMALL PRECAST COMPONENTS

9.1 Types of Precast Floors/Roofs

Earthquake resistance measures for floors and roofs with small precast components, as covered in this standard, have been dealt with as typical examples.

9.1.1 *Precast Reinforced Concrete Unit Roof/Floor*

The unit is a precast reinforced concrete component, channel (inverted trough) shaped in section



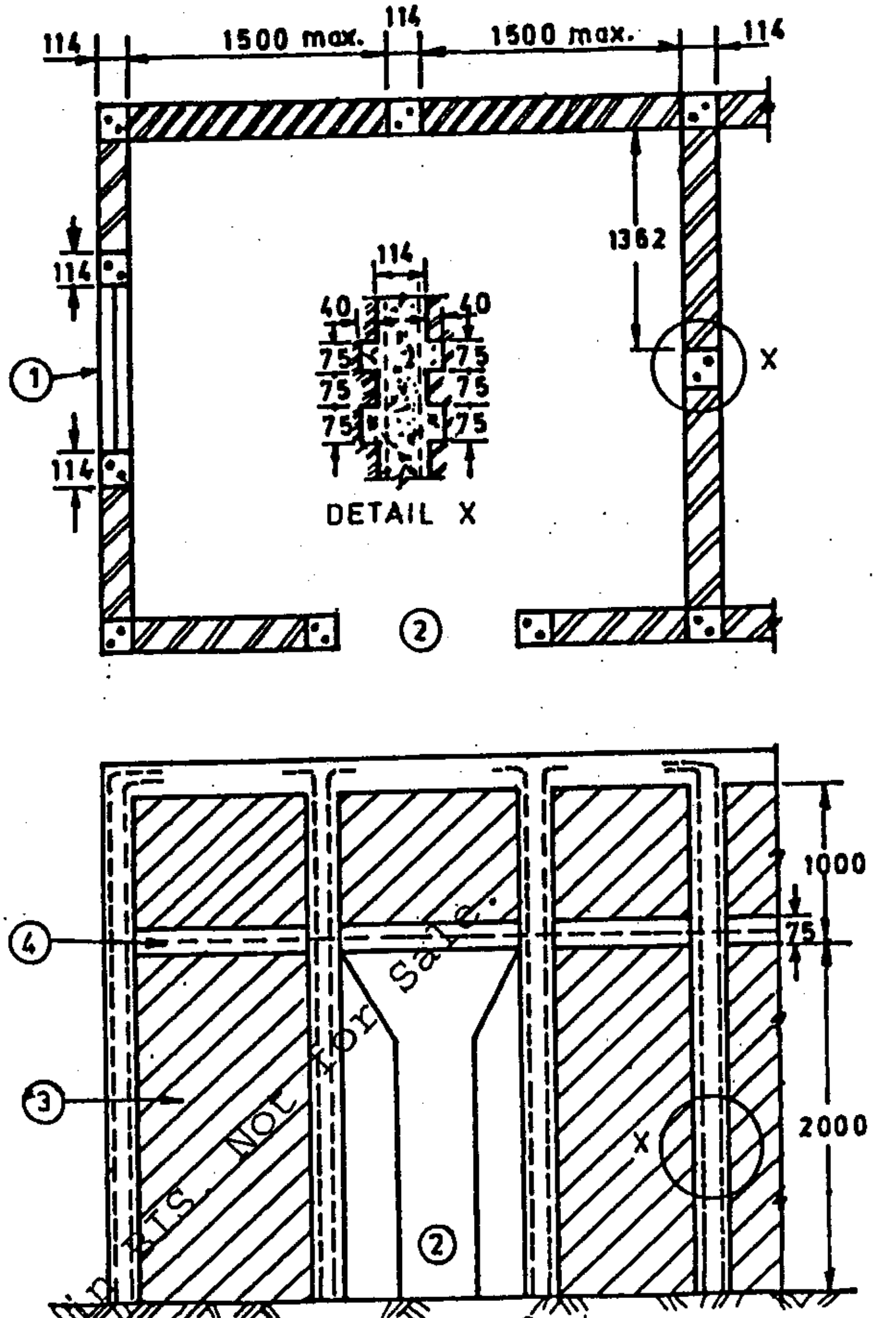
1 — One-brick length, $\frac{1}{2}$ — half - brick length, v — vertical steel bar with mortar/concrete filling in pocket

(a) and (b) — alternate courses in one brick

(c) and (d) — alternate courses at corner junction of $1\frac{1}{2}$ - brick wall

(e) and (f) — alternate courses at T-junction of $1\frac{1}{2}$ - brick wall

FIG. 12 TYPICAL DETAILS OF PROVIDING VERTICAL STEEL BARS IN BRICK MASONRY



1 — window
3 — brick panel
2 — door
4 — lintel band

All dimensions in millimetres.

FIG. 13 FRAMING OF THIN LOAD-BEARING BRICK WALLS

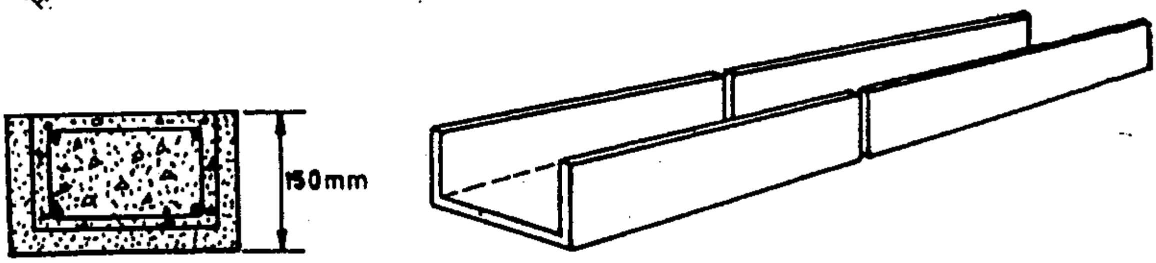


FIG. 14 U-BLOCKS FOR HORIZONTAL BANDS

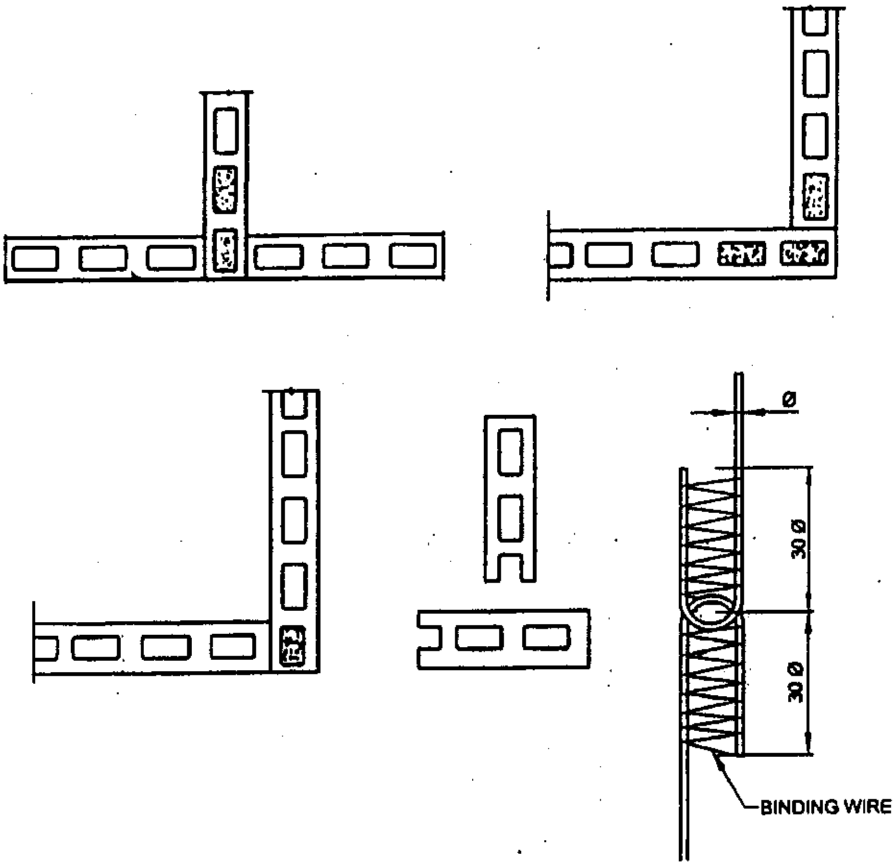


FIG. 15 VERTICAL REINFORCEMENT IN CAVITIES

(*see* Fig. 16). The nominal width of the unit varies from 300 to 600 mm, its height from 150 to 200 mm and a minimum flange thickness of 30 mm. Length of unit shall vary according to room dimensions, but the maximum length is restricted to 4.2 m from stiffness considerations. Horizontal corrugations are provided on the two longitudinal faces of the units so that the structural roof/floor acts monolithic after concrete grouted in the joints between the units attains strength (*see* Fig. 17).

9.1.2 *Precast Reinforced Concrete Cored Unit Roof/Floor*

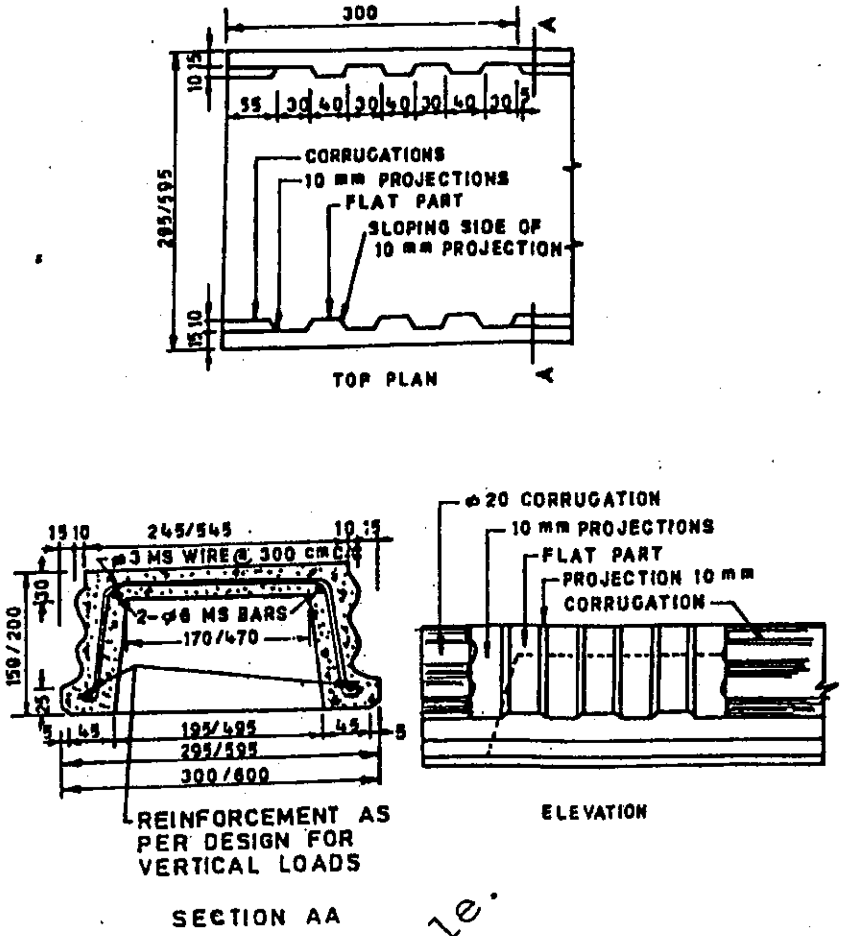
The unit is a reinforced concrete component having a nominal width of 300 to 600 mm and thickness of 130 to 150 mm having two circular hollows 90 mm diameter, throughout the length of the unit (*see* Fig. 18). The minimum flange/web thickness of the unit shall be 20 mm. Length of unit varies according to room dimensions, but the maximum length shall be restricted to 4.2 m from stiffness considerations. Horizontal corrugations are provided on the two longitudinal faces of the units so that the structural roof/floor acts monolithic after concrete grouted in the joints between the units attains strength (*see* Fig. 19).

9.1.3 *Precast Reinforced Concrete Plank and Joist Scheme for Roof/Floor*

The scheme consists of precast reinforced concrete planks supported on partially precast reinforced concrete joists. The reinforced concrete planks are 300 mm wide and the length varies according to the spacing of the joists, but it shall not exceed 1.5 m (*see* Fig. 20). To provide monolithicity to the roof/floor and to have T-beam effect with the joists, the planks shall be made partially 30 mm thick and the partially 60 mm thick and *in-situ* concrete shall be filled in the depressed portions to complete the roof/floor structurally (*see* Fig. 21).

9.1.4 *Prefabricated Brick Panel System for Roof/Floor*

It consists of prefabricated reinforced brick panels (*see* Fig. 22) supported on precast reinforced concrete joists with nominal reinforced 35 mm thick structural deck concrete over the brick panels and joists (*see* Fig. 23). The width of the brick panels shall be 530 mm for panels made of bricks of conventional size and 450 mm for panels made of bricks of modular size. The thickness of the panels shall be 75 mm or 90 mm respectively depending upon whether conventional or modular bricks are used. The length of the panels shall



All dimensions in millimetres.

FIG. 16 CHANNEL UNITS

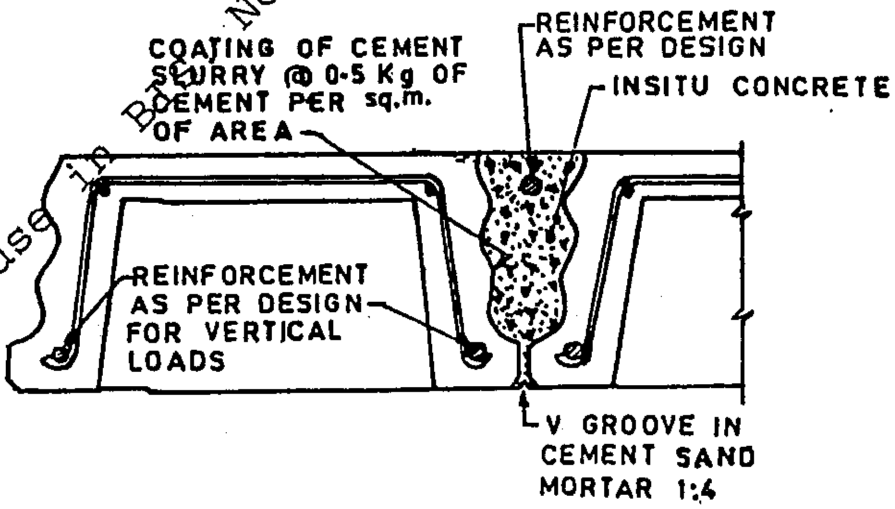
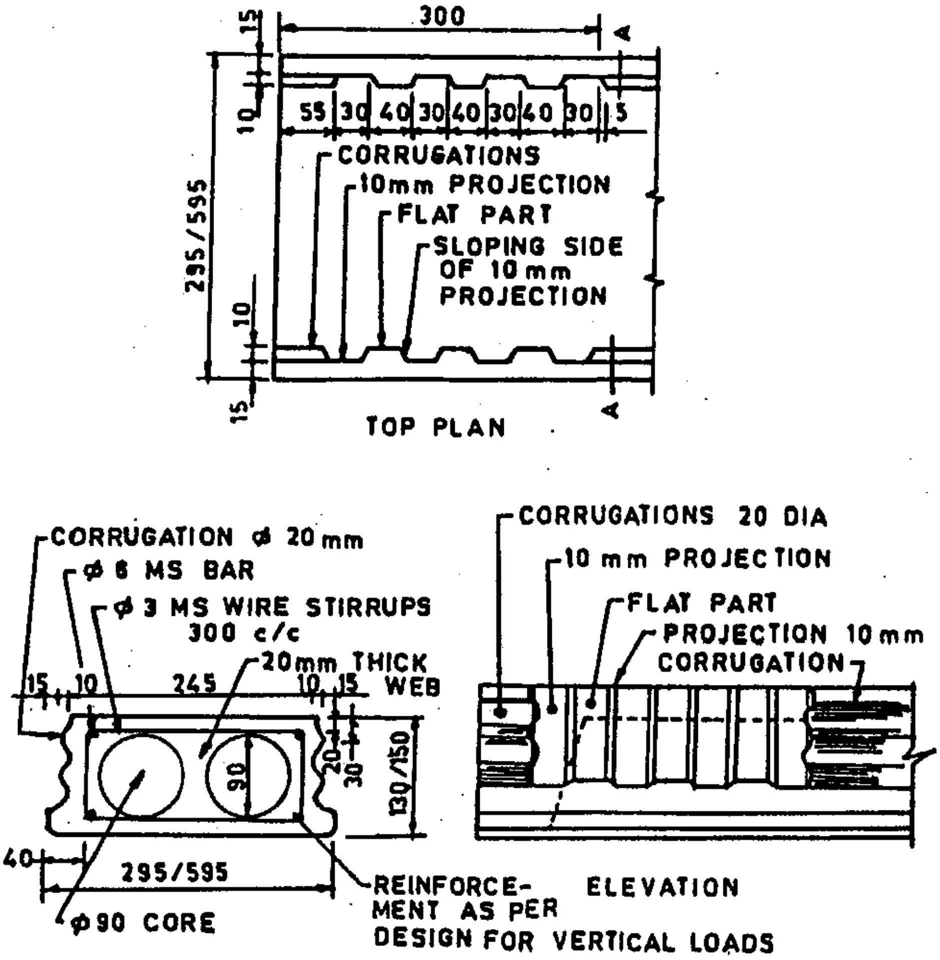
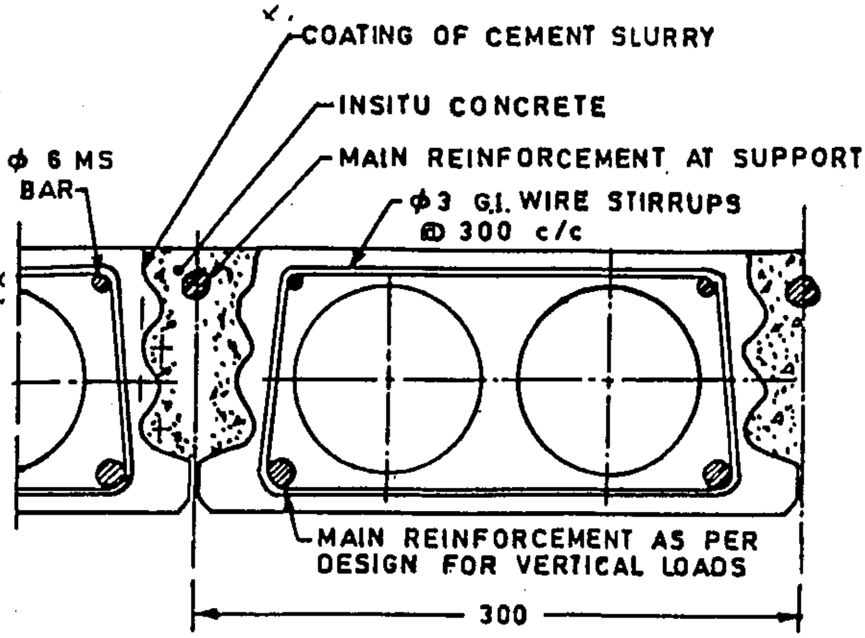


FIG. 17 CHANNEL UNIT FLOOR



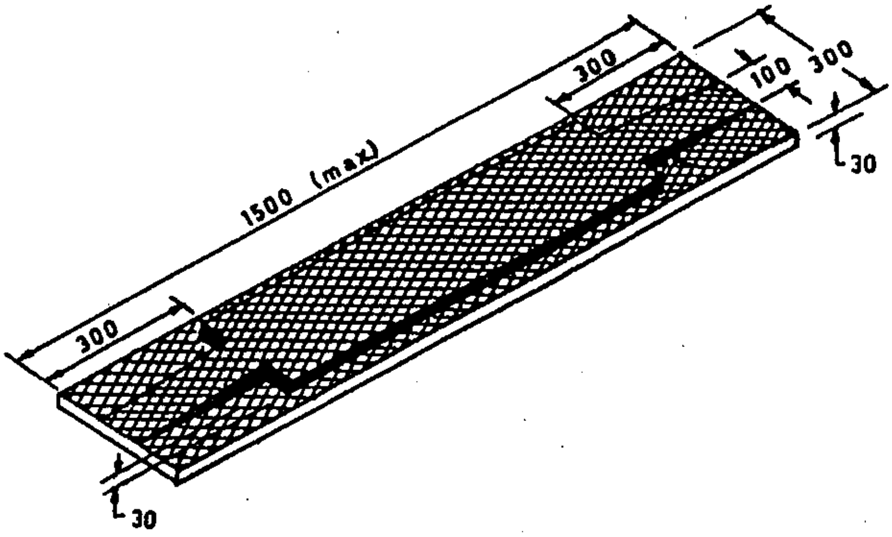
All dimensions in millimetres.

FIG. 18 CORE UNITS



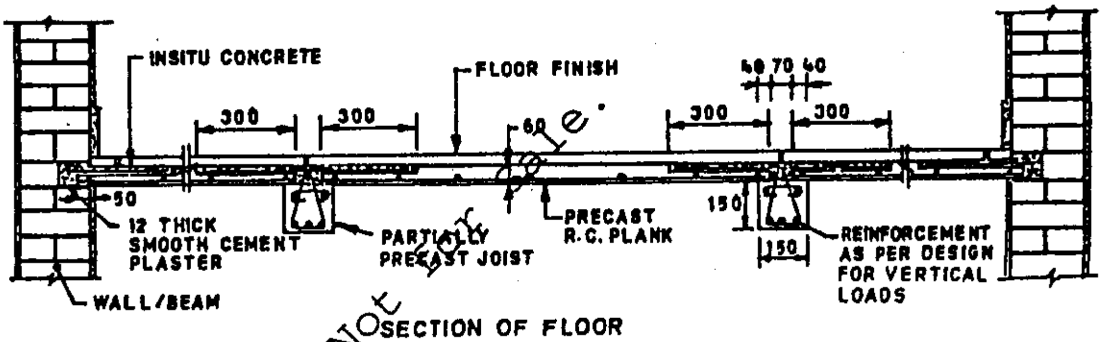
All dimensions in millimetres.

FIG. 19 CORED, UNIT FLOOR



All dimensions in millimetres.

FIG. 20 PRECAST REINFORCED CONCRETE PLANK



All dimensions in millimetres.

FIG. 21 PRECAST REINFORCED CONCRETE PLANK FLOOR

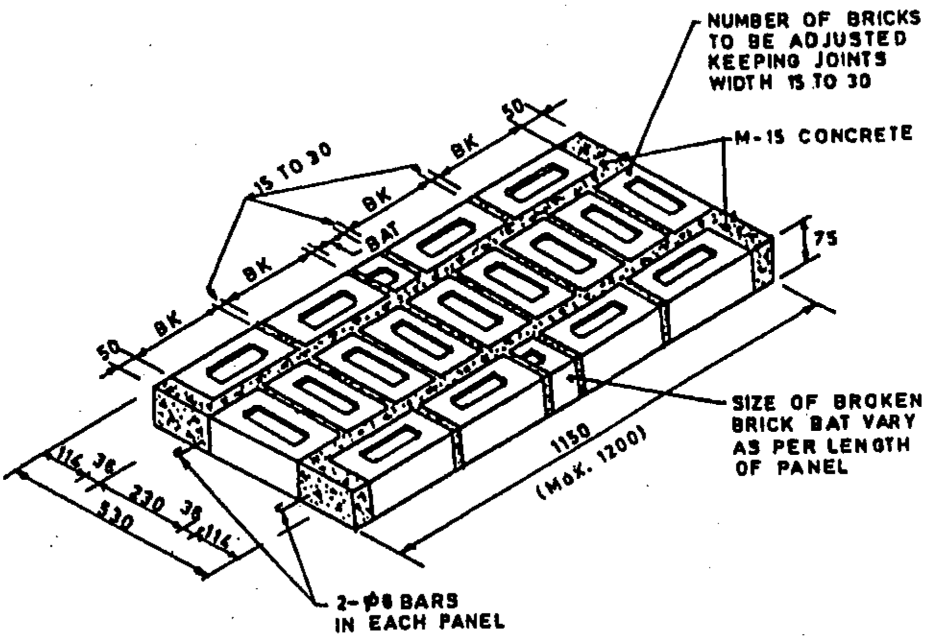
vary depending upon the spacing of the joists, but the maximum length shall not exceed 1.2 m.

9.1.5 *Precast Reinforced Concrete Waffle Unit Roof/Floor*

Waffle units are of the shape of inverted troughs, square or rectangular in plan, having lateral dimensions up to 1.2 m and depth depending upon the span of the roof/floor to be covered (*see* Fig. 24 and Fig. 25). The minimum thickness of flange/web shall be 35 mm. Horizontal projections may be provided on all the four external faces of the unit and the unit shall be so shaped that it shall act monolithic with *in-situ* concrete to ensure load transfer. Vertical castallations, called shear keys, shall be provided on all the four external faces of the precast units to enable them to transfer horizontal shear force from one unit to adjacent unit through *in-situ* concrete filled in the joints between the units. The waffle units shall be laid in a grid pattern with gaps between two adjacent units, and reinforcement, as per design, and structural concrete shall be provided in the gaps between the units in both the directions. The scheme is suitable for two way spanning roofs and floors of buildings having large spans.

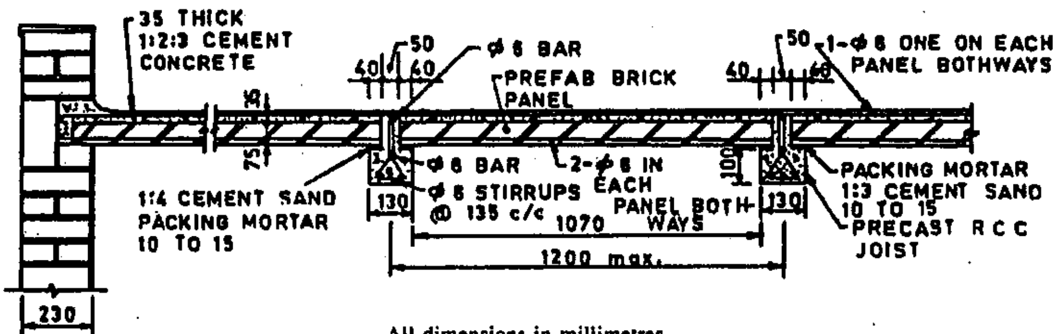
9.2 Seismic Resistance Measures

9.2.1 All floors and roofs to be constructed with small precast components shall be strengthened as



All dimensions in millimetres.

FIG. 22 PREFAB BRICK PANEL



All dimensions in millimetres.

FIG. 23 BRICK PANEL FLOOR

specified for various categories of buildings in Table 8. The strengthening measures are detailed in 9.2.3 and 9.2.8.

9.2.2 Vertical castallations called shear keys, shall be provided on the longitudinal faces of the channel, cored and waffle units to enable them to transfer horizontal shear force from one unit to the adjacent unit through the *in-situ* concrete filled in the joints between the units. The minimum percentage of area of shear keys as calculated below, on each face of the unit, shall be 15.

Shear keys shall have a minimum width of 40 mm at its root with the body of the component and shall be to the full height of the component and preferably at uniform spacing. Percentage of area of shear keys shall be calculated as:

$$\frac{\text{No. of shear keys on one face of the component} \quad 40}{\text{Length of the face of the component, in mm}} \quad 100$$

Table 8 Strengthening Measures for Floors/Roofs with Small Precast Components

(*Clause* 9.2.1)

Sl No.	Building Category	Number of Storeys	Strengthening to be Provided in Floor/Roof with			
			Channel/ Cored Unit	R.C. Planks and Joists	Brick Panels and Joists	Waffle Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	B	1 to 3	a	a	a	a
		4	a, c	a, c	a, d	a
ii)	C	1 and 2	a, b	a	a	a
		3 and 4	a, b, c	a, c	a, d	a, e
iii)	D	1 - 4	a, b c	a, c	a, d	a, c, e
iv)	E	1 - 3	a, b, c	a, c	a, d	a, c, e

NOTE:

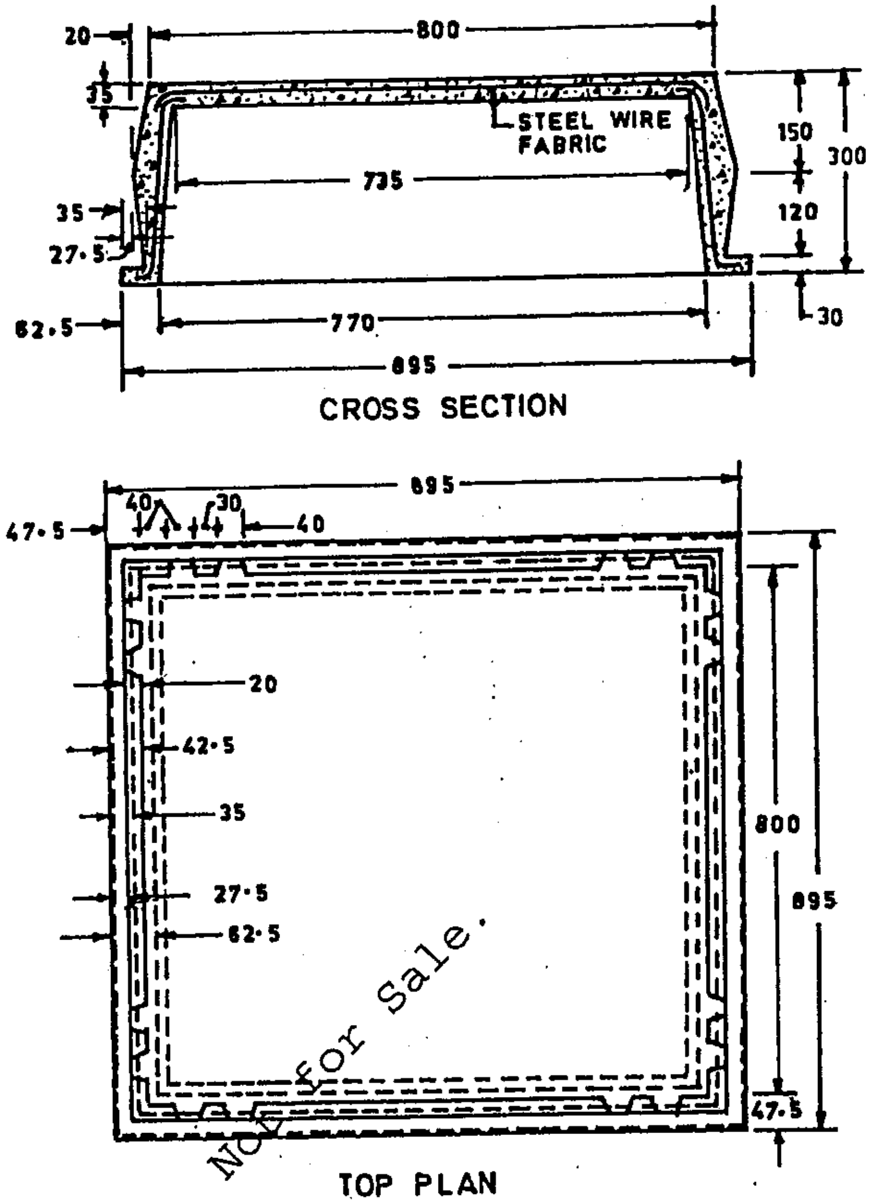
a — tie beam as per 9.2.3;

b — reinforcing bars of precast unit and tied to tie beam reinforcement as per 9.2.4;

c — reinforced deck concrete as per 9.2.5;

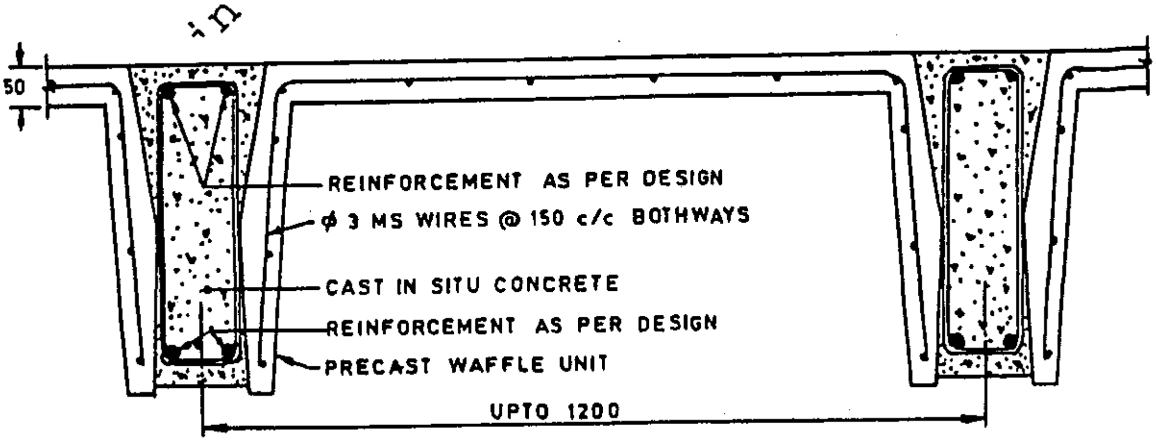
d — reinforced deck concrete as per 9.2.6; and

e — reinforced bars in joint between precast waffle units tied to tie beam reinforcement as per 9.2.7.



All dimensions in millimetres.

FIG. 24 WAFFLE UNITS



All dimensions in millimetres.

FIG. 25 WAFFLE UNIT FLOOR

9.2.3 Tie beam (*see* Table 8) is a beam provided all round the floor or roof to bind together all the precast components to make it a diaphragm. The beams shall be to the full width of the supporting wall or beam less the bearing of the precast components. The depth of the beam shall be equal to the depth of the precast components plus the thickness of structural deck concrete, where used over the components. The beam shall be made of cement concrete of grade not leaner than M15 and shall be reinforced as indicated in Table 6. If depth of tie is more than 75 mm, equivalent reinforcement shall be provided with one bar of minimum diameter 8 mm at each corner. Tie beams shall be provided on all longitudinal and cross walls. Typical details of the beams are shown in Fig. 26 to Fig. 30.

NOTE — Adequate edge support say 60 mm, shall be provided to precast element on the wall so as to avert its slippage during seismic ground motion.

9.2.4 Top reinforcement in the channel or cored units (*see* Table 8) shall be projected out at both the ends for anchorage length and tied to tie beam reinforcement.

9.2.5 Structural deck concrete (*see* Table 8) of grade not leaner than M15 shall be provided over precast components or act monolithic. Wherever, deck concrete is to be provided, the top surface of the components shall be finished rough. Cement slurry with 0.5 kg of cement/m^2 of the surface area shall be applied over the components immediately before laying the deck concrete and the concrete shall be compacted using plate vibrators. The minimum thickness of deck concrete shall be 35 mm or 40 mm reinforced with 6 mm diameter bars and 150 mm apart both ways and anchored into the tie beam placed all round. The maximum size of coarse aggregate used in deck concrete shall not exceed 12 mm.

NOTE — Under conditions of economic constraints, the deck concrete itself could serve as floor finish. The concrete is laid in one operation (*see* Fig. 30) without joints.

9.2.6 The deck concrete normally used over the brick panel with joist floor shall be reinforced with 6 mm diameter bars spaced 150 mm apart both ways (*see* Table 8).

9.2.7 For floors/roofs with precast waffle units, two 16 mm diameter high strength deformed bars shall be provided as top reinforcement in the joints between waffle units, in addition to reinforcement required for taking bending moment for vertical loads. This reinforcement (*see* Table 8) shall be fixed to tie beam reinforcement.

9.2.8 In case of floors/roofs with precast components other than those indicated in Table 8, the buildings shall be analyzed for maximum expected seismic forces and the floor/roof shall be designed to act as diaphragm and take care of the resulting forces.

10 TIMBER CONSTRUCTION

10.1 Timber has higher strength per unit weight and is, therefore, very suitably for earthquake resistant construction. Materials, design and construction in timber shall generally conform to IS 883.

10.2 Timber construction shall generally be restricted to two storeys with or without the attic floor.

10.3 In timber construction attention shall be paid to fire safety against electric short circuiting, kitchen fire, etc.

10.4 The superstructure of timber buildings shall be made rigid against deformations by adopting suitable construction details at the junctions of the framing members and in wall panels as given in 10.6 to 10.10 so that the construction as a whole behaves as one unit against earthquake forces.

10.5 Foundations

10.5.1 Timber construction shall preferably start above the plinth level, the portion below shall be in masonry or concrete.

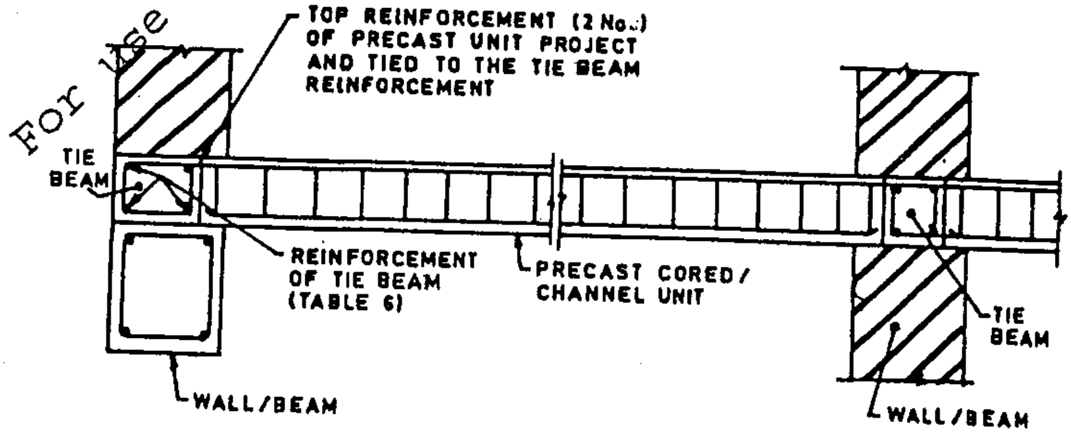
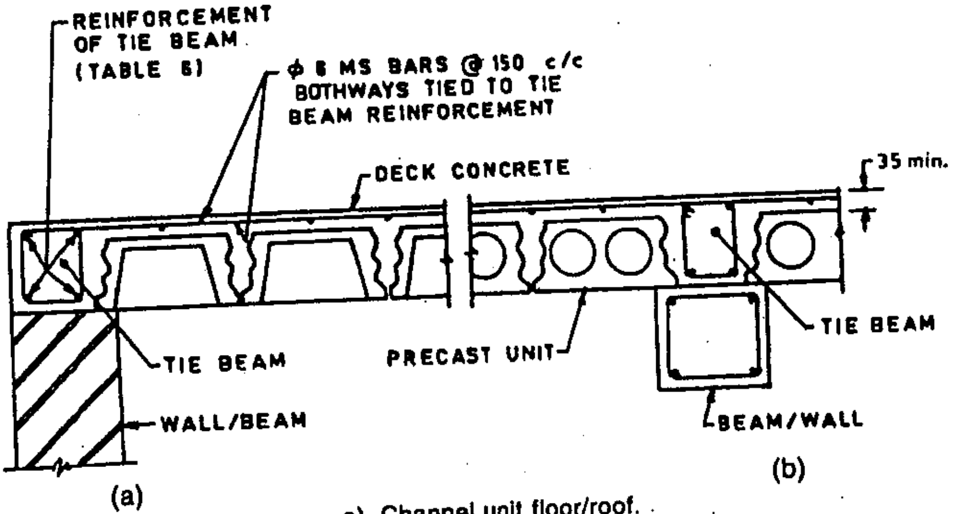


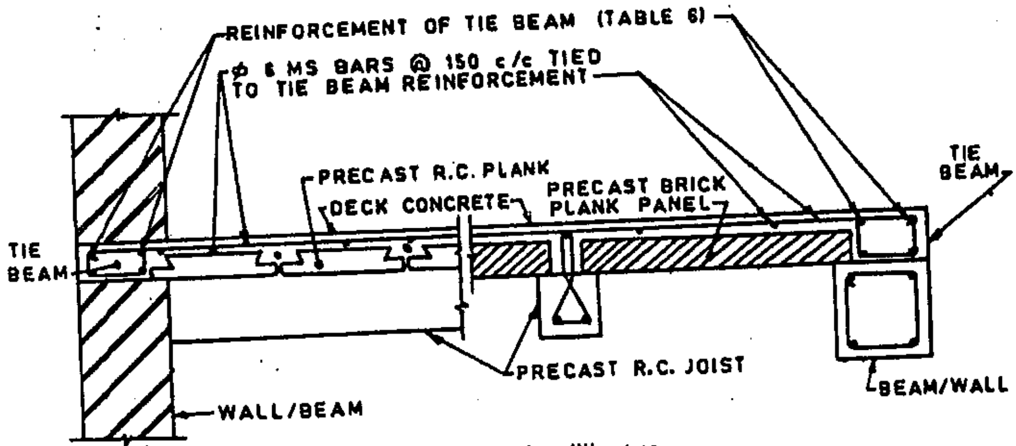
FIG. 26 CONNECTION OF PRECAST CORED/CHANNEL UNIT WITH TIE BEAM



a) Channel unit floor/roof.
b) Cored unit floor/roof.

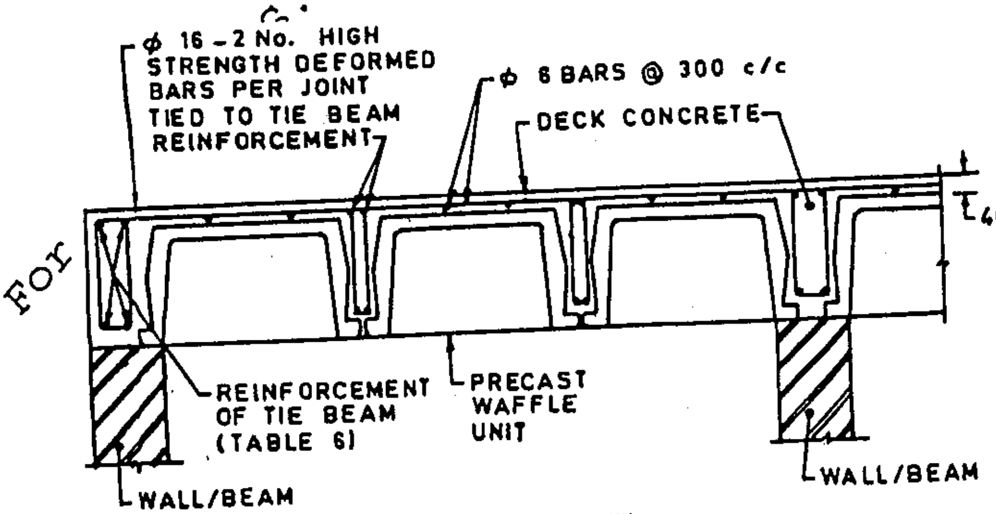
All dimensions in millimetres.

FIG. 27 CONNECTION OF CHANNEL/CORED UNIT FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM



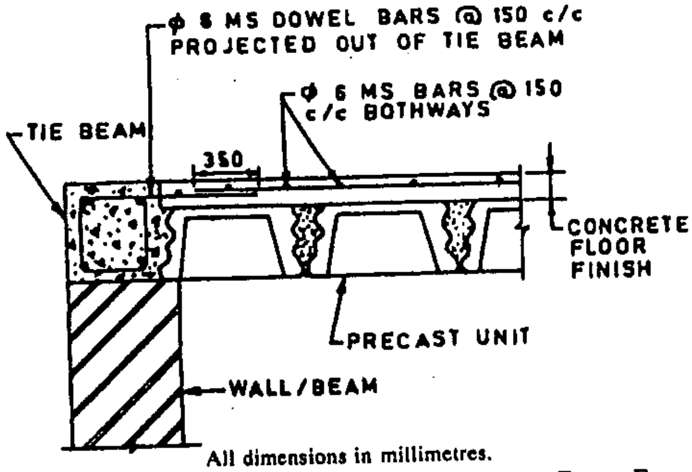
All dimensions in millimetres.

FIG. 28 CONNECTION OF PRECAST REINFORCED CONCRETE PLANK AND PRECAST BRICK PANEL FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM



All dimensions in millimetres.

FIG. 29 CONNECTION OF PRECAST WAFFLE UNIT FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM



All dimensions in millimetres.

FIG. 30 PROVISION OF REINFORCEMENT IN CONCRETE FLOOR FINISH

10.5.2 The superstructure may be connected with the foundation in one of the two ways as given in **10.5.2.1** to **10.5.2.2**.

10.5.2.1 The superstructure may simply rest on the plinth masonry, or in the case of small buildings of one storey having plan area less than about 50 m², it may rest on firm plane ground so that the building is free to slide laterally during ground motion.

NOTES

1 Past experience has shown that superstructure of the buildings not fixed with the foundation escaped collapse even in a severe earthquake although they were shifted sideways.

2 Where fittings for water supply or water borne sanitation from the house are to be installed, proper attention should be given to permit movement so as to avoid fracture of damage to pipes.

10.5.2.2 The superstructure may be rigidly fixed into the plinth masonry or concrete foundation as given in Fig. 31 or in case of small building having plan area less than 50 m², it may be fixed to vertical poles embedded into the ground. In each case the building is likely to move along with its foundation. Therefore, the superstructure shall be designed to carry the resulting earthquake shears.

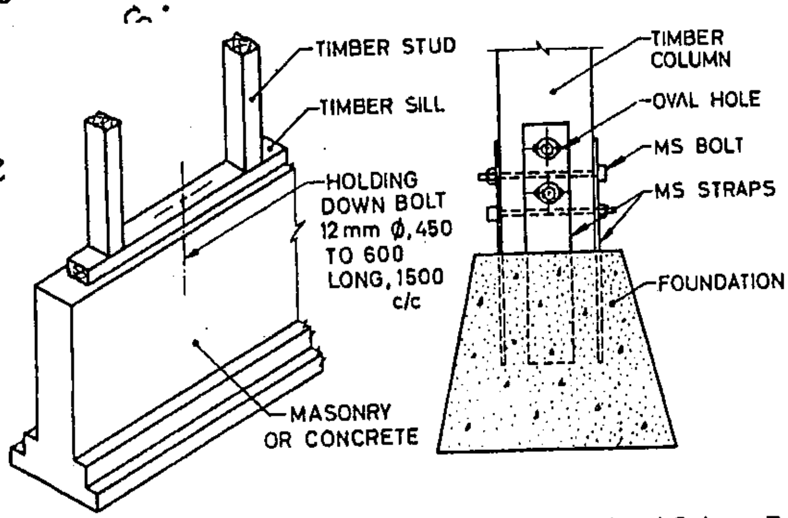
10.6 Types of Framing

The types of construction usually adopted in timber building are as follows:

a) Stud wall construction; and
b) Brick nogged timber frame construction.

10.7 Stud Wall Construction

10.7.1 The stud wall construction consists of timber studs and corner posts framed into sills, top plates and wall plates. Horizontal struts and diagonal braces are used to stiffen the frame against lateral loads. The wall covering may consist of EKRA, timber or like. Typical details of stud walls are shown in Fig. 32. Minimum sizes and spacing of various members used are specified in **10.7.2** to **10.7.10**.



31A Suitable for Strip Foundation 31B Suitable for Isolated Column Footings

All dimensions in millimetres.

FIG. 31 DETAILS OF CONNECTION OF COLUMN WITH FOUNDATION

10.7.2 The timber studs for use in load bearing walls shall have a minimum finished size of 40 × 90 mm and their spacing shall not exceed those given in Table 9.

Table 9 Maximum Spacing of 40 mm × 90 mm Finished Size Studs in Stud Wall Construction

Sl No.	Group of Timber [Grade I[1)]]	Single Storeyed or First Floor of the Double Storeyed Buildings		Ground Floor of Double Storeyed Buildings	
		Exterior Wall cm	Interior Wall cm	Exterior Wall cm	Interior Wall cm
(1)	(2)	(3)	(4)	(5)	(6)
i)	Group A, B	100	80	50	40
ii)	Group C	100	100	50	50

[1)]Grade I timbers as defined in Table 5 of IS 883.

10.7.3 The timber studs in non-load bearing walls shall not be less than 40 × 70 mm in finished cross-section. Their spacing shall not exceed 1 m.

10.7.4 There shall be at least one diagonal brace for every 1.6 m × 1 m area of load bearing walls. Their minimum finished sizes shall be in accordance with Table 10.

10.7.5 The horizontal struts shall be spaced not more than 1 m apart. They shall have a minimum size of 30 × 40 mm for all locations.

10.7.6 The finished sizes of the sill, the wall plate and top plate shall not be less than the size of the studs used in the wall.

10.7.7 The corner posts shall consists of three timbers, two being equal in size to the studs used in the walls meeting at the corner and the third timber being of a size to fit so as to make a rectangular sections (*see* Fig. 32).

10.7.8 The diagonal braces shall be connected at their ends with the stud wall members by means of wire nails having 6 gauge (4.88 mm diameter) and 10 cm length. Their minimum number shall be 4 nails for 20 mm × 40 mm braces and 6 nails for 30 mm × 40 mm braces. The far end of nails may be clutched as far as possible.

10.7.9 Horizontal bracing shall be provided at corners of T-junctions of walls at sill, first floor and eave levels. The bracing members shall have a minimum finished size of 20 mm × 90 mm and shall be connected by means of wire nails to the wall plates at a distance between 1.2 m and 1.8 m measured from the junction of the walls. There shall be a minimum number of six nails of 6 gauge (4.88 mm diameter) and 10 cm length with clutching as far ends.

10.7.10 Unsheathed studding shall not be used adjacent to the wall of another building. The studding must be sheathed with close jointed 20 mm or thicker boards.

10.8 Brick Nogged Timber Frame Construction

10.8.1 The brick nogged timber frame consists of intermediate verticals, columns, sills, wall plates, horizontal nogging members and diagonal braces framed into each other and the space between framing members filled with tight-fitting brick masonry in stretcher bond. Typical details of brick nogged timber frame construction are shown in Fig. 33. Minimum sizes and spacing of various elements used are specified in **10.8.2** to **10.8.9**.

10.8.2 The vertical framing members in brick nogged load bearing walls shall have minimum finished sizes as specified in Table 10.

10.8.3 The minimum finished size of the vertical members in non-load bearing walls shall be 40 mm × 100 mm spaced not more than 1.5 m apart.

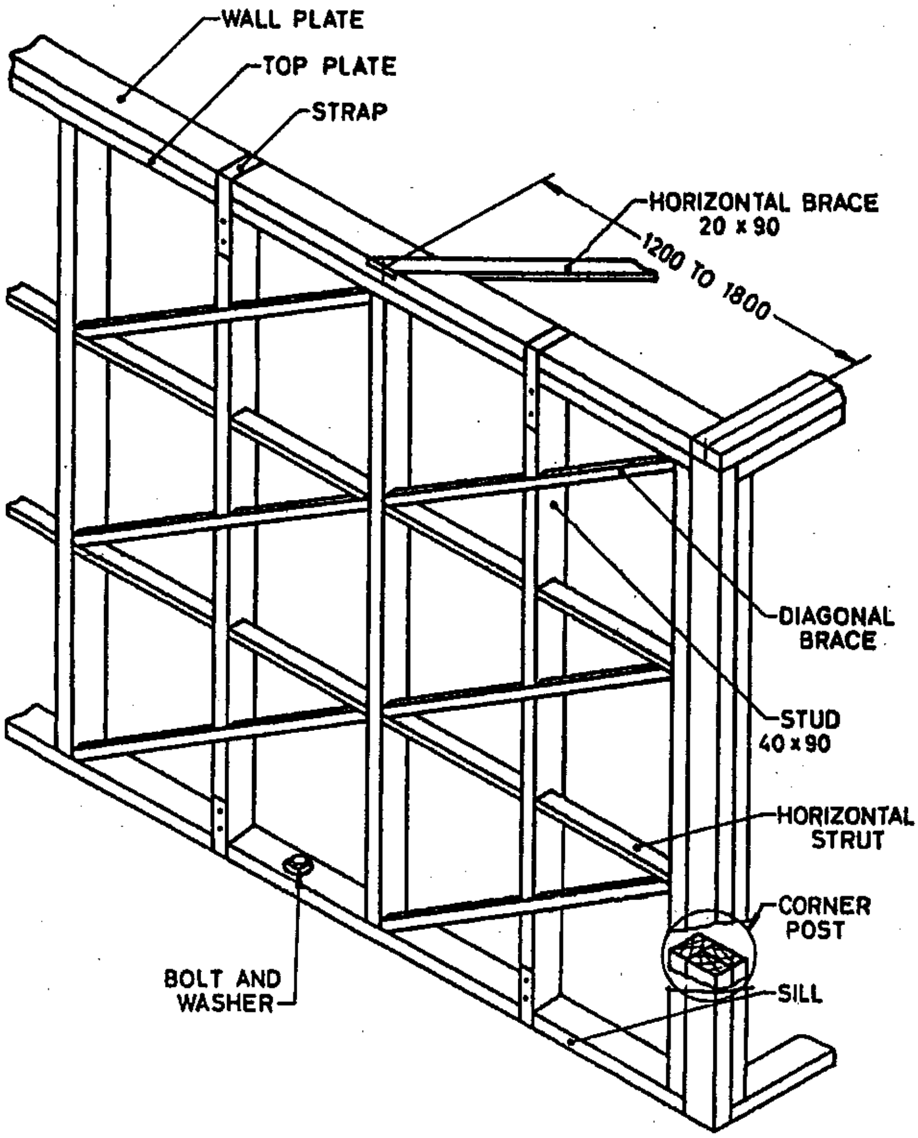
10.8.4 The sizes of diagonal bracing members shall be the same as in Table 10.

10.8.5 The horizontal framing members in brick-nogged construction shall be spaced not more than 1 m apart. Their minimum finished sizes shall be in accordance with Table 11 and Table 12.

Table 10 Minimum Finished Sizes of Diagonal Braces
(*Clause* 10.8.4)

Sl No.	Building Category (*see* Table 2)	Group of Timber [Grade I[1)]]	Single Storeyed or First Floor of Double Storeyed Buildings		Ground Floor of Double Storeyed Buildings	
			Exterior Wall mm × mm	Interior Wall mm × mm	Exterior Wall mm × mm	Interior Wall mm × mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	B, C	All	20 × 40	20 × 40	20 × 40	20 × 40
ii)	D and E	Group A and Group B	20 × 40	20 × 40	20 × 40	30 × 40
iii)	Group C	Group C	20 × 40	30 × 40	30 × 40	30 × 40

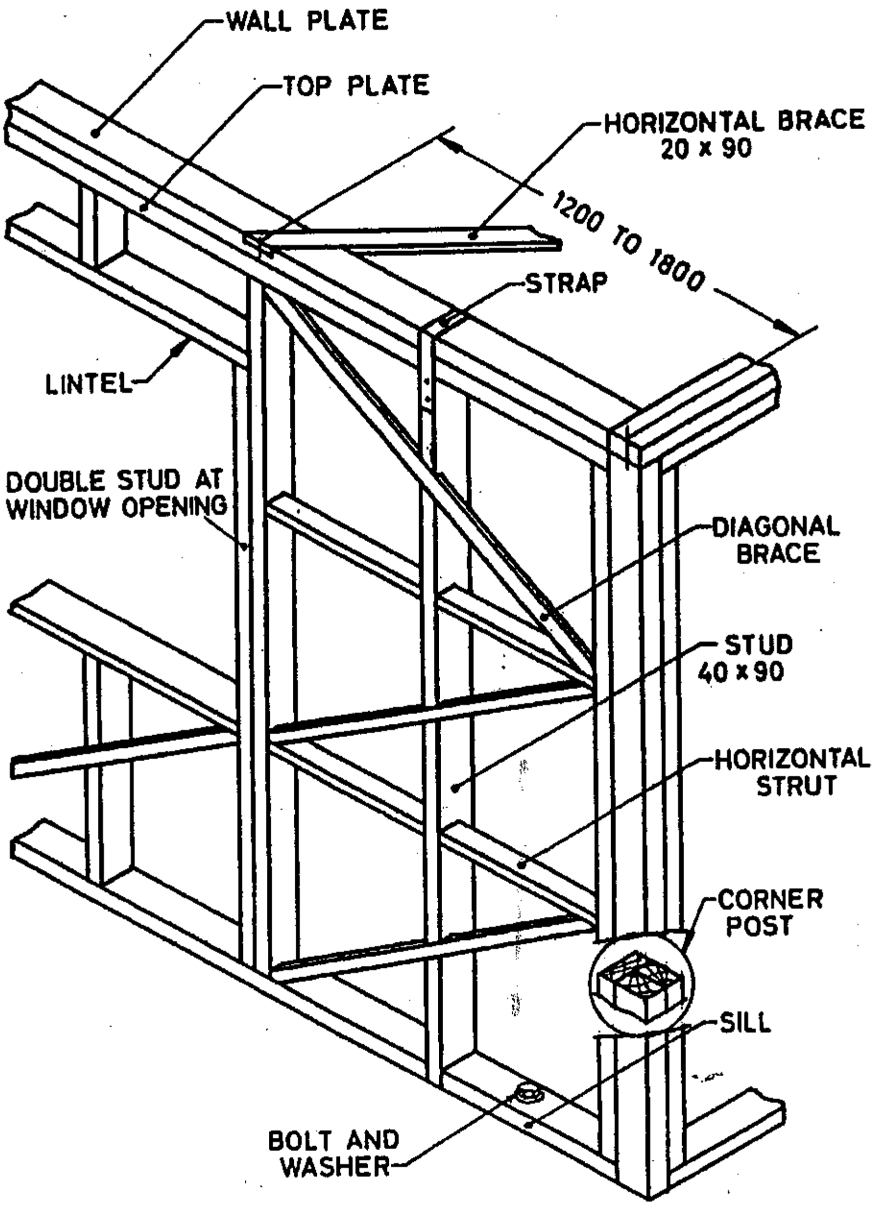
[1)]Grade I timber as defined in Table 5 of IS 883.



32A Timber Framing in Stud Wall Construction without Opening in Wall

All dimensions in millimetres.

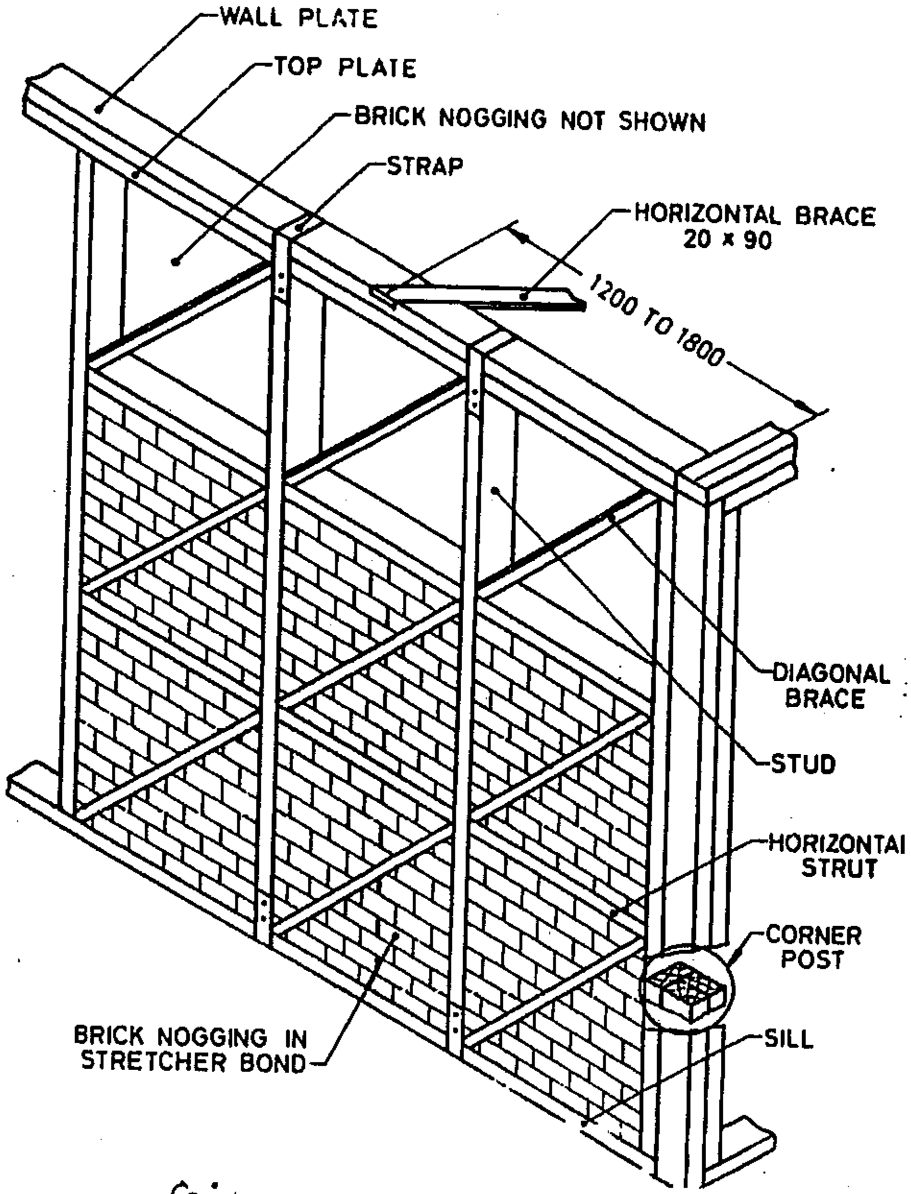
FIG. 32 STUD WALL CONSTRUCTION (*Continued*)



32B Timber Framing in Stud Wall Construction with Opening in Wall

All dimensions in millimetres.

FIG. 32 STUD WALL CONSTRUCTION



All dimensions in millimetres.

FIG. 33 BRICK NOGGED TIMBER FRAME CONSTRUCTION

Table 11 Minimum Finished Sizes of Vertical in Brick Nogged Timber Frame Construction

(*Clause* 10.8.5)

Sl No.	Spacing	Group of Timber [Grade I[1]]	Single Storeyed or First Floor of Double Storeyed Buildings		Ground Floor of Double Storeyed Buildings	
			Exterior Wall	Interior Wall	Exterior Wall	Interior Wall
	m		mm × mm	mm × mm	mm × mm	mm × mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	1	Group A, B	50 × 100	50 × 100	50 × 100	50 × 100
		Group C	50 × 100	70 × 100	70 × 100	90 × 100
ii)	1.5	Group A, B	50 × 100	70 × 100	70 × 100	80 × 100
		Group C	70 × 100	80 × 100	80 × 100	100 × 100

[1]Grade I timber as defined in Table 5 of IS 883.

Table 12 Minimum Finished Size of Horizontal Nogging Members

(*Clause* 10.8.5)

Sl No. (1)	Spacing of Verticals m (2)	Size mm (3)
i)	1.5	70 × 100
ii)	1	50 × 100
iii)	0.5	25 × 100

10.8.6 The finished sizes of the sill, wall plate and top plate shall be not less than the size of the vertical members used in the wall.

10.8.7 Corner posts shall consist of three vertical timbers as described in **10.7.7**.

10.8.8 The diagonal braces shall be connected of their ends with the other members of the wall by means of wire nails as specified in **10.7.8**.

10.8.9 Horizontal bracing members of corners of T-junctions of wall shall be as specified in **10.7.9**.

10.9 Notching and Cutting

10.9.1 Timber framing frequently requires notching and cutting of the vertical members. The notching or cutting should in general be limited to 20 mm in depth unless steel strips are provided to strengthen the notched face of the members. Such steel strips, where necessary shall be at least 1.5 mm thick and 35 mm wide extending at least 15 cm beyond each side of the notch or cut and attached to the vertical member by means of bolts or screws at each end.

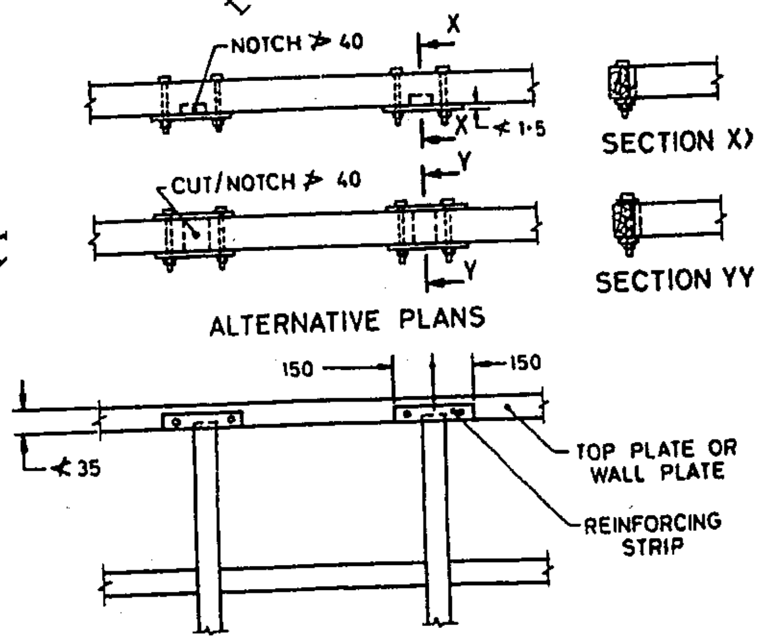
10.9.2 The top plate, the wall plate or the sill of a wall may be notched or cut, if reinforcing strip of iron is provided as specified in **10.9.1**. In case the member is notched or cut not to exceed 40 mm in depth, such reinforcing strip may be placed along the notched edge only. Where the notch or cut is more than 40 mm in depth or the member is completely cut through, such reinforcing strips shall be placed on both edges of the member. The details of notching and cutting are shown in Fig. 34.

10.9.3 Joints in timber shall preferably be bound by metallic fasteners.

10.10 Bridging and Blocking

10.10.1 All wooden joists shall have at least one row of cross bridging for every 3.5 m length of span. The cross-section of the bridging member shall be a minimum of 40 × 70 mm and the member shall be screwed or nailed to the joists.

10.10.2 All spaces between joists shall be blocked at all bearing with solid blocks not less than 40 mm thick and the full depth of the joists. The block shall be screwed or nailed to the joists as well as to the bearings.



All dimensions in millimetres.

FIG. 34 NOTCHING AND CUTTING

ANNEX A
(Foreword)

COMMITTEE COMPOSITION

Earthquake Engineering Sectional Committee, CED 39

Organization	*Representative(s)*
In personal capacity (*L 801, Design Arch Building Sector-5, Vaishali, Ghaziabad 201010*)	Prof A. S. Arya (*Chairman*)
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Indian Standard

DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

(Third Reprint NOVEMBER 1996)

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Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

IS 4326 : 1976 'Code of practice for earthquake resistant design and construction of buildings' while covering certain special features for the design and construction of earthquake resistant buildings included some details for achieving ductility in reinforced concrete buildings. With a view to keep abreast of the rapid developments and extensive research that has been carried out in the field of earthquake resistant design of reinforced concrete structures, the technical committee decided to cover provisions for the earthquake resistant design and detailing of reinforced concrete structures separately.

This code incorporates a number of important provisions hitherto not covered in IS 4326 : 1976. The major thrust in the formulation of this standard is one of the following lines:

a) As a result of the experience gained from the performance, in recent earthquakes, of reinforced concrete structures that were designed and detailed as per IS 4326 : 1976, many deficiencies thus identified have been corrected in this code.

b) Provisions on detailing of beams and columns have been revised with an aim of providing them with adequate toughness and ductility so as to make them capable of undergoing extensive inelastic deformations and dissipating seismic energy in a stable manner.

c) Specifications on a seismic design and detailing of reinforced concrete shear walls have been included.

The other significant changes incorporated in this code are as follows:

a) Material specifications are indicated for lateral force resisting elements of frames.

b) Geometric constraints are imposed on the cross section for flexural members. Provisions on minimum and maximum reinforcement have been revised. The requirements for detailing of longitudinal reinforcement in beams at joint faces, splices, and anchorage requirements are made more explicit. Provision are also included for calculation of design shear force and for detailing of transverse reinforcement in beams.

c) For members subjected to axial load and flexure, the dimensional constraints have been imposed on the cross section. Provisions are included for detailing of lap splices and for the calculation of design shear force. A comprehensive set of requirements is included on the provision of special confining reinforcement in those regions of a column that are expected to undergo cyclic inelastic deformations during a severe earthquake.

d) Provisions have been included for estimating the shear strength and flexural strength of shear wall sections. Provisions are also given for detailing of reinforcement in the wall web, boundary elements, coupling beams, around openings, at construction joints, and for the development, splicing and anchorage of reinforcement.

Whilst the common methods of design and construction have been covered in this code, special systems of design and construction of any plain or reinforced concrete structure not covered by this code may be permitted on production of satisfactory evidence regarding their adequacy for seismic performance by analysis or tests or both.

The Sectional Committee responsible for the preparation of this standard has taken into consideration the view of manufacturers, users, engineers, architects, builders and technologists and has related the standard to the practices followed in the country in this field. Due weightage has also been given to the need for international co-ordination among standards prevailing in different seismic regions of the world.

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

i) ACI 318-89/318R-89, Building code requirements for reinforced concrete and commentary, published by American Concrete Institute.

ii) ATC-11, Seismic resistance of reinforced concrete shear walls and frame joints : Implications of recent research for design engineers, published by Applied Technology Council, USA.

iii) CAN3-A23. 3-M84, 1984, Design of concrete structures for buildings, Canadian Standards Association.

iv) SEADC, 1980, Recommended lateral force requirements and commentary, published by Structural Engineers Association of California, USA.

The composition of the technical committees responsible for formulating this standard is given in Annex A.

Indian Standard

DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

1 SCOPE

1.1 This standard covers the requirements for designing and detailing of monolithic reinforced concrete buildings so as to give them adequate toughness and ductility to resist severe earthquake shocks without collapse.

1.1.1 Provisions of this code shall be adopted in all reinforced concrete structures which satisfy one of the following four conditions.

a) The structure is located in seismic zone IV or V;

b) The structure is located in seismic zone III and has the importance factor (I) greater than 1·0;

c) The structure is located in seismic zone III and is an industrial structure; and

d) The structure is located in seismic zone III and is more than 5 storey high.

NOTE — The definition of seismic zone and importance factor are given in IS 1893 : 1984.

1.1.2 The provisions for reinforced concrete construction given herein apply specifically to monolithic reinforced concrete construction. Precast and/or prestressed concrete members may be used only if they can provide the same level of ductility as that of a monolithic reinforced concrete construction during or after an earthquake.

2 REFERENCES

2.1 The Indian Standards listed below are necessary adjunct to this standard:

IS No.	*Title*
456 : 1978	Code of practice for plain and reinforced concrete (*third revision*)
1786 : 1985	Specification for high strength deformed steel bars and wires for concrete reinforcement (*third revision*)
1893 : 1984	Criteria for earthquake design of structures (*fourth revision*)

3 TERMINOLOGY

3.0 For the purpose of this standard, the following definitions shall apply.

3.1 Boundary Elements

Portions along the edges of a shear wall that are strengthened by longitudinal and transverse reinforcement. They may have the same thickness as that of the wall web.

3.2 Crosstie

Is a continuous bar having a 135° hook with a 10-diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

3.3 Curvature Ductility

Is the ratio of curvature at the ultimate strength of the section to the curvature at first yield of tension steel in the section.

3.4 Hoop

Is a closed stirrup having a 135° hook with a 10-diameter extension (but not < 75 mm) at each end, that is embedded in the confined core of the section. It may also be made of two pieces of reinforcement; a U-stirrup with a 135° hook and a 10-diameter extension (but not < 75 mm) at each end, embedded in the confined core and a crosstie.

3.5 Lateral Force Resisting System

Is that part of the structural system which resists the forces induced by earthquake.

3.6 Shear Wall

A wall that is primarily designed to resist lateral forces in its own plane.

3.7 Shell Concrete

Concrete that is not confined by transverse reinforcement, is also called concrete cover.

3.8 Space Frame

A three dimensional structural system composed of interconnected members, without shear or bearing walls, so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

3.8.1 *Vertical Load Carrying Space Frame*

A space frame designed to carry all vertical loads.

3.8.2 *Moment Resisting Space Frame*

A vertical load carrying space frame in which the members and joints are capable of resisting forces primarily by flexure.

4 SYMBOLS

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. All dimensions are in mm, loads in Newton and stresses in MPa (N/sq mm) unless otherwise specified.

A_g — gross cross sectional area of column, wall

A_h — horizontal reinforcement area within spacing S_v

A_k — area of concrete core of column

A_{sd} — reinforcement along each diagonal of coupling beam

A_{sh} — area of cross section of bar forming spiral or hoop

A_{st} — area of uniformly distributed vertical reinforcement

A_v — vertical reinforcement at a joint

C_w — centre to centre distance between boundary elements

D — overall depth of beam

D_k — diameter of column core measured to the outside of spiral or hoop

d — effective depth of member

d_w — effective depth of wall section

E_s — elastic modulus of steel

f_{ck} — characteristic compressive strength of concrete cube

f_y — yield stress of steel

h — longer dimension of rectangular confining hoop measured to its outer face

h_{st} — storey height

L_{AB} — clear span of beam

l_0 — length of member over which special confining reinforcement is to be provided

l_w — horizontal length of wall

l_s — clear span of coupling beam

M_u — factored design moment on entire wall section

$M^{Ah}_{u,lim}$ — hogging moment of resistance of beam at end A

$M^{As}_{u,lim}$ — sagging moment of resistance of beam at end A

$M^{Bh}_{u,lim}$ — hogging moment of resistance of beam at end B

$M^{Bs}_{u,lim}$ — sagging moment of resistance of beam at end B

$M^{bL}_{u,lim}$ — moment of resistance of beam framing into column from the left

$M^{bR}_{u,lim}$ — moment of resistance of beam framing into column from the right

M_{uv} — flexural strength of wall web

P_u — factored axial load

S — pitch of spiral or spacing hoops

S_v — vertical spacing of horizontal reinforcement in web

t_w — thickness of wall web

V^{D+L}_a — shear at end A of beam due to dead and live loads with a partial factor of safety of 1·2 on loads

V^{D+L}_b — shear at end B of beam due to dead and live loads with a partial factor of safety of 1·2 on loads

V_j — shear resistance at a joint

V_u — factored shear force

V_{us} — shear force to be resisted by reinforcement

x_u, x_u^* — depth of neutral axis from extreme compression fibre

α — inclination of diagonal reinforcement in coupling beam

ρ — vertical reinforcement ratio

ρ_c — compression reinforcement ratio in a beam

ρ_{max} — maximum tension reinforcement ratio for a beam

ρ_{min} — minimum tension reinforcement ratio for a beam

τ_c — shear strength of concrete

$\tau_{c,max}$ — maximum permissible shear stress in section

τ_v — nominal shear stress

5 GENERAL SPECIFICATION

5.1 The design and construction of reinforced concrete buildings shall be governed by the provisions of IS 456 : 1978, except as modified by the provisions of this code.

5.2 For all buildings which are more than 3 storeys in height, the minimum grade of concrete shall preferably be M20 ($f_{ck} = 20$ MPa).

5.3 Steel reinforcements of grade Fe 415 (*see* IS 1786 : 1985) or less only shall be used.

6 FLEXURAL MEMBERS

6.1 General

These requirements apply to frame members resisting earthquake induced forces and designed to resist flexure. These members shall satisfy the following requirements.

6.1.1 The factored axial stress on the member under earthquake loading shall not exceed $0 \cdot 1 f_{ck}$.

6.1.2 The member shall preferably have a width-to-depth ratio of more than 0·3.

6.1.3 The width of the member shall not be less than 200 mm.

6.1.4 The depth D of the member shall preferably be not more than 1/4 of the clear span.

6.2 Longitudinal Reinforcement

6.2.1 a) The top as well as bottom reinforcement shall consist of at least two bars throughout the member length.

b) The tension steel ratio on any face, at any section, shall not be less than $\rho_{min} = 0 \cdot 24 \sqrt{f_{ck}/f_y}$; where f_{ck} and f_y are in MPa.

6.2.2 The maximum steel ratio on any face at any section, shall not exceed $\rho_{max} = 0 \cdot 025$.

6.2.3 The positive steel at a joint face must be at least equal to half the negative steel at that face.

6.2.4 The steel provided at each of the top and bottom face of the member at any section along its length shall be at least equal to one-fourth of the maximum negative moment steel provided at the face of either joint. It may be clarified that redistribution of moments permitted in IS 456 :1978 (clause **36.1**) will be used only for vertical load moments and not for lateral load moments.

6.2.5 In an external joint, both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90 degree bend(s) (*see* Fig. 1). In an internal joint, both face bars of the beam shall be taken continuously through the column.

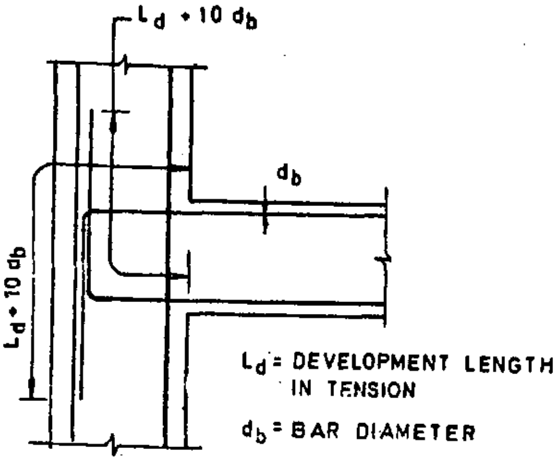


FIG. 1 ANCHORAGE OF BEAM BARS IN AN EXTERNAL JOINT

6.2.6 The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at a spacing not exceeding 150 mm (*see* Fig. 2). The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided (a) within a joint, (b) within a distance of 2*d* from joint face, and (c) within a quarter length of the member where flexural yielding may generally occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.

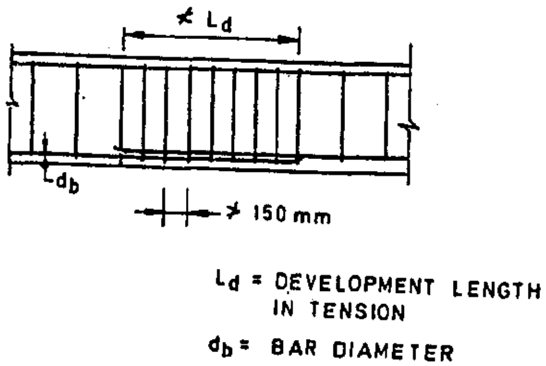


FIG. 2 LAP, SPLICE IN BEAM

6.2.7 Use of welded splices and mechanical connections may also be made, as per **25.2.5.2** of IS 456 : 1978. However, not more than half the reinforcement shall be spliced at a section where flexural yielding may take place. The location of splices shall be governed by **6.2.6**.

6.3 Web Reinforcement

6.3.1 Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end that is embedded in the confined core (*see* Fig. 3a). In compelling circumstances, it may also be made up of two pieces of reinforcement; a U-stirrup with a 135° hook and a 10 diameter extension (but not < 75 mm) at each end, embedded in the confined core and a crosstie (*see* Fig. 3b). A crosstie is a bar having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

6.3.2 The minimum diameter of the bar forming a hoop shall be 6 mm. However, in beams with clear span exceeding 5 m, the minimum bar diameter shall be 8 mm.

6.3.3 The shear force to be resisted by the vertical hoops shall be the maximum of :

a) calculated factored shear force as per analysis, and

b) shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span. This is given by (*see* Fig. 4):

i) for sway to right:

$$V_{u,a} = V_a^{D+L} - 1{\cdot}4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right]$$

$$\text{and } V_{u,b} = V_b^{D+L} + 1{\cdot}4\left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}}\right], \text{ and}$$

ii) for sway to left:

$$V_{u,a} = V_a^{D+L} + 1{\cdot}4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right]$$

$$\text{and } V_{u,b} = V_b^{D+L} - 1{\cdot}4\left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}}\right],$$

where $M_{u,lim}^{As}$, $M_{u,lim}^{Ah}$ and $M_{u,lim}^{Bs}$, $M_{u,lim}^{Bh}$ are the sagging and hogging moments of resistance of the beam section at ends A and B, respectively. These are to be calculated as per IS 456 : 1978. L_{AB} is clear span of beam. V_a^{D+L} and V_b^{D+L} are the shears at ends A and B, respectively, due to vertical loads with a partial safety factor of 1·2 on loads. The design shear at end A shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear at end B shall be the larger of the two values of $V_{u,b}$ computed above.

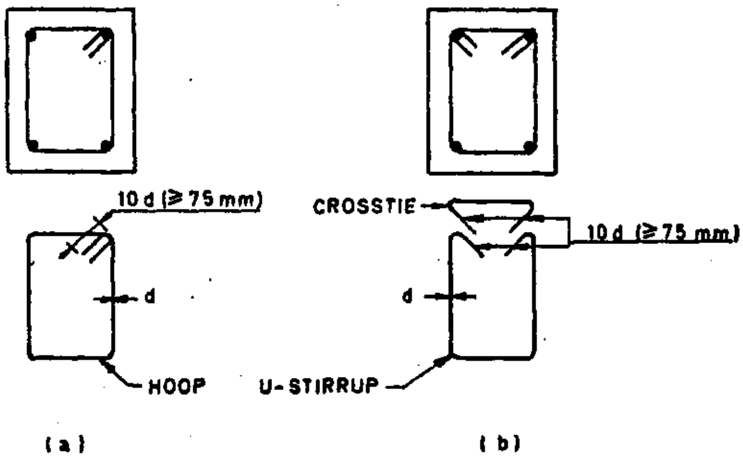


FIG. 3 BEAM WEB REINFORCEMENT

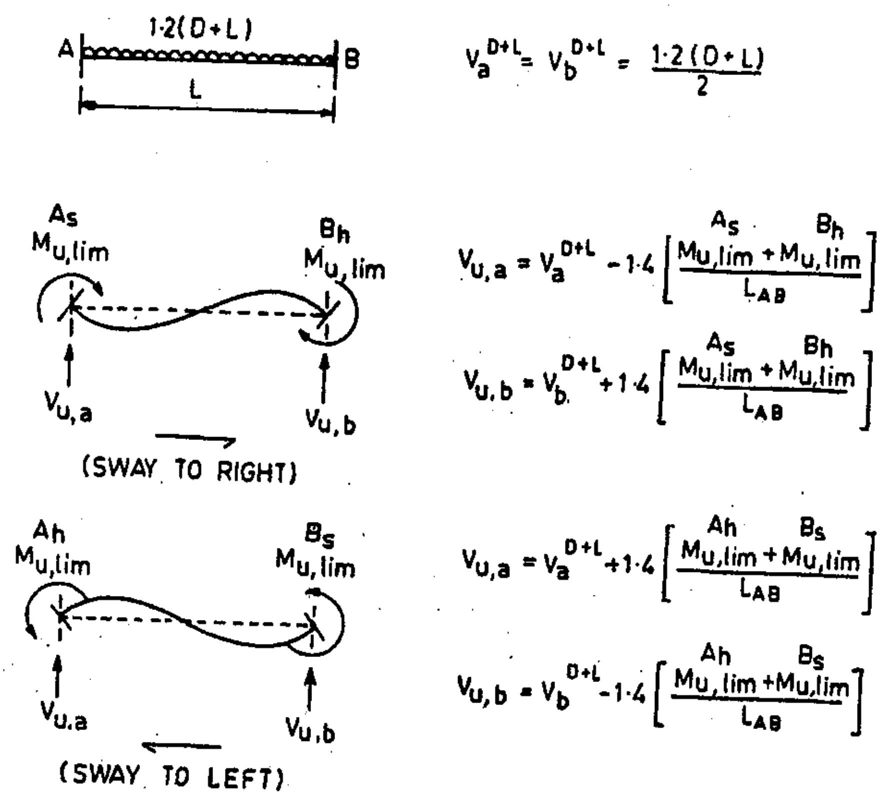


FIG. 4 CALCULATION OF DESIGN SHEAR FORCE FOR BEAM

6.3.4 The contribution of bent up bars and inclined hoops to shear resistance of the section shall not be considered.

6.3.5 The spacing of hoops over a length of 2d at either end of a beam shall not exceed (a) d/4, and (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm (*see* Fig. 5). The first hoop shall be at a distance not exceeding 50 mm from the joint face. Vertical hoops at the same spacing as above, shall also be provided over a length equal to 2d on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding d/2.

7 COLUMNS AND FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

7.1 General

7.1.1 These requirements apply to frame members which have a factored axial stress in excess of 0·1 f_{ck} under the effect of earthquake forces.

7.1.2 The minimum dimension of the member shall not be less than 200 mm. However, in frames which have beams with centre to centre span exceeding 5 m or columns of unsupported length exceeding 4 m, the shortest dimension of the column shall not be less than 300 mm.

7.1.3 The ratio of the shortest cross sectional dimension to the perpendicular dimension shall preferably not be less than 0·4.

7.2 Longitudinal Reinforcement

7.2.1 Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall be spliced at one section.

7.2.2 Any area of a column that extends more than 100 mm beyond the confined core due to architectural requirements, shall be detailed in the following manner. In case the contribution of this area to strength has been considered, then it will have the minimum longitudinal and transverse reinforcement as per this code.

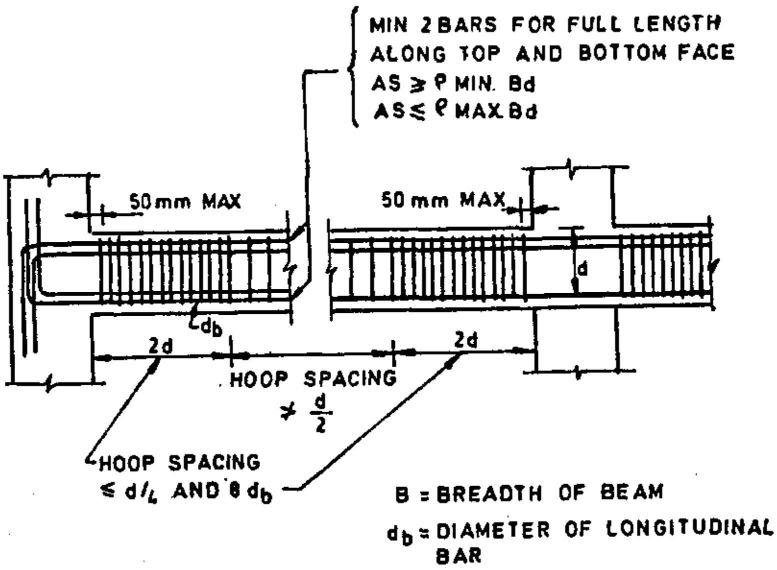


FIG. 5 BEAM REINFORCEMENT

However, if this area has been treated as non-structural, the minimum reinforcement requirements shall be governed by IS 456 : 1978 provisions minimum longitudinal and transverse reinforcement, as per IS 456 : 1978 (*see* Fig. 6).

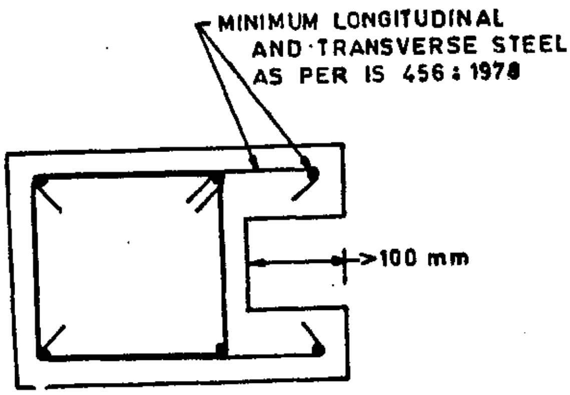


FIG. 6 REINFORCEMENT REQUIREMENT FOR COLUMN WITH MORE THAN 100 mm PROJECTION BEYOND CORE

7.3 Transverse Reinforcement

7.3.1 Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end, that is embedded in the confined core (*see* Fig 7A).

7.3.2 The parallel legs of rectangular hoops shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided (Fig. 7B). Alternatively, a pair of overlapping hoops may be provided within the columm (*see* Fig. 7C). The hooks shall engage peripheral longitudinal bars.

7.3.3 The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided, as per 7.4.

7.3.4 The design shear force for columns shall be the maximum of:

a) calculated factored shear force as per analysis, and

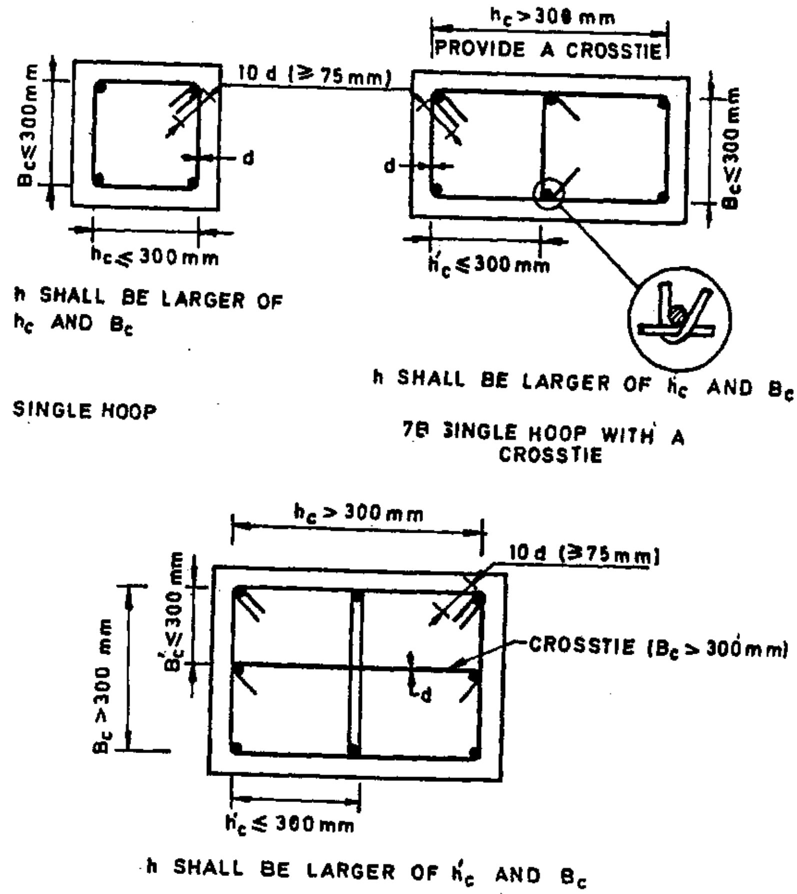
b) a factored shear force given by

$$V_u = 1.4\left[\frac{M^{bL}_{u,lim} + M^{bR}_{u,lim}}{h_{st}}\right]$$

where $M^{bL}_{u,lim}$ and $M^{bR}_{u,lim}$ are moment of resistance, of opposite sign, of beams framing into the column from opposite faces (*see* Fig. 8); and h_{st} is the storey height. The beam moment capacity is to be calculated as per IS 456 : 1978.

7.4 Special Confining Reinforcement

This requirement shall be met with, unless a larger amount of transverse reinforcement is required from shear strength considerations.



7A SINGLE HOOP

7B SINGLE HOOP WITH A CROSSTIE

7C OVERLAPPING HOOPS WITH A CROSSTIE

FIG. 7 TRANSVERSE REINFORCEMENT IN COLUMN

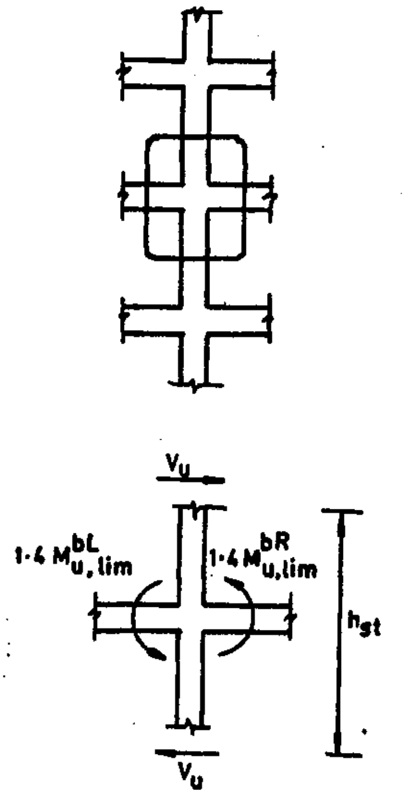
7.4.1 Special confining reinforcement shall be provided over a length l_0 from each joint face, towards midspan, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces (*see* Fig. 9.). The length 'l_0' shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) 1/6 of clear span of the member, and (c) 450 mm.

7.4.2 When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (*see* Fig. 10).

7.4.3 When the calculated point of contraflexure, under the effect of gravity and earthquake loads, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

7.4.4 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (*see* Fig. 11). This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

7.4.5 Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result



$$V_u = 1{\cdot}4\left(\frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}}\right)$$

FIG. 8 CALCULATION OF DESIGN SHEAR FORCE FOR COLUMN

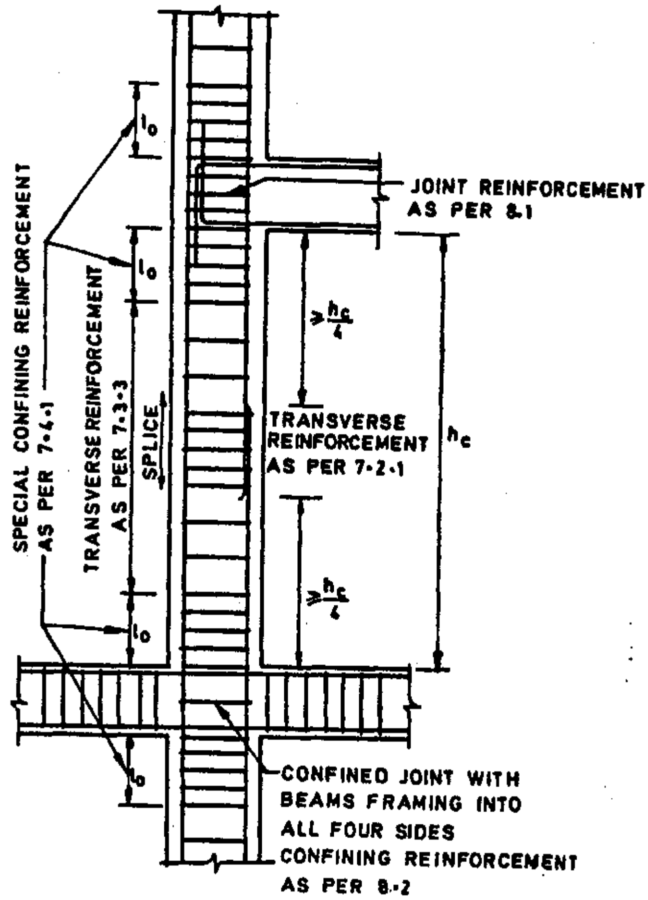


FIG. 9 COLUMN AND JOINT DETAILING

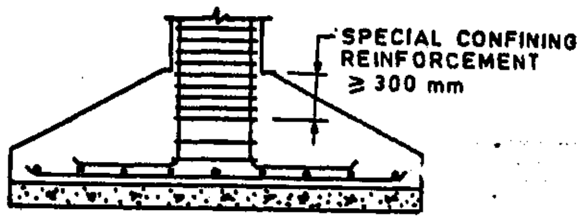


FIG. 10 PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTINGS

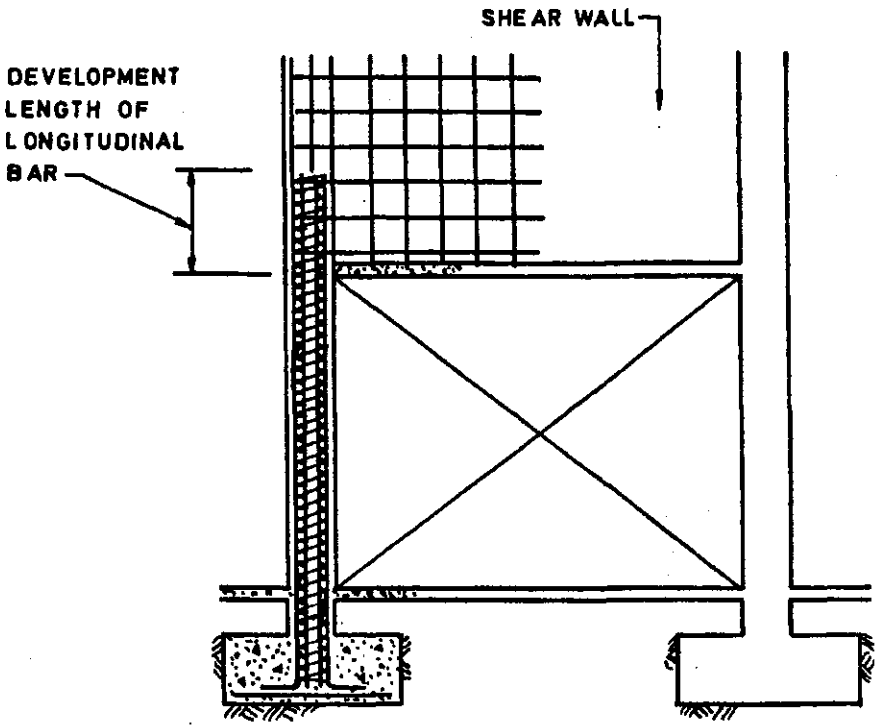


FIG. 11 SPECIAL CONFINING REINFORCEMENT REQUIREMENT FOR COLUMNS UNDER DISCONTINUED WALLS

due to the presence of bracing, a mezzanine floor or a R.C.C. wall on either side of the column that extends only over a part of the column height (*see* Fig. 12).

7.4.6 The spacing of hoops used as special confining reinforcement shall not exceed 1/4 of minimum member dimension but need not be less than 75 mm nor more than 100 mm.

7.4.7 The area of cross section, A_{sh}, of the bar forming circular hoops or spiral, to be used as special confining reinforcement, shall not be less than

$$A_{sh} = 0{\cdot}09\ S D_k \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1{\cdot}0\right]$$

where

- A_{sh} = area of the bar cross section,
- S = pitch of spiral or spacing of hoops,
- D_k = diameter of core measured to the outside of the spiral or hoop,
- f_{ck} = characteristic compressive strength of concrete cube,
- f_y = yield stress of steel (of circular hoop or spiral),
- A_g = gross area of the column cross section, and
- A_k = area of the concrete core $= \frac{\pi}{4} D_k^2$

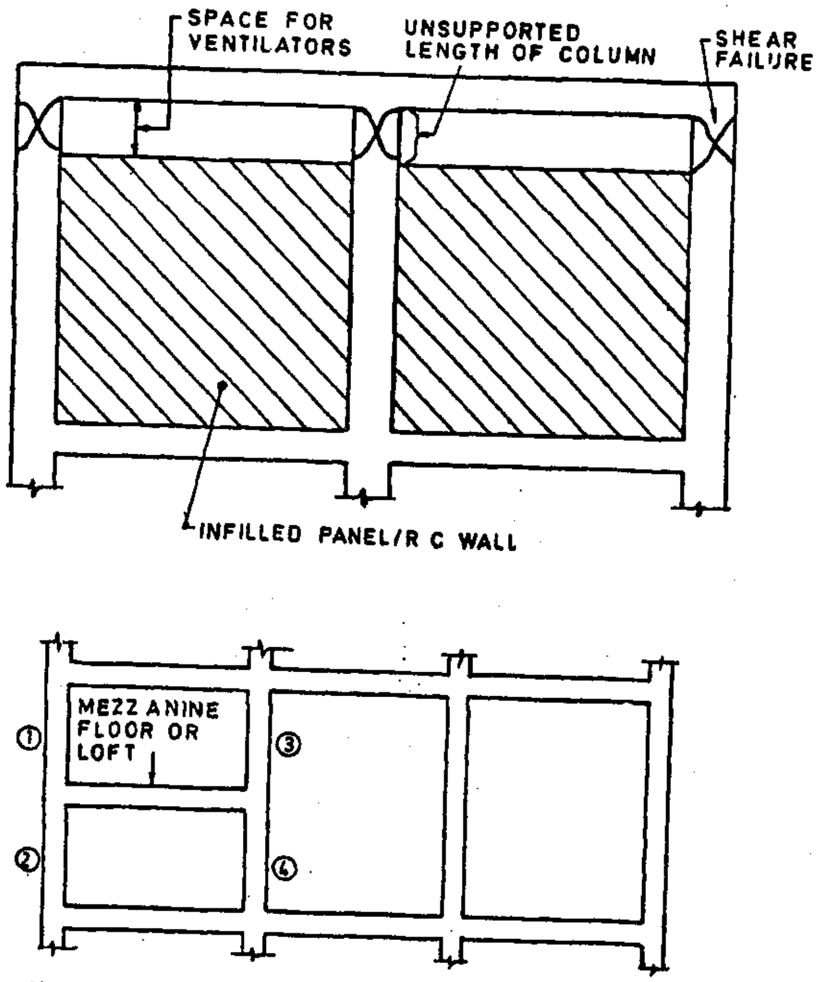
Example : Consider a column of diameter 300 mm. Let the grade of concrete be M20, and that of steel Fe 415, for longitudinal and confining reinforcement. The spacing of circular hoops, S, shall not exceed the smaller of (a) 1/4 of minimum member dimension = 1/4 × 300 = 75 mm, and (b) 100 mm. Therefore, S = 75 mm. Assuming 40 mm clear cover to the longitudinal reinforcement and circular hoops of diameter 8 mm, D_k = 300 − 2 × 40 + 2 × 8 = 236 mm. Thus, the area of cross section of the bar forming circular hoop works out to be 47·28 mm². This is less than the cross sectional area of 8 mm bar (50·27 mm²). Thus, circular hoops of diameter 8 mm at a spacing of 75 mm centre to centre will be adequate.

7.4.8 The area of cross section, A_{sh}, of the bar forming rectangular hoop, to be used as special confining reinforcement shall not be less than

$$A_{sh} = 0{\cdot}18\ S h \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1{\cdot}0\right]$$

where

h = longer dimension of the rectangular confining hoop measured to its outer



(1), 2), (3) and (4) relatively stiff columns — They attract parge seismic shear force.

FIG. 12 COLUMNS WITH VARYING STIFFNESS

face. It shall not exceed 300 mm (*see* Fig. 7), and

A_k = area of confined concrete core in the rectangular hoop measured to its outside dimensions.

NOTE : The dimension 'h' of the hoop could be reduced by introducing crossties, as shown in Fig. 7B. In this case, A_k shall be measured as the overall core area, regardless of the hoop arrangement. The hooks of crossties shall engage peripheral longitudinal bars.

Example : Consider a column of 650 mm × 500 mm. Let the grade of concrete be M20 and that of steel Fe 415, for the longitudinal and confining reinforcement. Assuming clear cover of 40 mm to the longitudinal reinforcement and rectangular hoops of diameter 10 mm, the size of the core is 590 mm × 440 mm. As both these dimensions are greater than 300 mm, either a pair of overlapping hoops or a single hoop with crossties, in both directions, will have to be provided. Thus, the dimension 'h' will be the larger of (i) 590/2 = 295 mm, and (ii) 440/2 = 220 mm. The spacing of hoops, S, shall not exceed the smaller of (a) 1/4 of minimum member dimensions = 1/4 × 500 = 125 mm, and (b) 100 mm. Thus, S = 100 mm. The area of cross section of the bar forming rectangular hoop works out to be 64·47 mm². This is less than the area of cross section of 10 mm bar (78·54 mm²). Thus, 10 mm diameter rectangular hoops at 100 mm c/c will be adequate. Similar calculations indicate that, as an alternative, one could also provide 8 mm diameter rectangular hoops at 70 mm c/c.

8 JOINTS OF FRAMES

8.1 The special confining reinforcement as required at the end of column shall be provided through the joint as well, unless the joint is confined as specified by **8.2**.

8.2 A joint which has beams framing into all vertical faces of it and where each beam width is at least 3/4 of the column width, may be provided with half the special confining reinforcement required at the end of the column. The spacing of hoops shall not exceed 150 mm.

9 SHEAR WALLS

9.1 General Requirements

9.1.1 The requirements of this section apply to the shear walls, which are part of the lateral force resisting system of the structure.

9.1.2 The thickness of any part of the wall shall preferably, not be less than 150 mm.

9.1.3 The effective flange width, to be used in the design of flanged wall sections, shall be assumed to extend beyond the face of the web for a distance which shall be the smaller of (a) half the distance to an adjacent shear wall web, and (b) 1/10 th of the total wall height.

9.1.4 Shear walls shall be provided with reinforcement in the longitudinal and transverse directions in the plane of the wall. The minimum reinforcement ratio shall be 0·002 5 of the gross area in each direction. This reinforcement shall be distributed uniformly across the cross section of the wall.

9.1.5 If the factored shear stress in the wall exceeds $0.25 \sqrt{f_{ck}}$ or if the wall thickness exceeds 200 mm, reinforcement shall be provided in two curtains, each having bars running in the longitudinal and transverse directions in the plane of the wall.

9.1.6 The diameter of the bars to be used in any part of the wall shall not exceed 1/10th of the thickness of that part.

9.1.7 The maximum spacing of reinforcement in either direction shall not exceed the smaller of $l_w/5$, 3 t_w, and 450 mm; where l_w is the horizontal length of the wall, and t_w is the thickness of the wall web.

9.2 Shear Strength

9.2.1 The nominal shear stress, τ_v, shall be calculated as:

$$\tau_v = \frac{V_u}{t_w \, d_w}$$

where

V_u = factored shear force,

t_w = thickness of the web, and

d_w = effective depth of wall section. This may by taken as 0·8 l_w for rectangular sections.

9.2.2 The design shear strength of concrete, τ_c, shall be calculated as per Table 13 of IS 456 : 1978.

9.2.3 The nominal shear stress in the wall, τ_v, shall not exceed $\tau_{c,max}$, as per Table 14 of IS 456 : 1978.

9.2.4 When τ_v is less than τ_c shear reinforcement shall be provided in accordance with **9.1.4** **9.1.5** and **9.1.7**.

9.2.5 When τ_v is greater than τ_c, the area of horizontal shear reinforcement, A_h, to be provided within a vertical spacing, S_v, is given by

$$V_{us} = \frac{0.87 f_y \, A_h \, d_w}{S_v}$$

where $V_{us} = (V_u - \tau_c \, t_w \, d_w)$, is the shear force to be resisted by the horizontal reinforcement. However, the amount of horizontal reinforcement provided shall not be less than the minimum, as per **9.1.4**.

9.2.6 The vertical reinforcement, that is uniformly distributed in the wall, shall not be less than the horizontal reinforcement calculated as per **9.2.5**.

9.3 Flexural Strength

9.3.1 The moment of resistance, M_{uv}, of the wall section may be calculated as for columns subjected to combined bending and axial load as per IS 456 : 1978. The moment of resistance of slender rectangular shear wall section with uniformly distributed vertical reinforcement is given in Annex A.

9.3.2 The cracked flexural strength of the wall section should be greater than its uncracked flexural strength.

9.3.3 In walls that do not have boundary elements, vertical reieforcement shall be concentrated at the ends of the wall. Each concentration shall consist of a minimum of 4 bars of 12 mm diameter arranged in at least 2 layers.

9.4 Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. Though they may have the same thickness as that of the wall web it is advantageous to provide them with greater thickness.

9.4.1 Where the extreme fibre compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds $0.2 f_{ck}$, boundaty elements shall be provided along the vertical boundaries of walls. The boundary elements may be discontinued where the calculated compressive stress becomes less than $0 \cdot 15 f_{ck}$. The compressive stress shall be calculated using a linearly elastic model and gross section properties.

9.4.2 A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry an axial compression equal to the sum of factored gravity load on it and the additional compressive load induced by the seismic force. The latter may be calculated as:

$$\frac{M_u - M_{uv}}{C_w}$$

where

M_u = factored design moment on the entire wall section,

M_{uv} = moment of resistance provided by distributed vertical reinforcement across the wall section, and

C_w = center to center distance between the boundary elements along the two vertical edges of the wall.

9.4.3 If the gravity load adds to the strength of the wall, its load factor shall be taken as 0·8.

9.4.4 The percentage of vertical reinforcement in the boundary elements shall not be less than 0·8 percent, nor greater than 6 percent. In order to avoid congestion, the practical upper limit would be 4 percent.

9.4.5 Boundary elements, where required, as per 9.4.1, shall be provided throughout their height, with special confining reinforcement, as per 7.4.

9.4.6 Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per 7.4.

9.5 Coupled Shear Walls

9.5.1 Coupled shear walls shall be connected by ductile coupling beams. If the earthquake induced shear stress in the coupling beam exceeds

$$\frac{0 \cdot 1\, l_s \sqrt{f_{ck}}}{D}$$

where l_s is the clear span of the coupling beam and D is its overall depth, the entire earthquake induced shear and flexure shall, preferably, be resisted by diagonal reinforcement.

9.5.2 The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shall be:

$$A_{sd} = \frac{V_u}{1 \cdot 74\, f_y \sin \alpha}$$

where V_u is the factored shear force, and α is the angle made by the diagonal reinforcement with the horizontal. At least 4 bars of 8 mm diameter shall be provided along each diagonal. The reinforcement along each diagonal shall be enclosed by special confining reinforcement, as per 7.4. The pitch of spiral or spacing of ties shall not exceed 100 mm.

9.5.3 The diagonal or horizontal bars of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1·5 times the development length in tension.

9.6 Openings in Walls

9.6.1 The shear strength of a wall with openings should be checked along critical planes that pass through openings.

9.6.2 Reinforcement shall be provided along the edges of openings in walls. The area of the vertical and horizontal bars should be such as to equal that of the respective interrupted bars. The vertical bars should extend for the full storey height. The horizontal bars should be provided with development length in tension beyond the sides of the opening.

9.7 Discontinuous Walls

Columns supporting discontinuous walls shall be provided with special confining reinforcement, as per 7.4.4.

9.8 Construction Joints

The vertical reinforcement ratio across a horizontal construction joint shall not be less than:

$$\frac{0 \cdot 92}{f_y}\left(\tau_v - \frac{P_u}{A_g}\right)$$

where τ_v is the factored shear stress at the joint, P_u is the factored axial force (positive for compression), and A_g is the gross cross sectional area of the joint.

9.9 Development, Splice and Anchorage Requirement

9.9.1 Horizontal reinforcement shall be anchored near the edges of the wall or in the confined core of the boundary elements.

9.9.2 Splicing of vertical flexural reinforcement should be avoided, as far as possible, in regions where yielding may take place. This zone of flexural yielding may be considered to extend for a distance of l_w above the base of the wall or one sixth of the wall height, whichever is more. However, this distance need not be greater than $2\, l_w$. Not more than one third of this vertical reinforcement shall be spliced at such a section. Splices in adjacent bars should be staggered by a minimum of 600 mm.

9.9.3 Lateral ties shall be provided around lapped spliced bars that are larger than 16 mm in diameter. The diameter of the tie shall not be less than one fourth that of the spliced bar nor less than 6 mm. The spacing of ties shall not exceed 150 mm center to center.

9.9.4 Welded splices and mechanical connections shall confirm to **25.2.5.2** of IS 456 : 1978. However, not more than half the reinforcement shall be spliced at a section, where flexural yielding may take place.

ANNEX A

(*Clause* 9.3.1)

MOMENT OF RESISTANCE OF RECTANGULAR SHEAR WALL SECTION

A-1 The moment of resistance of a slender rectangular shear wall section with uniformly distributed vertical reinforcement may be estimated as follows:

(a) For $x_u/l_w \leqslant x_u^* / l_w$,

$$\frac{M_{uv}}{f_{ck}\, t_w\, l_w^2} = \phi\left[\left(1 + \frac{\lambda}{\phi}\right)\left(\frac{1}{2} - 0.416\,\frac{x_u}{l_w}\right) - \left(\frac{x_u}{l_w}\right)^2\left(0.168 + \frac{\beta^2}{3}\right)\right]$$

where

$$\frac{x_u}{l_w} = \left(\frac{\phi + \lambda}{2\phi + 0.36}\right); \quad \frac{x_u^*}{l_w} = \left(\frac{0.003\,5}{0.003\,5 + 0.87\,f_y/E_s}\right);$$

$$\phi = \left(\frac{0.87\,f_y\,\rho}{f_{ck}}\right); \quad \lambda = \left(\frac{P_u}{f_{ck}\, t_w\, l_w}\right);$$

ρ = vertical reinforcement ratio = $A_{st}/(\,t_w\, l_w\,)$,

A_{st} = area of uniformly distributed vertical reinforcement,

β = $0.87\,f_y/(\,0.003\,5\,E_s\,)$,

E_s = elastic modulus of steel, and

P_u = axial compression on wall.

(b) For $x_u^* / l_w < x_u/l_w < 1.0$,

$$\frac{M_{uv}}{f_{ck}\, t_w\, l_w^2} = \alpha_1\left(\frac{x_u}{l_w}\right) - \alpha_2\left(\frac{x_u}{l_w}\right)^2 - \alpha_3 - \frac{\lambda}{2}$$

where

$$\alpha_1 = \left[0.36 + \phi\left(1 - \frac{\beta}{2} - \frac{1}{2\beta}\right)\right]$$

$$\alpha_2 = \left[0.15 + \frac{\phi}{2}\left(1 - \beta - \frac{\beta^2}{2} - \frac{1}{3\beta}\right)\right]; \text{ and } \alpha_3 = \frac{\phi}{6\beta}\left(\frac{1}{(x_u/l_w)} - 3\right)$$

The value of x_u/l_w to be used in this equation, should be calculated from the quadratic equation

$$\alpha_1\left(\frac{x_u}{l_w}\right)^2 + \alpha_4\left(\frac{x_u}{l_w}\right) - \alpha_5 = 0,$$

where

$$\alpha_4 = \left(\frac{\phi}{\beta} - \lambda\right); \text{ and } \alpha_5 = \left(\frac{\phi}{2\beta}\right).$$

These equations were derived, assuming a rectangular wall section of depth l_w and thickness t_w that is subjected to combined uni-axial bending and axial compression. The vertical reinforcement is represented by an equivalent steel plate along the length of the section. The stress-strain curve assumed for concrete is as per IS 456 : 1978 whereas that for steel is assumed to be bi-linear. Two equations are given for calculating the flexural strength of the section. Their use depends on whether the section fails in flexural tension or in flexural compression.

ANNEX B

(Foreword)

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(*Continued on page 16*)

(*Continued from page* 15)

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AMENDMENT NO. 1 NOVEMBER 1995
TO
IS 13920 : 1993 DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

[*Page* 3, *clause* 6.2.1(b)] — Substitute the following for the existing formula:

'$\rho \min = 0.24\sqrt{fck}/fy$

(CED 39)

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AMENDMENT NO. 2 MARCH 2002 TO IS 13920 : 1993 DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

(*Page* 1, *clause* 1.1.1) —Substitute the following for the existing:

'**1.1.1** Provisions of this code shall be adopted in all reinforced concrete structures which are located in seismic zone III, IV or V.'

(*Page* 3, *clause* 5.2, *line* 3) — Delete the word 'preferably'.

(*Page* 3, *clause* 5.3) — Insert the following at the end of the clause:

'However, high strength deformed steel bars, produced by the thermo-mechanical treatment process, of grades Fe 500 and Fe 550, having elongation more than 14.5 percent and conforming to other requirements of IS 1786 : 1985 may also be used for the reinforcement.'

(CED 39)

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3 **रूचि रखने वाले अन्य निकाय**

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प्रलेख संख्या	शीर्षक
सीईडी 39(7941)	भूकंपीय बल के प्रभाव के अंतर्गत प्रबलित कंकरीट संरचनाओं का तन्य विस्तार - रीति संहिता [आईएस 13920 का पहला पुनरीक्षण]

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DRAFT IN WIDE CIRCULATION

DOCUMENT DESPATCH ADVICE

Reference	Date
CED 39/T- 8	28 03 2014

TECHNICAL COMMITTEE: EARTHQUAKE ENGINEERING SECTIONAL COMMITTEE, CED 39

ADDRESSED TO :

1. Interested Members of Civil Engineering Division Council, CEDC
2. All members of CED 39
3. All others interested

Dear Sir,

Please find enclosed the following document:

Doc No.	Title
CED 39 (7941)	**Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces - Code of Practice (First Revision of IS 13920)**

Kindly examine the draft standard and forward your views stating any difficulties which you are likely to experience in your business or profession, if this is finally adopted as National Standard.

Last Date for comments: **31 05 2014**

Comments if any, may please be made in the format as given overleaf and mailed to the undersigned at sak.bis@nic.in .

In case no comments are received or comments received are of editorial nature, you will kindly permit us to presume your approval for the above document as finalized. However, in case of comments of technical in nature are received then it may be finalized either in consultation with the Chairman, Sectional Committee or referred to the Sectional Committee for further necessary action if so desired by the Chairman, Sectional Committee.

The document is also hosted on BIS website **www.bis.org.in**.

Thanking you,

Yours faithfully,

Sd/-

(J. Roy Chowdhury)
Sc 'F' & Head (Civil Engg.)

Encl: as above

Draft Indian Standard

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DUCTILE DESIGN AND DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

(*First Revision* of IS 13920)

ICS : 91.120.25

Earthquake Engineering Sectional Committee, CED 39

Last Date for Comments **31 May 2014**

FOREWORD

Formal clause will be added later.

After the formulation of IS 4326 'Code of Practice for Earthquake-Resistant Design and Construction of Buildings' which had provisions for addressing special features in the design and construction of earthquake-resistant RC buildings, and certain details for achieving ductility in reinforced concrete (RC) buildings; in order to keep abreast with the rapid developments and extensive research on earthquake-resistant design of RC structures, the technical committee prepared a separate standard for earthquake-resistant design and detailing of RC structures, namely IS 13920:1993.

The first edition (1993) of this standard incorporated some important provisions that were not covered in IS 4326:1976 for design of RC Structures. With the revision of IS 4326 as IS 4326:2013 (*third revision*), the revision of IS 13920 takes importance and hence, this standard addresses the following salient aspects:

(a) Significant experience was gained from performance of reinforced concrete structures (that were designed and detailed as per IS 4326) during past earthquakes. Many deficiencies were identified and corrected.

(b) Provisions on design and detailing of beams and columns as given in IS 4326 were revised with an aim to provide them with adequate stiffness, strength and ductility and to make them capable of undergoing extensive inelastic deformations and dissipating seismic energy in a stable manner.

(c) Specifications were included on lower limits for strengths of material of earthquake-resistant RC structural systems.

(d) Geometric constraints were imposed on cross-sections of flexural members. Provisions were revised on minimum and maximum reinforcement limits. Requirements were made explicit for detailing of longitudinal reinforcement in beams at joint faces, splices and anchorage requirements. Provisions were included for calculating seismic design shear force, and detailing transverse reinforcement in beams.

(e) For members subjected to axial load and bending moment, constraints were imposed on cross-sectional aspect ratio and on absolute dimensions. Also, provisions are included for (i) location of lap splices, (ii) calculation of seismic design shear force of structural walls, and (iii) special confining reinforcement in regions of columns that are expected to undergo cyclic inelastic deformations during a severe earthquake shaking.

(f) Specifications were included on a seismic design and detailing of reinforced concrete structural walls. These provisions assisted in (i) estimation of design shear force and bending moment demand on structural wall sections, (ii) estimation of design moment capacity of wall sections, (iii) detailing of reinforcement in the wall web, boundary elements, coupling beams, around openings, at construction joints, and (iv) providing sufficient length for development, lap splicing and anchorage of longitudinal steel.

Following the earthquakes that occurred after the release of IS 13920 in 1993 (especially the 1997 Jabalpur, 2001 Bhuj, 2004 Sumatra, 2006 Sikkim, and 2011 Sikkim earthquakes), it was felt that this standard needs further improvement.

In this first revision of IS 13920, the following changes are incorporated:

(a) The title is revised to reflect the "Design" provisions that existed and new ones added, that determine the sizing, proportioning and reinforcement in RC members meant to resist earthquake shaking.
(b) The following new provisions are added:
   (i) Column-to-beam strength ratio provision has been added in keeping with the strong column – weak beam design philosophy for moment resisting frames;
   (ii) Shear design of beam-column joints;
   (iii) Design of slender RC structural walls is improved. The principle of superposition is dropped for estimating the design moment of resistance of structural walls with boundary elements. Instead, procedure is mentioned for estimating the same.
(c) Most provisions that existed earlier have been redrafted. Also, the sequence of sections is re-organised for greater clarity to designers and for removing ambiguities.
(d) The name of the standard has been modified to reflect various provisions included that pertain to "design" of RC components participating in seismic resistance.

Further, while the common methods of design and construction have been covered in this standard for RC structural systems with moment resisting frames and RC structural systems with moment resisting frames and structural walls that participate in resisting earthquake force, design and construction of other lateral load resisting structural systems made of reinforced concrete but not covered by this standard, may be permitted by the approving agency or a committee constituted by the agency only on production of satisfactory evidence from experiments on prototype sub-assemblages and structures, and nonlinear analyses demonstrating their adequacy to resist earthquake shaking expected in the region where the structures are expected to be built. Such nonlinear analyses shall demonstrate that the collapse mechanism of the proposed structure is desirable and that the lateral deformation capacity of the structure is sufficient to resist the ground deformation imposed in the region where the structure is located. The committee of the approving agency shall comprise of competent engineers with the necessary experience and shall have the authority to review the data submitted, ask for additional data, tests and to frame special rules for such structural systems not covered under this standard.

In the formulation of this standard, due weightage has been given to the need for international coordination among standards prevailing in different seismic regions of the world. In the preparation of this standard, assistance has been derived from the following publications:

(i) ACI 318-11, Building code requirements for reinforced concrete and commentary, published by American Concrete Institute.
(ii) IBC 2003 International Building Code, published by International Code Council, Inc.

(iii) EN 1998-1:2003(E) Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, Brussels.
(iv) NZS 3102:Part 1:1995 Concrete structure standard, published by Standards Council, New Zealand.

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 'Rules for rounding off numerical values (*Revised*)'. The number of significant places retained in the rounded off value should be same as that of the specified value in this standard.

Draft Indian Standard

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DUCTILE DESIGN AND DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

(*First Revision* of IS 13920)

ICS : 91.120.25

Earthquake Engineering Sectional Committee, CED 39	Last Date for Comments **31 May 2014**

1. SCOPE

1.1 This standard covers the requirements for designing and detailing of members of reinforced concrete (RC) structures designed to resist lateral effects of earthquake shaking, so as to give them adequate stiffness, strength and ductility to resist severe earthquake shaking without collapse. Even though the general concepts adopted in this standard for structures are applicable for RC bridge systems, provisions of this standard shall be taken only as a guide for RC bridge piers and wells of large cross-sections, but are not sufficient. This standard addresses lateral load resisting structural systems of RC structures composed of:

(a) RC *Moment Resisting Frames,*
(b) RC *Moment Resisting Frames with Unreinforced Masonry Infill Walls*; and
(c) RC *Moment Resisting Frames with RC Structural Walls.*
(d) RC Structural Walls

1.1.1 Provisions of this standard shall be adopted in all lateral load resisting systems of RC structures located in Seismic Zone III, IV or V.

1.1.2 The provisions for RC structures given herein apply specifically to monolithic RC construction, and not for precast RC structures. Precast and/or prestressed concrete members may be used, only if they are designed to provide similar level of ductility as that of monolithic RC structures during or after an earthquake. Specialist literature must be referred to for design and construction of such structures. The adequacy of such designs shall be demonstrated by adequate, appropriate experimentation and nonlinear dynamic structural analyses.

1.1.3 RC monolithic members assumed not to participate in the lateral force resisting system (**3.7**) shall be permitted provided that their effect on the seismic response of the system is accounted for. Consequence of failure of structural and non-structural members not part of the lateral force resisting system shall also be considered in design.

2. REFERENCES

2.1 The standards listed below contain provision 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 agreement based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated therein.

IS No.	Title
456 : 2000	Code of Practice for Plain and Reinforced Concrete (*Fourth Revision*)
800 : 2007	Code of Practice for Structural Steel
1343 : 2012	Code of Practice for Prestressed Concrete Structures (*Second Revision*)
1786 : 2008	Specification for High Strength Deformed Steel Bars and Wires for Concrete Reinforcement (*Fourth Revision*)
1893 (Part 1) : 2002	Criteria for Earthquake Design of Structures (*Fifth Revision*)
xxxxx : 201x	Specification for Reinforcement couplers for mechanical splices of bars for concrete reinforcement *(under print)*

3. TERMINOLOGY

3.0 For the purpose of this standard, the following definitions shall apply.

3.1 Beams
These are horizontal members of the moment resisting frames with flexural and shearing actions.

3.2 Boundary Elements
These are portions along the ends of a structural wall that are strengthened by longitudinal and transverse reinforcement. They may have the same thickness as that of the wall web.

3.3 Columns
These are vertical members of the moment resisting frames with axial, flexural and shearing actions.

3.4 Cover Concrete
It is that concrete which is not confined by transverse reinforcement.

3.5 Cross-tie
It is a continuous bar having a 135° hook with an extension of 6 times diameter (but not < 75 mm) at one end and a hook not less than 90° with an extension of 6 times diameter (but not < 65 mm) at the other end. The hooks shall engage peripheral longitudinal bars. In general, the 90° hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

3.6 Gravity Columns in Buildings
It is a column, which is not part of the lateral load resisting system and designed only for force actions (*i.e.*, axial force, shear force and bending moments) due to gravity loads. But, it should be able to resist the gravity loads at lateral displacement imposed by the earthquake forces.

3.7 Lateral Force Resisting System
It is that part of the structural system which participates in resisting forces induced by earthquake.

3.8 Moment-Resisting Frame
It is a three-dimensional structural system composed of interconnected members, without structural walls, so as to function as a complete self-contained unit with or

without the aid of horizontal diaphragms or floor bracing systems, in which the members resist gravity and lateral forces primarily by flexural actions.

3.8.1 Special Moment Resisting Frame (SMRF)
It is a moment-resisting frame specially detailed to provide ductile behaviour as per the requirements specified in **5**, **6**, **7** and **8** of this standard.

3.8.2 Ordinary Moment Resisting Frame (OMRF)
It is a moment-resisting frame not meeting special detailing requirements for ductile behaviour specified in this standard.

3.9 Stirrup
It is a single steel bar bent into a closed loop having a 135° hook with an extension of 6 times diameter (but not < 65 mm) at each end, which is embedded in the confined core of the section, and placed normal to the longitudinal axis of the RC beam or column.

3.10 Structural Wall
It is a vertically oriented planar element that is primarily designed to resist lateral force effects (axial force, shear force and bending moment) in its own plane. It is commonly known as *Shear Wall*.

3.11 Special Structural Wall
It is a structural wall meeting special detailing requirements for ductile behaviour specified in **10**.

4. SYMBOLS

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. All dimensions are in mm, loads in Newton and stresses in MPa, unless otherwise specified.

A_e – effective cross sectional area of a joint

A_{ej} – effective shear area of a joint

A_g – gross cross sectional area of column / wall

A_h – horizontal reinforcement area within spacing S_v

A_k – area of concrete core of column

A_{sd} – reinforcement along each diagonal of coupling beam

A_{sh} – area of cross section of bar forming spiral or stirrup

A_{st} – area of uniformly distributed vertical reinforcement

A_v – vertical reinforcement at a joint

b_b – width of beam

b_c – width of column

b_j – effective width of a joint

D – overall depth of beam

D_k – diameter of column core measured to the outside of spiral or stirrup

d – effective depth of member

d_w – effective depth of wall section

E_s – elastic modulus of steel

f_{ck} – characteristic compressive strength of concrete cube (in MPa)

f_y – yield stress of steel reinforcing bars, or 0.2% proof strength of steel reinforcing steel (in MPa)

h – longer dimension of rectangular confining stirrup measured to its outer fa ce

h_c – depth of column

h_j – effective depth of a joint

h_{st} – clear storey height

h_w – overall height of RC structural wall

L_{AB} – clear span of beam

L_d – development length of bar in tension

L_o – length of member over which special confining reinforcement is to be provided

L_w – horizontal length of wall

L_s – clear span of couplings beam

M_u – design moment of resistance of entire RC beam, column or wall section

M_{c1} – Design moment of resistance of column section

M_{c2} – Design moment of resistance of column section

M_{g1} – Design moment of resistance of beam section

M_{g2} – Design moment of resistance of beam section

M_u^{Ah} – Hogging design moment of resistance of beam at end A

M_u^{As} – Sagging design moment of resistance of beam at end A

M_u^{Bh} – Hogging design moment of resistance of beam at end B

M_u^{Bs} – Sagging design moment of resistance of beam at end B

M_u^{BL} – Design moment of resistance of beam framing into column from the left

M_u^{BR} – Design moment of resistance of beam framing into column from the right

M_{uw} – Design moment of resistance of web of RC structural wall alone

P_u – Factored axial load

s_v – Spacing of stirrups along the longitudinal direction of beam or column

t_w – Thickness of web of RC structural wall

${V_{u,a}}^{D+L}$ – Factored shear force demand at end A of beam due to dead and live loads

${V_{u,b}}^{D+L}$ – Factored shear force demand at end B of beam due to dead and live loads

V_j – Design shear resistance of a joint

V_u – Factored shear force

V_{us} – *Design* shear resistance offered at a section by steel stirrups

x_u, x_u^* – Depth of neutral axis from extreme compression fibre

α – Inclination of diagonal reinforcement in coupling beam

ρ – Area of longitudinal reinforcement as a fraction of gross area of cross-section in a RC beam, column or structural wall

ρ_c – Area of longitudinal reinforcement on the compression face of a beam as a fraction of gross area of cross-section

$(\rho_h)_{min}$ – Minimum area of horizontal reinforcement of a structural wall as a fraction of gross area of cross-section

$(\rho_{v,be})_{min}$ – Minimum area of vertical reinforcement in each boundary element of a structural wall as a fraction of gross area of cross-section of each boundary element

$(\rho_{v,net})_{min}$ – Minimum area of vertical reinforcement of a structural wall as a fraction of gross area of cross-section of the wall

$(\rho_{v,web})_{min}$ Minimum area of vertical reinforcement in web of a structural wall as a fraction of gross area of cross-section of web

ρ_{max} – Maximum area of longitudinal reinforcement permitted on the tension face of a beam as a fraction of gross area of cross-section

ρ_{min} – Minimum area of longitudinal reinforcement to be ensured on the tension face of a beam as a fraction of gross area of cross-section

τ_c – Design shear strength of concrete

$\tau_{c,max}$ – Maximum nominal shear stress permitted at a section of RC beam, column or structural wall

τ_v – Nominal shear stress at a section of RC beam, column or structural wall

5. GENERAL SPECIFICATIONS

5.1 The design and construction of reinforced concrete buildings shall be governed by provisions of IS 456, except as modified by the provisions of this standard for those elements participating in lateral force resistance.

5.2 Minimum grade of structural concrete shall be M20, but M25 for buildings

(i) more than 15m in height in Seismic Zones III, IV and V,
(ii) but not less than that required by IS 456 based on exposure conditions.

5.3 Steel reinforcement resisting earthquake-induced forces in RC frame members and in boundary elements of RC structural walls shall comply with **5.3.1**, **5.3.2** and **5.3.3.**

5.3.1 Steel reinforcements used shall be

(i) of grade Fe 415 (conforming to IS 1786), and
(ii) of grade Fe 500 and Fe 550, i.e., high strength deformed steel bars produced by thermo-mechanical treatment process having elongation more than 14.5 percent, and conforming to IS 1786.

5.3.2 The actual 0.2% proof strength of steel bars based on tensile test must not exceed their characteristic 0.2% proof strength by more than 20%.

5.3.3 The ratio of the actual ultimate strength to the actual 0.2% proof strength shall be at least 1.25.

5.4 In RC frame buildings, lintel beams shall preferably not be integrated into the columns to avoid short column effect. When integrated, they shall be included in the analytical model for structural analysis. Similarly, plinth beams (where provided), and staircase beams and slabs framing into columns shall be included in the analytical model for structural analysis.

5.5 RC regular moment-resisting frame buildings shall have planar frames oriented along the two principal plan directions of buildings. Irregularities listed in IS 1893 (Part 1) shall be avoided. Buildings with any of the listed irregularities perform poorly during earthquake shaking; in addition, buildings with floating columns and set-back columns also perform poorly. When any such irregularities are adopted, detailed nonlinear analyses shall be performed to demonstrate that there is no threat to loss of life and property.

6. BEAMS

6.1 General

Requirements of this section shall apply to beams resisting earthquake-induced effects, in which the factored axial compressive stress due to gravity and earthquake effects does not exceed $0.08f_{ck}$. Beams, in which the factored axial compressive stress exceeds $0.08f_{ck}$, shall be designed as per requirements of 7.

6.1.1 Beams shall preferably have width-to-depth ratio of more than 0.3.

6.1.2 Beams shall not have width less than 200 mm.

6.1.3 Beams shall not have depth D more than 1/4th of clear span. This may not apply to the floor beam of frame staging of elevated RC water tanks.

6.1.4 Width of beam b_w shall not exceed the width of supporting member c_2 plus distance on either side of supporting member equal to the smaller of (a) and (b)

(a) Width of supporting member c_2
(b) 0.75 times breadth of supporting member c_1 (*see* Fig. 1a and 1b)

Transverse reinforcement for the width of a beam that exceeds width of the column c_2 shall be provided as shown in Fig. 1b throughout the beam span including within the beam column joint.

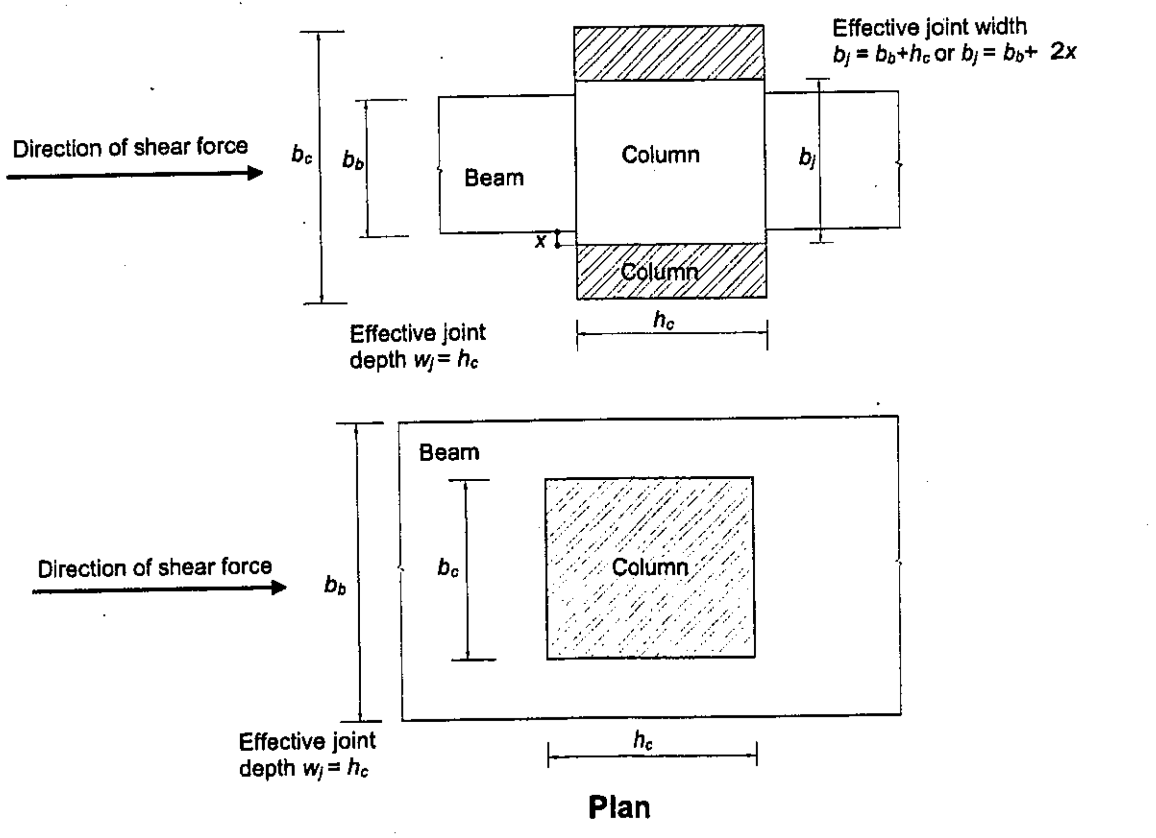

FIG. 1(A) PLAN VIEW OF A BEAM COLUMN JOINT SHOWING EFFECTIVE BREADTH AND WIDTH OF JOINT

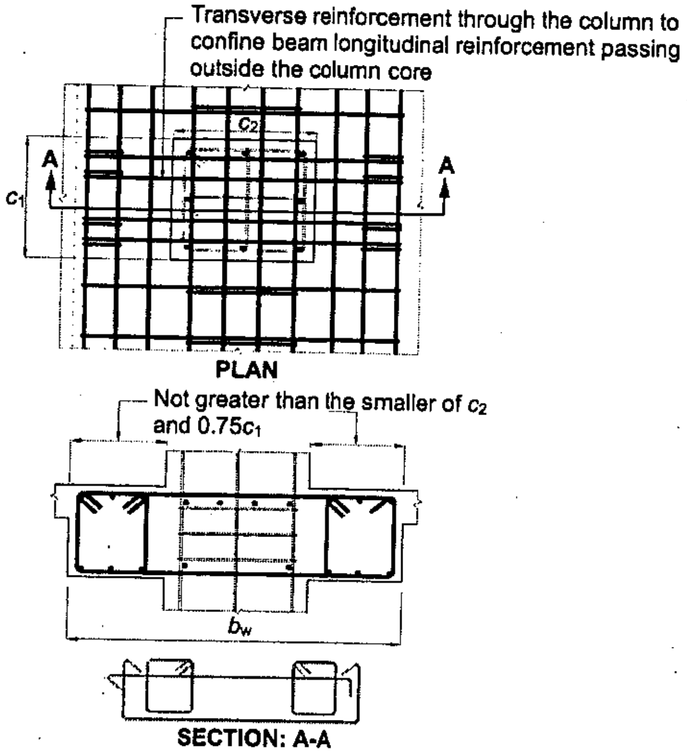

FIG. 1(B) MAXIMUM EFFECTIVE WIDTH OF WIDE BEAM AND REQUIRED TRANSVERSE REINFORCEMENT

6.2 Longitudinal Reinforcement

6.2.1 (a) Beams shall have at least two 12 mm diameter bars each at the top and bottom faces.

(b) Minimum longitudinal steel ratio ρ_{min} required on any face at any section is

$$\rho_{\min} = 0.24\frac{\sqrt{f_{ck}}}{f_y}.$$

6.2.2 Maximum longitudinal steel ratio ρ_{max} provided on any face at any section is 0.025.

6.2.3 Longitudinal steel on bottom face of a beam framing into a column (at the face of the column) shall be at least half the steel on its top face at the same section. At exterior joints, the anchorage length calculation shall consider this bottom steel to be tension steel.

6.2.4 Longitudinal steel in beams at any section on top or bottom face shall be at least 1/4th of longitudinal steel provided at the top face of the beam at the face of the column; when the top longitudinal steel in the beam at the two supporting column faces is different, the larger of the two shall be considered.

6.2.5 At an exterior joint, top and bottom bars of beams shall be provided with anchorage length beyond the inner face of the column, equal to development length of the bar in tension plus 10 times bar diameter minus the allowance for 90° bends (*see* Fig. 2); here,

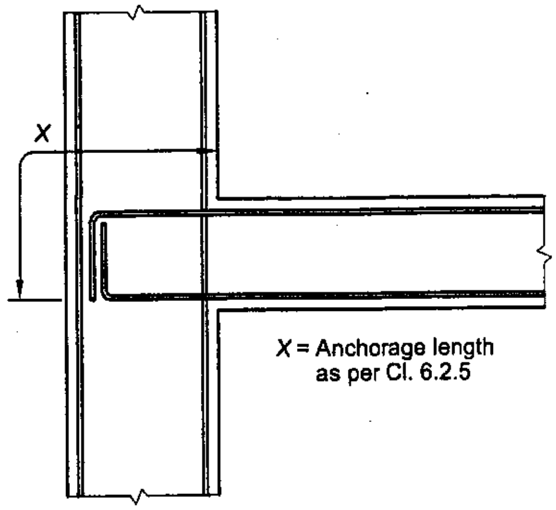

FIG. 2 ANCHORAGE OF LONGITUDINAL BEAM BARS AT EXTERIOR BEAM-COLUMN JOINT

6.2.6 Splicing of Longitudinal Bars

6.2.6.1 *Lap Splices*

When adopted, closed stirrups shall be provided over the entire length over which the longitudinal bars are spliced and,

(a) the spacing of these stirrups shall not exceed 150 mm (*see* Fig. 3).
(b) the lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
(c) lap splices shall not be provided
 (i) within a joint,
 (ii) within a distance of 2*d* from face of the column, and
 (iii) within a quarter length of the beam adjoining the location where flexural yielding may occur under earthquake effects.
(d) not more than 50% of area of steel bars on either top or bottom face shall be spliced at any one section.

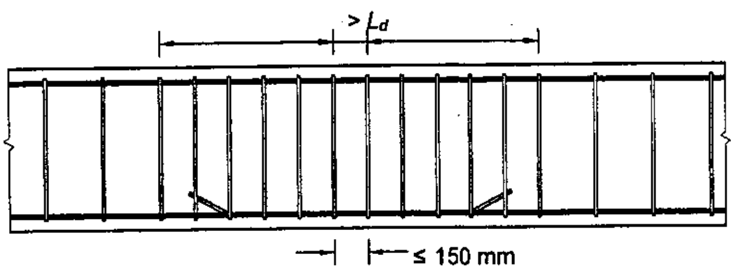

FIG. 3 LAP LENGTH AT LOCATION OF SPLICING OF LONGITUDINAL BARS IN BEAM

6.2.6.2 *Mechanical Couplers*

Mechanical couplers [conforming to IS xxxxx (*under print*)] shall be used when longitudinal steel bars have to be continued for beam spans larger than their manufacture lengths. Further,

(a) only those mechanical splices conforming to the above standard and capable of developing the specified tensile strength of spliced bar shall be permitted within a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place

(b) the spacing between adjacent longitudinal bars shall be based also on the outer size of the coupler to allow easy flow of concrete.

6.2.6.3 *Welded Splices*

Welded splices shall not be used in beams for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place at any location, not more than 50% of area of steel bars shall be spliced at any one section.

Welding of stirrups, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone.

6.3 Transverse Reinforcement

6.3.1 Only vertical stirrups shall be used in beams (*see* Fig. 4a); inclined stirrups shall not be used.

(a) In normal practice, a stirrup is made of a single bent bar. But, it may be made of two bars also, namely a U-stirrup with a 135° hook with an extension of 6 times diameter (but not less than 65 mm) at each end, embedded in the core concrete, and a cross-tie (*see* Fig. 4b).

(b) The hooks of the stirrups and cross-ties shall engage around peripheral longitudinal bars. Consecutive crossties engaging the same longitudinal bars shall have their 90° hooks at opposite sides of the beam. When the longitudinal reinforcement bars are secured by cross-ties in beams that have a slab on one side, the 90° hooks of the cross-ties shall be placed on that side.

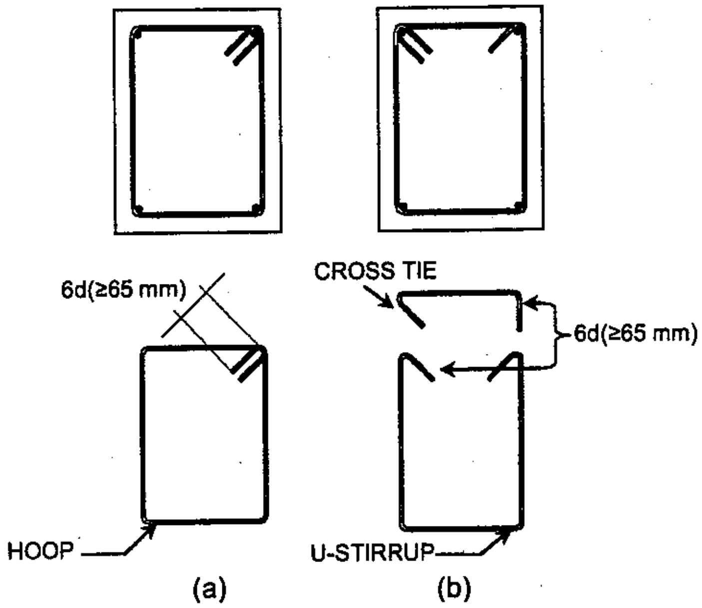

FIG. 4 DETAILS OF TRANSVERSE REINFORCEMENT IN BEAMS

6.3.2 The minimum diameter of a stirrup shall be 8 mm.

6.3.3 Shear force capacity of a beam shall be more than larger of:
(a) Factored shear force as per linear structural analysis, and
(b) Factored gravity shear force, plus equilibrium shear force when plastic hinges are formed at both ends of the beam (*see* Fig. 5) given by.

(i) For sway to right:

$$V_{u,a} = V_{u,a}^{D+L} - 1.4\ \frac{M_u^{As} + M_u^{Bh}}{L_{AB}} \quad \text{and}$$

$$V_{u,b} = V_{u,b}^{D+L} + 1.4\ \frac{M_u^{As} + M_u^{Bh}}{L_{AB}}.$$

(ii) For sway to left:

$$V_{u,a} = V_{u,a}^{D+L} - 1.4\ \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}} \quad \text{and}$$

$$V_{u,b} = V_{u,b}^{D+L} + 1.4\ \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$$

where M_u^{As}, M_u^{Ah}, M_u^{Bs} *and* M_u^{Bh} are sagging and hogging moments of resistance of the beam section at ends *A* and *B*, respectively. These shall be calculated as per IS 456. L_{AB} is clear span of beam. $V_{u,a}^{D+L}$ and $V_{u,b}^{D+L}$ are the factored shear forces at ends *A* and *B*, respectively, due to vertical loads acting on the span; the partial safety factor for dead and live loads shall be 1.2, and the beam shall be considered to be simply supported for this estimation.

The design shear force demand at end *A* of the beam shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear force demand at end *B* shall be the larger of the two values of $V_{u,b}$ computed above.

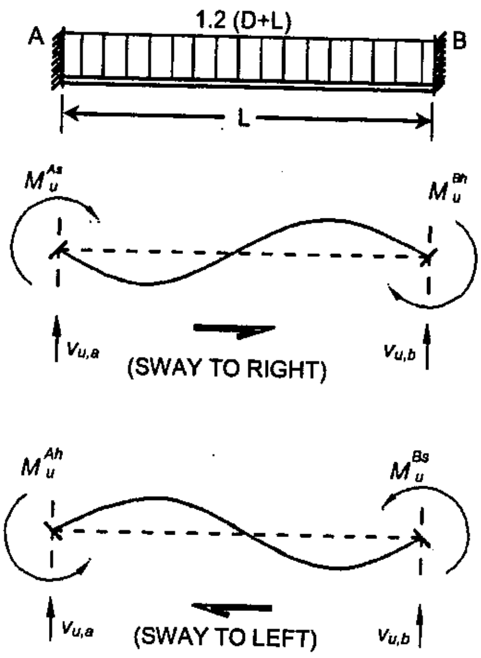

FIG. 5 CALCULATION OF DESIGN SHEAR FORCE DEMAND ON BEAMS UNDER PLASTIC HINGE ACTION AT THEIR ENDS

6.3.4 In the calculation of design shear force capacity of RC beams, contributions of the following shall NOT be considered:
(a) bent up bars ,
(b) inclined stirrups, and
(c) concrete in the RC section.

6.3.5 Close Spacing of Stirrups

Spacing of stirrups over a length of 2d at either end of a beam shall not exceed

(a) d/4,
(b) 8 times the diameter of the smallest longitudinal bar; and
(c) 100 mm (*see* Fig. 6).

6.3.5.1 The first stirrup shall be at a distance not exceeding 50 mm from the joint face.

6.3.5.2 Stirrups shall be provided over a length equal to 2d on either side of a section where flexural yielding may occur under earthquake effects. Over the remaining length of the beam, vertical stirrups shall be provided at a spacing not exceeding d/2.

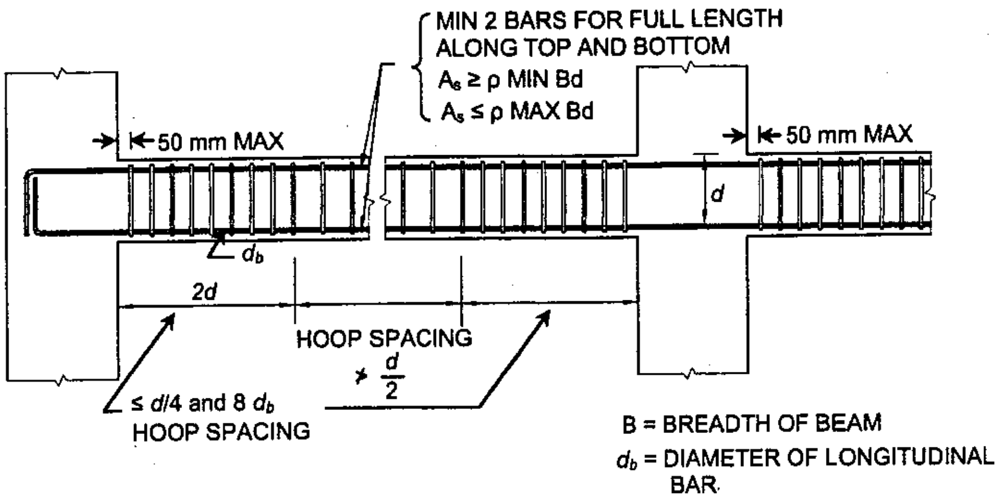

FIG. 6 DETAILS OF TRANSVERSE REINFORCEMENT IN BEAMS

7. COLUMNS AND INCLINED MEMBERS

7.1 Geometry

Requirements of this section shall apply to columns and inclined members resisting earthquake-induced effects, in which the factored axial compressive stress due to gravity and earthquake effects exceeds $0.08f_{ck}$.
The factored axial compressive stress considering all load combinations relating to seismic loads shall be limited to $0.40f_{ck}$ in all such members, except in those covered under **10**.

7.1.1 The minimum dimension of a column shall not be less than

(a) $20d_b$, where d_b is diameter of the largest diameter longitudinal reinforcement bar in the beam passing through or anchoring into the column at the joint, or
(b) 300 mm (*see* Fig. 7).

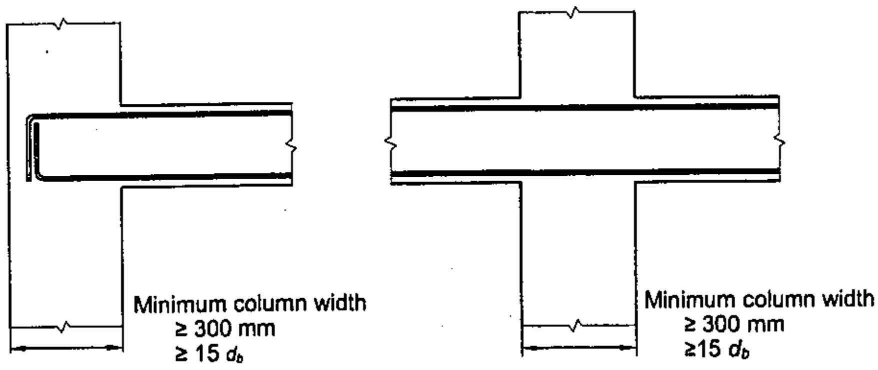

FIG. 7 MINIMUM SIZE OF RC COLUMNS BASED ON DIAMETER OF LARGEST LONGITUDINAL REINFORCEMENT BAR IN BEAMS FRAMING INTO IT

7.1.2 The cross-section aspect ratio (i.e., ratio of smaller dimension to larger dimension of the cross section of a column or inclined member) shall not be less than 0.4. Vertical members of RC buildings whose cross-section aspect ratio is less than 0.4 shall be designed as per requirements of **9**.

7.2 Relative Strengths of Beams and Columns at a Joint

7.2.1 At each beam-column joint of a moment-resisting frame, the sum of nominal design strength of columns meeting at that joint (with nominal strength calculated for the factored axial load in the direction of the lateral force under consideration so as to give least column nominal design strength) along each principal plane shall be at least 1.4 times the sum of nominal design strength of beams meeting at that joint in the same plane (*see* Fig. 8).

In the event of a beam-column joint not conforming to above, the columns at the joint shall be considered to be gravity columns only and shall not be considered as part of the lateral load resisting system.

7.2.1.1 The design moments of resistance of a beam shall be estimated based on the principles of mechanics and the limiting strain states of the limit state design method enunciated in IS 456. The design moment of resistance of a column shall be estimated as in case of beams corresponding to zero axial force on the design *P-M* interaction diagram.

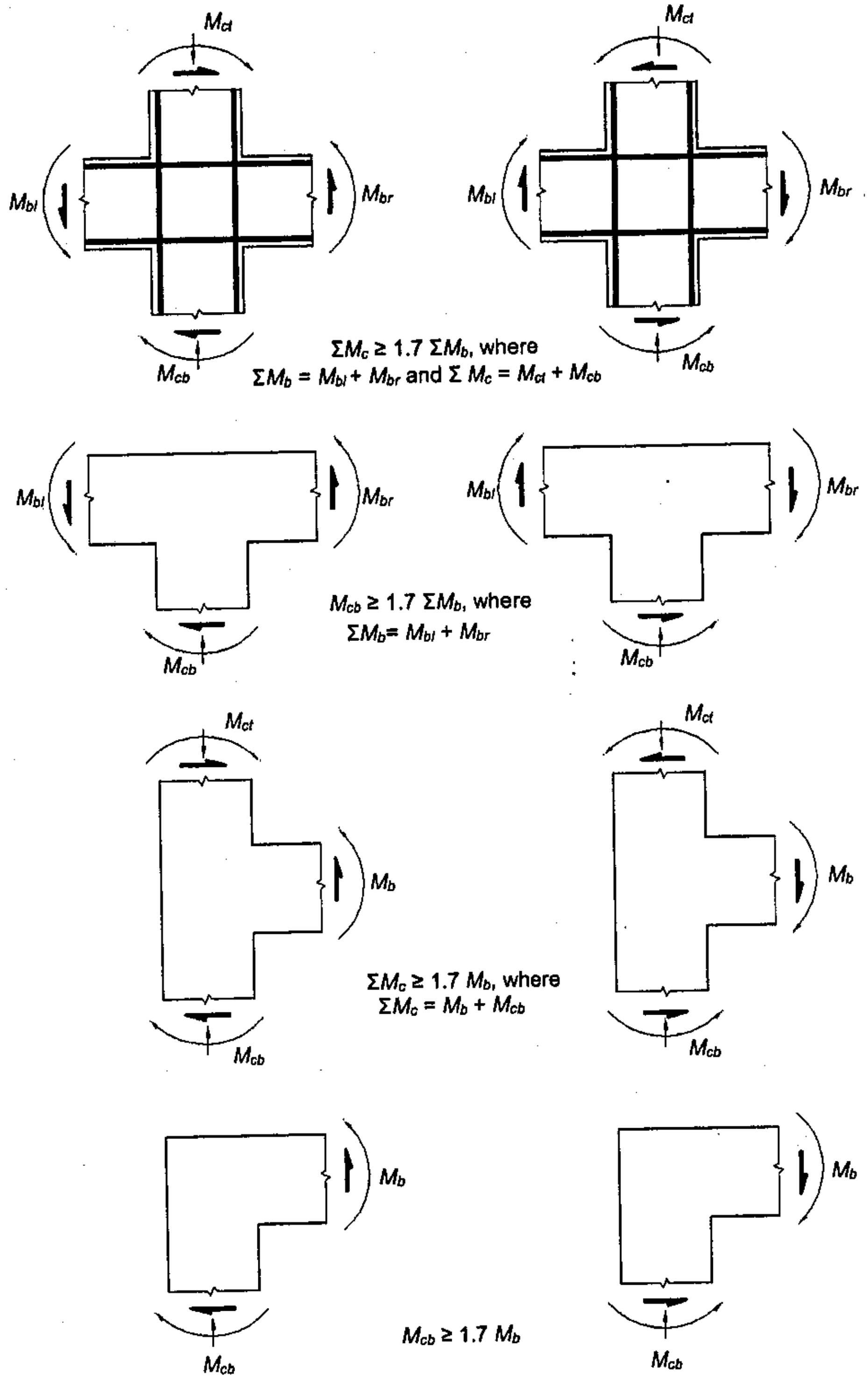

FIG. 8 STRONG COLUMN – WEAK BEAM REQUIREMENT

7.2.1.2 This check shall be performed at each joint for both positive and negative directions of shaking in the plane under consideration. Further, in this check, design moments of resistance in beam(s) meeting at a joint shall be considered in the same direction, and similarly the design moments of resistance of column(s) at the same joint shall be considered to be in the direction opposite to that of the moments in the beams.

7.2.1.3 This check shall be waived at all joints at roof level only, in buildings more than 4 storeys tall.

7.3 Longitudinal Reinforcement

7.3.1 Circular columns shall have minimum of 6 bars.

7.3.2 Splicing of Longitudinal Bars

7.3.2.1 Lap Splices

When adopted, closed stirrups shall be provided over the entire length over which the longitudinal bars are spliced.

(a) The spacing of these stirrups shall not exceed 150 mm.
(b) The lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
(c) Lap splices shall be provided only in the central half of clear column height, and not
  (i) within a joint, or
  (ii) within a distance of 2*d* from face of the beam.
(d) Not more than 50% of area of steel bars shall be spliced at any one section.

7.3.2.2 *Mechanical couplers*

Mechanical couplers [conforming to IS xxxxx (*under print*)] shall be used. Further, only those mechanical splices conforming to the above standard and capable of developing the specified tensile strength of spliced bar shall be permitted within a distance equal to two times the depth of the member from the column face or in any location where yielding of reinforcement is likely to take place

7.3.2.3 *Welded Splices*

Welded splices shall not be used in columns for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place. At any location, not more than 50% of area of steel bars shall be spliced at any one section.

But, welding of stirrups, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone.

7.3.3 A column that extends more than 100 mm beyond the confined core owing to architectural requirement (*see* Fig. 9).shall be detailed in the following manner.

(a) When the contribution of this area is considered in the estimate of strength of columns, it shall have at least the minimum longitudinal and transverse reinforcement given in this standard.
(b) When the contribution of this area is NOT considered in the estimate of strength of columns, it shall have at least the minimum longitudinal and transverse reinforcement given in IS 456.

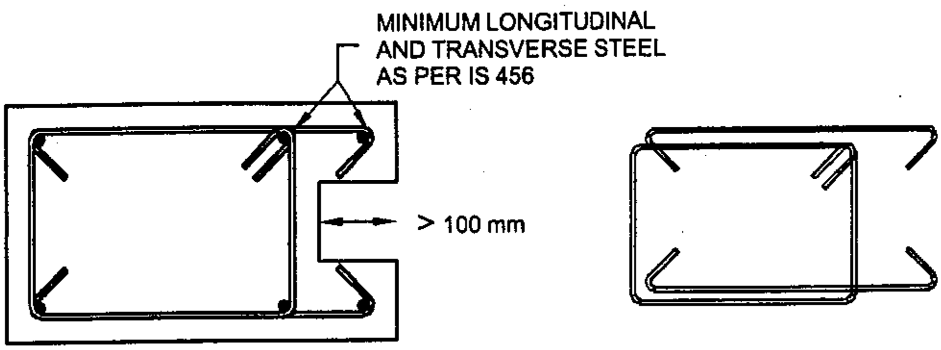

FIG. 9 REINFORCEMENT REQUIREMENT IN COLUMNS WITH PROJECTION MORE THAN 100 MM BEYOND CORE

7.4 Transverse Reinforcement

7.4.1 Transverse reinforcement shall consist of closed loop

(a) spiral or circular stirrups in circular columns, and
(b) rectangular stirrups in rectangular columns,

In either case, the closed stirrup shall have 135° hook ends with an extension of 6 times its diameter (but not < 65 mm) at each end, which are embedded in the confined core of the column (*see* Fig. 10a).

7.4.2 When rectangular stirrups are used,

(a) the minimum diameter permitted of transverse reinforcement bars is 8 mm, when diameter of longitudinal bar is less than or equal to 32 mm, and 10 mm, when diameter of longitudinal bar is more than 32 mm;
(b) the maximum spacing of parallel legs of stirrups shall be 300 mm centre to centre;
(c) a cross-tie shall be provided, if the length of any side of the stirrup exceeds 300 mm (*see* Fig. 10b); the cross-tie shall be placed perpendicular to this stirrup whose length exceeds 300mm. Alternatively, a pair of overlapping stirrups may be provided within the column (*see* Fig. 10c). In either case, the hook ends of the stirrups and cross-ties shall engage around peripheral longitudinal bars. Consecutive cross-ties engaging the same longitudinal bars shall have their 90°hooks on opposite sides of the column; and
(d) the maximum spacing of stirrups shall be half the least lateral dimension of the column, except where special confining reinforcement is provided as per **8**.

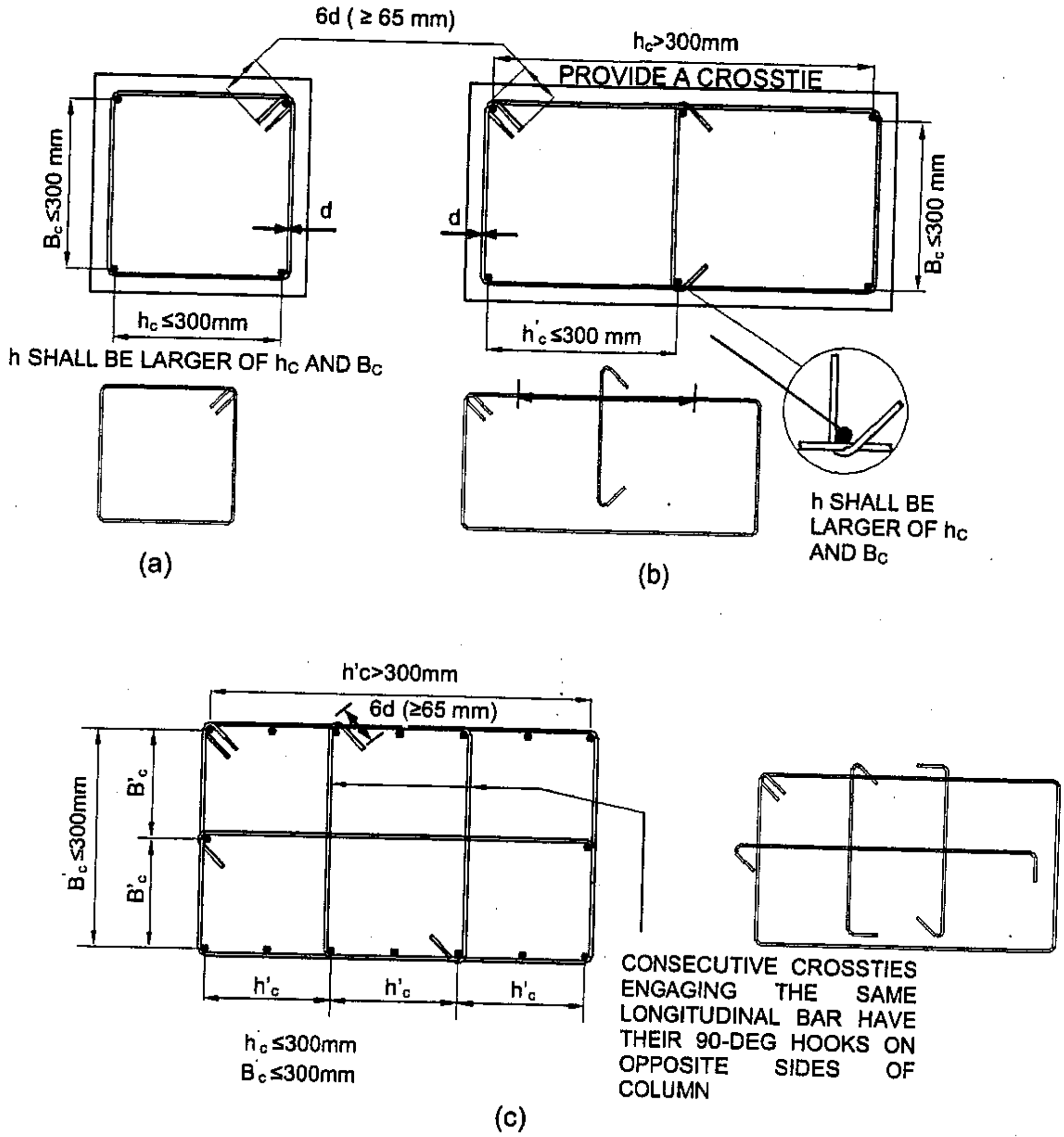

FIG. 10 DETAILS OF TRANSVERSE REINFORCEMENT IN COLUMNS.

7.5 Design Shear Force in Columns

The design shear force demand on columns is the larger of:

(a) Factored shear force demand as per linear structural analysis, and
(b) factored equilibrium shear force demand when plastic hinges are formed at both ends of the beams given by

(i) For sway to right:

$$V_u = 1.4\frac{\left(M_u^{As} + M_u^{Bh}\right)}{h_{st}},$$

(ii) For sway to left:

$$V_u = 1.4\frac{\left(M_u^{Ah} + M_u^{Bs}\right)}{h_{st}},$$

where M_u^{As}, M_u^{Ah}, M_u^{Bs} *and* M_u^{Bh} are design sagging and hogging moments of resistance of beams framing into the column on opposite faces *A* and *B*, respectively, with one hogging moment and the other sagging (*see* Fig. 11); and h_{st} the storey height. The design moments of resistance of beam sections shall be calculated as per IS 456.

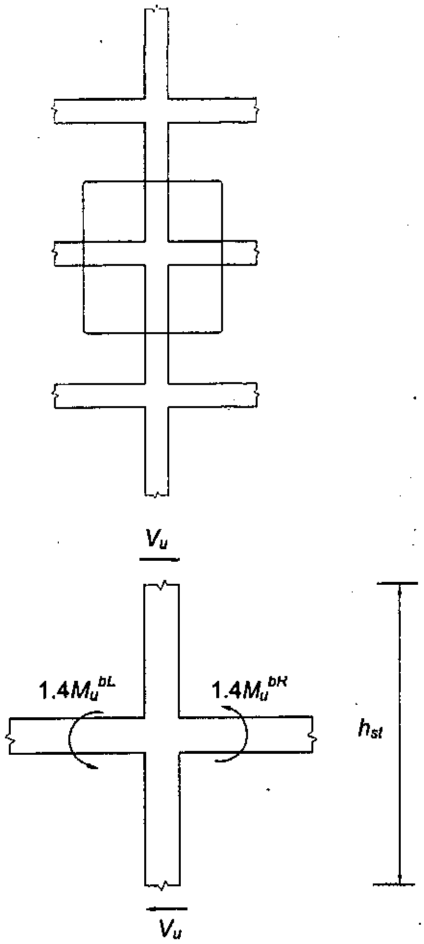

FIG. 11 EQUILIBRIUM DESIGN SHEAR FORCE DEMAND ON COLUMN WHEN PLASTIC HINGES ARE FORMED AT BEAM ENDS

7.5.1 The calculation of design shear force capacity of RC columns shall be calculated as per IS 456.

8. SPECIAL CONFINING REINFORCEMENT

The requirements of this section shall be met with in beams and columns, unless a larger amount of transverse reinforcement is required from shear strength considerations given in **6.3.3** for beams and **7.5** for columns.

8.1 Flexural yielding is likely in beams during strong earthquake shaking and in columns when the shaking intensity exceeds the expected intensity of earthquake shaking (*see* Fig. 12). This special confining reinforcement shall

(a) be provided over a length l_o from the face of the joint towards mid-span of beams and mid heights of columns, on either side of the joint; where l_o is not less than
 (i) larger lateral dimension of the member at the section where yielding occurs,
 (ii) 1/6 of clear span of the member, or
 (iii) 450 mm.
(b) have a spacing not more than
 (i) 1/4 of minimum member dimension of the beam or column,
 (ii) 6 times diameter of the smallest longitudinal reinforcement bars,
 (iii) 100 mm,
 but need not be less than 75 mm.
(c) have area A_{sh} of cross section of the bar forming stirrups or spiral of at least
 (i) in circular stirrups or spirals:

$$A_{sh} = Max\begin{bmatrix} 0.090 s_v D_k \dfrac{f_{ck}}{f_y}\left(\dfrac{A_g}{A_k}-1\right) \\ 0.024 s_v D_k \dfrac{f_{ck}}{f_y} \end{bmatrix}$$

where
s_v = pitch of spiral or spacing of stirrups,
D_k = diameter of core of circular column measured to outside of spiral/stirrup,
f_{ck} = characteristic compressive strength of concrete cube,
f_y = 0.2% Proof Strength of transverse steel reinforcement bars,
A_g = gross area of column cross-section, and
A_k = area of concrete core of column = $\dfrac{\pi}{4}D_k^2$

 (ii) in rectangular stirrups:

$$A_{sh} = Max\begin{bmatrix} 0.180 s_v h \dfrac{f_{ck}}{f_y}\left(\dfrac{A_g}{A_k}-1\right) \\ 0.05 s_v h \dfrac{f_{ck}}{f_y} \end{bmatrix}$$

where
h = longer dimension of rectangular stirrup measured to its outer face, which does not exceed 300 mm (*see* Fig. 10b), and
A_k= area of confined concrete core in rectangular stirrup measured to its outer dimensions.

h of the stirrup could be reduced by introducing crossties (*see* Fig. 10c). In such cases, A_k shall be measured as overall core area, regardless of stirrup arrangement. Hooks of cross-ties shall engage peripheral longitudinal bars.

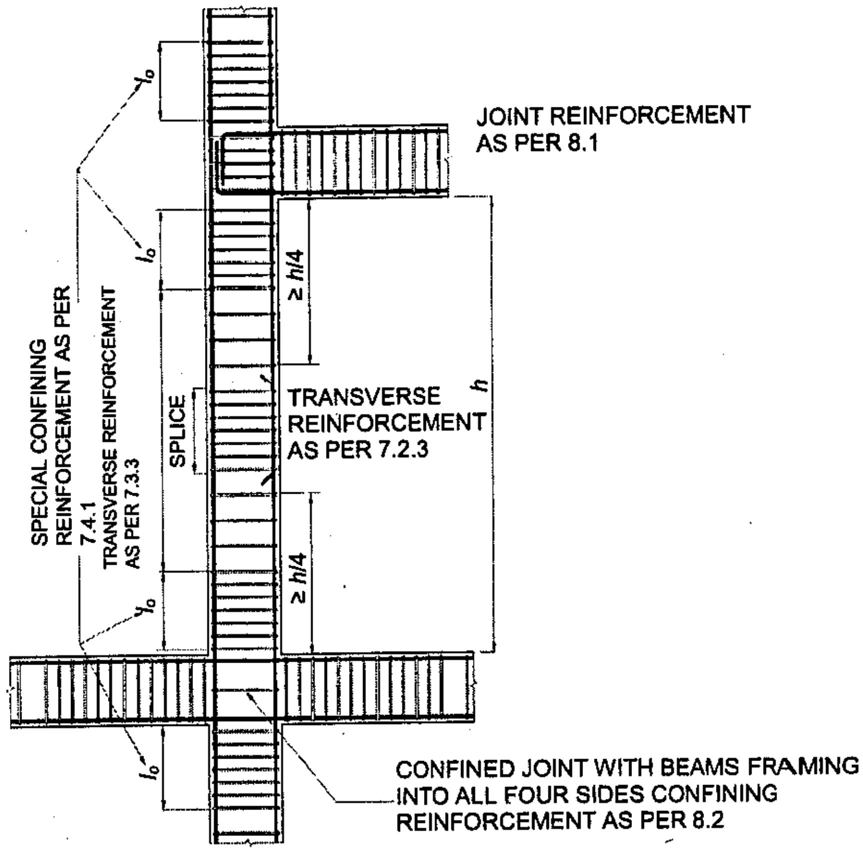

FIG. 12 COLUMN AND JOINT DETAILING

8.2 When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (*see* Fig. 13).

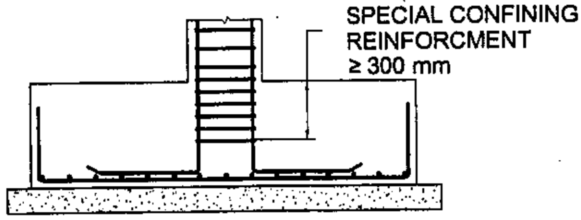

FIG. 13 PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTING

8.3 When the calculated point of contra-flexure, under the effect of gravity and earthquake effects, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

8.4 Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result due to abrupt changes in cross-section size, or unintended restraint to the column provided by stair-slab, mezzanine floor, plinth or lintel beams framing into the columns, RC wall or masonry wall adjoining column and extending only for partial column height

8.5 Columns carrying relatively small lateral forces are termed as gravity columns. Such columns supporting discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (*see* Fig. 14).This reinforcement shall be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; also it shall be provided below the discontinuity for the same development length.

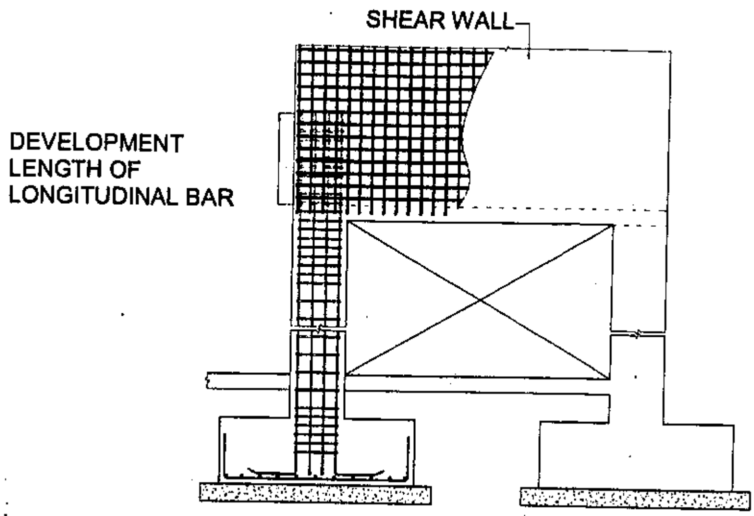

FIG. 14 COLUMNS WITH VARIABLE STIFFNESS

9. BEAM-COLUMN JOINTS OF MOMENT-RESISTING FRAMES

9.1 Design of Beam-Column Joint for Distortional Shear

9.1.1 *Shear Strength of Concrete in a Joint*

The nominal shear strength τ_{jc} of concrete in a beam-column joint shall be taken as

$$\tau_{jc} = \begin{cases} 1.5A_{ej}\sqrt{f_{ck}} & \text{for joints confined by beams on all four faces} \\ 1.2A_{ej}\sqrt{f_{ck}} & \text{for joints confined by beams on three faces} \\ 1.0A_{ej}\sqrt{f_{ck}} & \text{for other joints} \end{cases}$$

where A_{ej} is effective shear area of joint given by $b_j w_j$, in which b_j is the effective breadth of joint perpendicular to the direction of shear force and w_j the effective width of joint along the direction of shear force (*see* Fig. 15). In no case shall the area of joint be greater than the column cross-sectional area

9.1.2 *Design Shear Stress Demand on a Joint*

(a) Design Shear Stress Demand acting horizontally along each of the two principal plan directions of the joint shall be estimated from earthquake shaking considered along each of these directions, using

$$\tau_{jdX} = \frac{V_{djX}}{b_j w_j} \quad \text{for shaking along plan direction X of earthquake shaking}$$

$$\tau_{jdY} = \frac{V_{djY}}{b_j w_j} \quad \text{for shaking along plan direction Y of earthquake shaking}$$

It shall be ensured that the joint shear capacity of joint concrete estimated using **9.1.1** exceeds both τ_{jdX} and τ_{jdY}.

(b) Design Shear Force Demands V_{jdX} and V_{jdY} acting horizontally on the joint in principal plan directions X and Y shall be estimated considering that the longitudinal beam bars in tension reach a stress of $1.25f_y$ (when overstrength plastic moment hinges are formed at beam ends).

9.1.3 Width of Beam Column Joint

When beam reinforcement extends through beam-column joint, the minimum width of the column parallel to beam shall be 20 times the diameter of the largest longitudinal beam bar.

9.2 Transverse Reinforcement

9.2.1 *Confining Reinforcement in Joints*

(a) When all four vertical faces of the joint are having beams framing into them covering at least 75% of the width on each face,
   (i) At least half the special confining reinforcement required as per **8** at the two ends of columns, shall be provided through the joint within the depth of the shallowest beam framing into it, and
   (ii) Spacing of these transverse stirrups shall not exceed 150mm.

(b) When all four vertical faces of the joint are NOT having beams framing into them or when all four vertical faces have beams framing into them but do not cover at least 75% of the width on any face,

(i) Special confining reinforcement required as per **8** at the two ends of columns shall be provided through the joint within the depth of the shallowest beam framing into it, and

(ii) Spacing of these transverse stirrups shall not exceed 150mm.

9.2.2 In the exterior and corner joints, all 135° hooks of cross-ties should be along the outer face of columns.

10. SPECIALSTRUCTURAL WALLS

10.1 General Requirements

10.1.1 The requirements of this section apply to special structural walls that are part of lateral force resisting system of earthquake-resistant RC buildings.

10.1.2 The minimum thickness of special structural walls shall not be less than
(a) 150 mm
(b) 300 mm for buildings with coupled structural walls in any Seismic Zone.

10.1.3 Special structural walls shall be classified as squat, intermediate or slender depending on the overall height h_w to Length L_w ratio as
(a) Squat walls: $h_w / L_w < 1$,
(b) Intermediate walls: $1 \leq h_w / L_w \leq 2$, and
(c) Slender walls: $h_w / L_w > 2$.

10.1.4 In the design of flanged wall sections, only that part of the flange shall be considered which extends beyond the face of the web of the structural wall at least for a distance equal to smaller of
(a) Actual width available,
(b) Half the distance to the adjacent structural wall web, and
(c) $1/10^{th}$ of the total wall height.

10.1.5 Special structural walls shall be provided with uniformly spaced reinforcement in its cross-section along vertical and horizontal directions. At least a minimum area of reinforcement bars as indicated in Table 1 shall be provided along vertical and horizontal directions.

10.1.6 Reinforcement bars shall be provided in two curtains within the cross-section of the wall, with each curtain having bars running along vertical and horizontal directions, when
(a) Factored shear stress demand in the wall exceeds $0.25\sqrt{f_{ck}}$ MPa, or
(b) Wall thickness is 200 mm or higher.
When steel is provided in two layers, all vertical steel bars shall be contained within the horizontal steel bars; the horizontal bars shall form a closed core concrete area with closed loops and cross-ties.

10.1.7 The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed $1/10^{th}$ of the thickness of that part.

10.1.8 The maximum spacing of vertical or horizontal reinforcement shall not exceed smaller of
(a) $1/5^{th}$ horizontal length L_w of wall,
(b) 3 times thickness t_w of web of wall, and
(c) 450 mm.

10.1.9 Special structural walls shall be founded on properly designed foundations and shall not be discontinued to rest on beams, columns or inclined members.

Table 1: Minimum Reinforcement in RC Structural Walls

Type of Wall	*Reinforcement Details*
Squat Walls	$(\rho_h)_{min} = 0.0025$ $(\rho_v)_{min} = 0.0025 + 0.5\left(1 - \frac{h_w}{t_w}\right)(\rho_h - 0.0025)$ $(\rho_{v,net}) = (\rho_{v,web}) + \left(\frac{t_w}{L_w}\right).[0.02 - 2.5(\rho_{v,web})]$ $(\rho_v)_{provided} < (\rho_h)_{provided}$
Intermediate Walls	$(\rho_h)_{min} = 0.0025$ $(\rho_{v,be})_{min} = 0.0080$ $(\rho_{v,web})_{min} = 0.0025$ $(\rho_{v,net})_{min} = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right).$
Slender Walls	$(\rho_h)_{min} = 0.0025 + 0.5\left(\frac{h_w}{L_w} - 2\right)(\rho_h - 0.0025)$ $(\rho_{v,be})_{min} = 0.0080$ $(\rho_{v,web})_{min} = 0.0025$ $(\rho_{v,net})_{min} = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right).$

10.2 Design for Shear Force

10.2.1 Nominal shear stress demand τ_v on a wall shall be estimated as:

$$\tau_v = \frac{V_u}{t_w d_w},$$

where V_u is factored shear force, t_w thickness of the web, and d_w effective depth of wall section (along the length of the wall), which may be taken as $0.8L_w$ for rectangular sections.

10.2.2 Design shear strength τ_c of concrete shall be calculated as per Table 19 of IS 456.

10.2.3 When nominal shear stress demand τ_v on a wall is

(a) more than maximum design shear strength $\tau_{c,max}$ of concrete (given in Table 20 of IS 456), the wall section shall be redesigned;

(b) less than maximum design shear strength $\tau_{c,max}$ of concrete and more than design shear strength τ_c of concrete, design horizontal shear reinforcement shall be provided of area A_h given by

$$A_h = \frac{V_{us}}{0.87 f_y \left(\frac{d}{s_v}\right)_{Integral}} = \frac{V_u - \tau_c t_w d_w}{0.87 f_y \left(\frac{d}{s_v}\right)_{Integral}},$$

which shall not be less than the minimum area of horizontal steel per Clause 10.1.5; and

(c) less than design shear strength τ_c of concrete, horizontal shear reinforcement shall be the minimum area of horizontal steel per **10.1.5**.

10.3 Design for Axial Force and Bending Moment

10.3.1 Design moment of resistance M_u of the wall section subjected to combined bending moment and compressive axial load shall be estimated in accordance with requirements of limit state design method given in IS 456, using the principles of mechanics involving equilibrium equations, strain compatibility conditions and constitutive laws.

The moment of resistance of slender rectangular structural wall section with uniformly distributed vertical reinforcement may be estimated using expressions given in Annex A. Expressions given in Annex A are not applicable for structural walls with boundary elements,

10.3.2 The cracked flexural strength of a wall section shall be greater than its uncracked flexural strength.

10.3.3 In structural walls that do not have boundary elements, at least a minimum of 4 bars of 12 mm diameter arranged in 2 layers, shall be concentrated as vertical reinforcement at the ends of the wall over a length not exceeding twice the thickness of RC wall.

10.4 Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement even if they have the same thickness as that of the wall web. It is advantageous to provide boundary elements with dimension greater than thickness of the wall web.

10.4.1 Boundary elements shall be provided along the vertical boundaries of walls, when the extreme fiber compressive stress in the wall exceeds $0.2f_{ck}$ 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.

10.4.2 A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry axial compression arising from factored gravity load and lateral seismic shaking effects.

10.4.2.1 The load factor for gravity load shall be taken as 0.8, if gravity load gives higher axial compressive strength of the boundary element.

10.4.3 The vertical reinforcement in the boundary elements shall not be less than 0.8% and not greater than 6 %; the practical upper limit would be 4 % to avoid congestion.

10.4.4 Boundary elements, where required as per **10.4.1**, shall be provided with special confining reinforcement throughout their height, given by

$$A_{sh} = 0.05 s_v h \frac{f_{ck}}{f_y}.$$

10.4.6 Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per **8**.

10.5 Coupling Beams

10.5.1 Coplanar Special Structural walls may be connected by means coupling beams.

10.5.2 If earthquake induced shear stress τ_{ve} in coupling beam exceeds

$$\tau_{ve} > 0.1\sqrt{f_{ck}}\left(\frac{L_s}{D}\right),$$

where L_s is clear span of coupling beam and D overall depth, the entire earthquake-induced shear, bending moment and axial compression shall be resisted by diagonal reinforcement alone.

(a) Area of this diagonal reinforcement along each diagonal shall be estimated as:

$$A_{st} = \frac{V_u}{1.74 f_y \sin\alpha},$$

where V_u is factored shear force on the coupling beam and α the angle made by diagonal reinforcement with the horizontal.

(b) At least 4 bars of 8 mm diameter shall be provided along each diagonal. All longitudinal bars along each diagonal shall be enclosed by special confining transverse reinforcement as per Clause 8 at a spacing not exceeding 100 mm.

10.5.3 The diagonal of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension. (*see* Fig. 15)

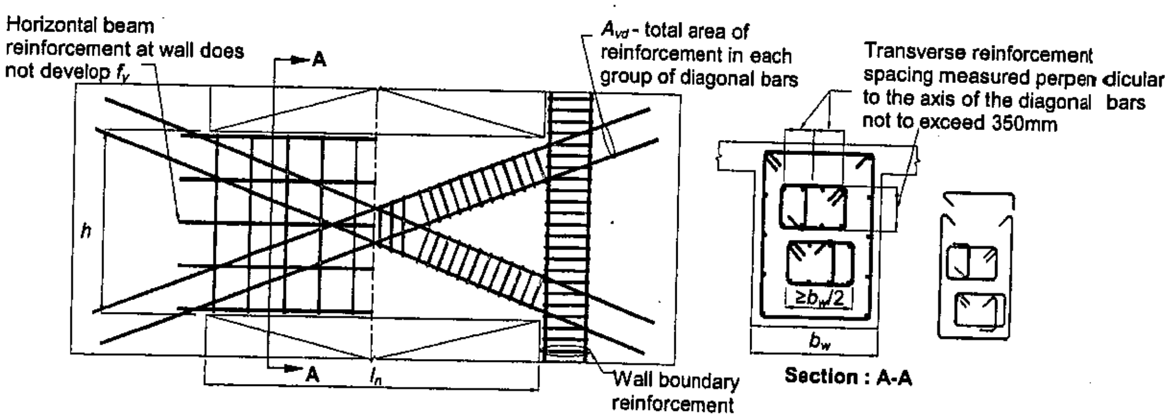

FIG. 15 COUPLING BEAMS WITH DIAGONAL REINFORCEMENT

10.6 Openings in Walls

10.6.1 Shear strength of a wall with openings should be checked at critical horizontal planes passing through openings.

10.6.2 Additional steel reinforcement shall be provided along all four edges of openings in walls.

(a) The area of these vertical and horizontal steel should be equal to that of the respective interrupted bars, provided half on either side of the wall in each direction.
(b) These vertical bars should extend for full height of the storey in which this opening is present.
(c) The horizontal bars should be provided with development length in tension beyond the edge of the opening.

10.7 Construction Joints

Vertical reinforcement across a horizontal construction joint shall have area A_{st} given by:

$$\frac{A_{st}}{A_g} \geq \frac{0.92}{f_y}\left(\tau_v - \frac{P_u}{A_g}\right)$$

where τ_v is factored shear stress at the joint, P_u factored axial force (positive for compression), and A_g gross cross-sectional area of joint.

10.8 Development, Splice and Anchorage Requirement

10.8.1 Horizontal reinforcement shall be anchored near the edges of wall or in confined core of boundary elements.

10.8.2 In slender walls (H/L_w > 2), splicing of vertical flexural reinforcement should be avoided, as far as possible, in regions where flexural yielding may take place, which extends for a distance larger of
(a) L_w above the base of the wall, and
(b) 1/6th of the wall height,
but not larger than $2L_w$.

10.8.3 Splices

10.8..3.1 Lap Splices

When adopted, closed stirrups shall be provided over the entire length over which the longitudinal bars are spliced.

(a) The spacing of these stirrups shall not exceed 150 mm.
(b) The lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
(c) Lap splices shall be provided only in the central half of clear wall height, and not
 (i) within a joint, or
 (ii) within a distance of 2*d* from face of the beam.
(d) Not more than 50% of area of steel bars shall be spliced at any one section.

10.8.3.2 Mechanical couplers [conforming to IS xxxxx (*under print*)] shall be used. Further, only those mechanical splices conforming to the above standard and capable of developing the specified tensile strength of spliced bar shall be permitted within a distance equal to two times the depth of the member from the beam-column joint or in any location where yielding of reinforcement is likely to take place.

10.8.3.4 *Welded Splices*

Welded splices shall be avoided as far as possible. In no case shall they be used for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place. At any location, not more than 50% of area of steel bars shall be spliced at any one section.

Welding of stirrups, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone.

10.8.4 In buildings located in Seismic Zones II and III, closed loop transverse stirrups shall be provided around lapped spliced bars larger than 16 mm in diameter. The minimum diameter of such stirrups shall be 1/4th of diameter of spliced bar but not less than 8 mm at spacing not exceeding 150 mm centers.

11. GRAVITY COLUMNS IN BUILDINGS

Gravity columns in buildings shall be detailed according to Clauses **11.1** and **11.2** for bending moments induced when subjected to twice the design lateral displacement under the factored equivalent static design seismic loads given by IS 1893 (Part 1).

11.1 Clauses **11.1.1** and **11.1.2** shall be satisfied, when induced bending moments and horizontal shear forces under the said lateral displacement combined with factored gravity bending moment and shear force *do not* exceed the design moment of resistance and design lateral shear capacity of the column.

11.1.1 Gravity columns shall satisfy **7.3.2**, **7.4.1** and **7.4.2**. But, spacing of stirrups along the full column height shall not exceed 6 times diameter of smallest longitudinal bar or 150 mm.

11.1.2 Gravity columns with factored gravity axial stress exceeding 0.4 f_{ck} shall satisfy **11.1.1** and shall have transverse reinforcement at least one half of special confining reinforcement required by **8**.

11.2 When induced bending moments and shear forces under said lateral displacement combined with factored gravity bending moment and shear force exceed design moment and shear strength of the frame, **11.2.1** and **11.2.2** shall be satisfied.

11.2.1 Mechanical and welded splices shall satisfy **7.3.2.2** and **7.3.2.3**.

11.2.2 Gravity columns shall satisfy **7.4** and **8**.

ANNEX A
(*Clause* 10.3.1)

A.1 MOMENT OF RESISTANCE OF RECTANGULAR SHEAR WALL SECTION

The moment of resistance M_u of a slender rectangular structural wall section with uniformly distributed vertical reinforcement may be estimated as:

(a) For $(x_u/l_w) < (x_u^*/l_w)$

$$\frac{M_u}{f_{ck}t_w L_w^2} = \varphi\left[\left(1+\frac{\lambda}{\varphi}\right)\left(\frac{1}{2}-0.416\frac{x_u}{L_w}\right)-\left(\frac{x_u}{L_w}\right)^2\left(0.168+\frac{\beta^2}{3}\right)\right]$$

where

$$\frac{x_u}{L_w} = \left(\frac{\varphi+\lambda}{2\varphi+0.36}\right);$$

$$\frac{x_u^*}{L_w} = \frac{0.0035}{0.0035+\left(0.002+0.87f_y/E_s\right)};$$

$$\varphi = \left(\frac{0.87f_y\rho}{f_{ck}}\right);$$

$$\lambda = \left(\frac{P_u}{f_{ck}t_w L_w}\right);$$

ρ = vertical reinforcement ratio = $\left(\frac{A_{st}}{t_w L_w}\right)$,

A_{st} = area of uniformly distributed vertical reinforcement,

$$\beta = \frac{\left(0.002+0.87f_y/E_s\right)}{0.0035},$$

E_s = elastic modulus of steel, and

P_u = factored compressive axial force on wall.

(b) For $(x_u^*/L_w) < (x_u/L_w) < 1.0$

$$\frac{M_u}{f_{ck}t_w L_w^2} = \left(\frac{x_u}{L_w}\right) - \alpha_2\left(\frac{x_u}{L_w}\right)^2 - \alpha_3 - \frac{\lambda}{2}$$

where

$$\alpha_1 = \left[0.36+\varphi\left(1-\frac{\beta}{2}-\frac{1}{2\beta}\right)\right]$$

$$\alpha_2 = \left[0.15+\frac{\varphi}{2}\left(1-\beta+\frac{\beta^2}{3}-\frac{1}{3\beta}\right)\right] \text{ and}$$

$$\alpha_3 = \frac{\varphi}{6\beta}\left(\frac{1}{x_u/L_w}-3\right).$$

x_u/L_w to be used in this expression shall be obtained by solving the equation:

$$\left(\frac{x_u}{L_w}\right)^2 + \alpha_4\left(\frac{x_u}{L_w}\right) - \alpha_5 = 0$$

where

$\alpha_4 = \left(\dfrac{\varphi}{\beta} - \lambda\right)$ and

$\alpha_5 = \left(\dfrac{\varphi}{2\beta}\right)$.

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भारतीय मानक

चिनाई वाले भवनों के भूकंपीय मूल्यांकन मरम्मत और सुद ढ़ीकरण — मार्गदर्शिका

(पहला पुनरीक्षण)

Indian Standard

SEISMIC EVALUATION, REPAIR AND STRENGTHENING OF MASONRY BUILDINGS — GUIDELINES

(First Revision)

ICS 91.120.25

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

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Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard (First Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

Himalayan-Naga Lushai region, Indo-Gangetic Plain, Western India and Kutch and Kathiawar regions are geologically unstable parts of the country and some devastating earthquakes of the world have occurred there. A major part of the peninsular India has also been visited by moderate earthquake, but these were relatively few in number and had considerably lesser intensity. It has been a long felt need to rationalize the earthquake resistant design and construction of structures taking into account seismic data from studies of the Indian earthquakes, particularly in view of the heavy construction programme all over the country. It is to serve this purpose that IS 1893 : 1962 'Criteria for earthquake resistant design of structures' was formulated and subsequently revised in 1966, 1970, 1975, 1984 and 2002. It lays down the seismic zones, the basic seismic coefficients and other factors and criteria for various structures. As an adjunct to IS 1893, IS 4326 'Code of practice for earthquake resistant design and construction of buildings' was formulated in 1967 and revised in 1976 and in 1993. The 1976 version contained some recommendations for low strength brick masonry and stone buildings which have now been covered in greater detail in IS 13828 : 1993 'Guidelines for improving earthquake resistance of low strength masonry building'.

Earthquake damages to buildings in Himachal Pradesh, North Bihar and hill districts of Uttar Pradesh emphasized the need to formulate this standard to cover guidelines for repair and strengthening of these buildings from any future earthquakes and it was first published in 1993.

The following are the major important modifications made in this revision:

a) Non-shrink grouts and fiber reinforced plastics have been incorporated for repair, restoration work and strengthening.
b) Damageability assessment of existing masonry buildings under earthquake occurrences has been incorporated.
c) Assessment of retrofitting requirements and actions for retrofitting also incorporated.
d) Provision of seismic belts around door and window openings.
e) Rapid visual screening method along with RVS survey forms for masonry buildings for seismic hazards evaluation has been incorporated.

In this revision, due weightage has been given to international coordination among standards and practices prevailing in different countries in the field in the country.

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 '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 specified value in this standard.

Indian Standard

SEISMIC EVALUATION, REPAIR AND STRENGTHENING OF MASONRY BUILDINGS — GUIDELINES

(First Revision)

1 SCOPE

1.1 This standard covers the selection of materials and techniques to be used for repair and seismic strengthening of damaged buildings during earthquakes. It also covers the damageability assessment and retrofitting for upgrading of seismic resistance of existing masonry buildings covered under IS 4326 and IS 13828.

1.2 The repair materials and techniques described herein may be used for all type of masonry buildings and construction.

1.3 The provisions of this standard are applicable for buildings in seismic Zones III to V of IS 1893 (Part 1). These are based on damaging seismic intensities VII and more on M. S. K. Intensity scales. The scheme of strengthening should satisfy the requirements stipulated for the seismic zone of IS 1893 (Part 1), building categories of IS 4326 and provisions made in this Code and in IS 13828 for low strength masonry building. No special seismic resistance features are considered necessary for buildings in seismic Zone II, but the important buildings in this Zone may also be considered for upgrading their seismic resistance.

1.4 The suggested reinforcing of horizontal and vertical seismic belts in this standard follows IS 4326 requirements of horizontal seismic bands and vertical bars at critical sections. For special buildings having larger span and heights beyond the dimensions considered in IS 4326 and in this standard, special analysis may be carried out at the responsibility of the specialist.

2 REFERENCES

The standards given below 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 given below:

IS No.	*Title*
1893 (Part 1) : 2002	Criteria for earthquake design of structures: Part 1 General Provisions and buildings
4326 : 1993	Code of practice for earthquake resistant design and construction of buildings (*third revision*)
13828 : 1993	Guidelines for improving earthquake resistance of low strength masonry buildings

3 TERMINOLOGY

For the purpose of this Code, the definitions given in IS 1893 (Part 1) and the following shall apply.

3.1 Box System — A bearing wall structure without a space frame, the horizontal forces being resisted by the walls acting as shear walls.

3.2 Centre of Rigidity — The point in a structure, where a lateral force shall be applied to produce equal deflections of its components, at any one level in any particular direction.

3.3 Design Seismic Coefficients — The value of horizontal seismic coefficient computed taking into account the zone factor, soil system, the importance factor and the response reduction factor as specified in IS 1893 (Part 1).

3.4 Seismic Band — A reinforced concrete, reinforced brick or wooden runner provided horizontally in the walls to tie them together and to impart horizontal bending strength in them.

3.5 Seismic Belt — A cast-in-place Ferro-cement plating installed post-construction on the masonry wall in lieu of the seismic bands or vertical reinforcing bars specified in IS 4326 and IS 13828.

3.6 Seismic Zone and Seismic Coefficient — Classification of seismic Zones II to V and the corresponding basic seismic coefficients shall be as specified in IS 1893 (Part 1).

3.7 Shear Wall — A wall designed to resist lateral force in its own plane. Braced frames, subjected primarily to axial stresses, shall be considered as shear walls for the purpose of this definition.

4 GENERAL PRINCIPLES AND CONCEPTS

4.1 Non-structural/Architectural Repairs

4.1.1 The buildings affected by earthquake may suffer both non-structural and structural damages. Non-structural repairs may cover the damages to civil and electrical items including the services in the building. Repairs to non-structural components need to be taken up after the structural repairs and retrofitting work are carried out. Care should be taken about the connection details of architectural components to the main structural components to ensure their stability.

4.1.2 Non-structural and architectural components get easily affected/dislocated during the earthquake. These repairs involve one or more of the following:

a) Patching up of defects such as cracks and fall of plaster;
b) Repairing doors, windows, replacement of glass panes;
c) Checking and repairing electric conduits/ wiring;
d) Checking and repairing gas pipes, water pipes and plumbing services;
e) Re-building non-structural walls, smoke chimneys, parapet walls, etc;
f) Replastering of walls as required;
g) Rearranging disturbed roofing tiles;
h) Relaying cracked flooring at ground level; and
j) Redecoration — white washing, painting, etc.

The architectural repairs as stated above do not restore the original structural strength of structural components in the building and any attempt to carry out only repairs to architectural/non-structural elements neglecting the required structural repairs may have serious implications on the safety of the building. The damage would be more severe in the event of the building being shaken by the similar shock because original energy absorption capacity of the building would have been reduced.

4.2 Structural Repairs/Restoration

4.2.1 Prior to taking up of the structural repairs for restoration of original strength and any strengthening measures, it is necessary to conduct detailed damage assessment to determine:

a) the structural condition of the building to decide whether a structure is amendable for repair; whether continued occupation is permitted; to decide the structure as a whole or a part require demolition, if considered dangerous;
b) if the structure is considered amendable for structural repair then detailed damage assessment of the individual structural components (mapping of the crack pattern, distress location; crushed concrete, reinforcement bending/yielding, etc). Non-destructive testing techniques could be employed if found necessary, to determine the residual strength of the members; and
c) to work out the details of temporary supporting arrangement of the distressed members so that they do not undergo further distress due to gravity loads.

4.2.2 After the assessment of the damage of individual structural elements, appropriate repair methods are to be carried out component wise depending upon the extent of damage. The restoration work may consist of the following:

a) Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of non-shrinking mortar will be preferable.
b) Addition of reinforcing mesh on both faces of the cracked wall, holding it to the wall through spikes or bolts and then covering it, suitably, with micro-concrete (maximum size of aggregate limited to 6 mm or less as suitable), and may be with use of micro-reinforcement as fibre or ferro-cement.
c) Injecting cement, polymer-cement mixture or epoxy materials which are strong in tension, into the cracks in walls.
d) The cracked reinforced concrete elements like slabs, beams and lintels may be repaired by epoxy grouting and could be strengthened by epoxy or polymer mortar application like shotcreting, jacketing, etc.

NOTE — In mortar for masonry or plaster, fibers can be used.

4.3 Seismic Strengthening

The main purpose of the seismic strengthening is to upgrade the seismic resistance of a damaged building while repairing so that it becomes safer under future earthquake occurrences. This work may involve some of the following actions:

a) Increasing the lateral strength in one or both directions by increasing column and wall areas or the number of walls and columns.
b) Giving unity to the structure, by providing a

proper connection between its resisting elements, in such a way that inertia forces generated by the vibration of the building can be transmitted to the members that have the ability to resist them. Typical important aspects are the connections between roofs or floors and walls, between intersecting walls and between walls and foundations.

c) Eliminating features that are sources of weakness or that produce concentration of stresses in some members. Asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor to the other, concentration of large masses and large openings in walls without a proper peripheral reinforcement are examples of defects of this kind.

d) Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members.

4.4 Seismic Retrofitting

Many existing buildings do not meet the seismic strength requirements of present earthquake codes due to original structural inadequacies and material degradation over time or alterations carried out during use over the years. Their earthquake resistance can be upgraded to the level of the present day codes by appropriate seismic retrofitting techniques, such as mentioned about seismic strengthening in **4.3**.

4.5 Strengthening or Retrofitting *Versus* Reconstruction

4.5.1 Replacement of damaged buildings or existing unsafe buildings by reconstruction is, generally, avoided due to a number of reasons, the main ones among them being:

a) Higher cost of re-building than that of strengthening or retrofitting,

b) Preservation of historical architecture, and

c) Maintaining functional social and cultural environment.

In most instances, however, the relative cost of retrofitting to reconstruction cost determines the decision. As a thumb rule, if the cost of repair and seismic strengthening is less than about 30 percent of the reconstruction cost, the retrofitting is adopted. This may also require less working time and much less dislocation in the living style of the population. On the other hand reconstruction may offer the possibility of modernization of the habitat and may be preferred by well-to-do communities.

4.5.2 Cost wise the building construction including the seismic code provisions in the first instance, works out the cheaper in terms of its own safety and that of the occupants. Retrofitting an existing inadequate building may involve as much as 2.5 to 3 times the initial extra expenditure required on seismic resisting features. Repair and seismic strengthening of a damaged building may even be 4 to 6 times as expensive. It is therefore very much safe as well as cost-effective to construct earthquake resistant buildings at the initial stage itself according to the relevant seismic Indian Standards.

5 SELECTION OF MATERIALS AND TECHNIQUES

5.1 General

The most common materials for repair, restoration works of various types of buildings are cement and steel. In many situations suitable admixture may be added to cement mortar/cement concrete to improve their properties, such as, non-Shrinkage, bond, etc. Steel may be required in many forms like bolts, rods, angles, beams, channels, expanded metal and welded wire fabric. Wood and bamboo are the most common material for providing temporary supports and scaffolding, etc, and will be required in the form of rounds, sleepers, planks, etc.

Besides the above, special materials and techniques are available for best results in the repair and strengthening operations. These should be selected appropriately depending on the nature and cost of the building that is to be repaired, materials availability and feasibility, and use of available skills, etc. Some special materials and techniques are described below.

5.2 Non-shrink Grouts

Currently ready grout contents consisting of a polymer, non-shrink cement and special sands are available in the market which is suitable to prepare the desired grout for the crack width observed in the masonry. The polymer improves the adhesion of the grout with the masonry as well imparts higher tensile strength.

5.3 Shotcrete

Shotcrete is cement mortar or cement concrete (with coarse aggregate size maximum 10 mm) conveyed through a hose and pneumatically placed under high velocity on to a prepared concrete or masonry surface. The force of the jet impingement on the surface, compacts the shotcrete material and produces a dense homogeneous mass. Basically there are two methods of shotcreting; wet mix process and dry mix process. In the wet mix process, all the ingredients, including water are mixed together before they enter the delivery hose. In the dry mix process, the mixture of damp sand

and cement is passed through the delivery hose to the nozzle where the water is added. The dry mix process is generally used in the repair of concrete elements. The bond between the prepared concrete surface of the damaged member and the layer of shotcrete is ensured with the application of suitable epoxy adhesive formulation. The shear transfer between the existing and new layer of concrete is ensured with the provision of shear keys. Addition of fibers enhances tensile strength and toughness.

5.4 Epoxy Resins

Epoxy resins are excellent binding agents with high tensile strength. These are chemical preparations the compositions of which can be changed as per requirements. The epoxy components are mixed just prior to application. Some products are of low viscosity and can be injected in fine cracks too. The higher viscosity epoxy resin can be used for surface coating or filling larger cracks or holes. The epoxy resins may also be used for gluing steel plates to the distressed members.

5.5 Epoxy Mortar

For larger void spaces, it is possible to combine the epoxy resins of either low viscosity or higher viscosity with sand aggregate to form epoxy mortar. Epoxy mortar mixture has higher compressive strength, higher tensile strength and a lower modulus of elasticity than cement concrete. The sand aggregate mixed to form the epoxy mortar increases its modulus of elasticity.

5.6 Quick-Setting Cement Mortar

This material is non-hydrous magnesium phosphate cement with two components, that is, a liquid and a dry powder, which can be mixed in a manner similar to cement concrete.

5.7 Mechanical Anchors

Mechanical types of anchors employ wedging action to provide anchorage. Some of the anchors provide both shear and tension resistance. Such anchors are manufactured to give sufficient strength.

Alternatively, chemical anchors bonded in drilled holes through polymer adhesives can be used.

5.8 Fibre Reinforced Plastics (FRP)

Fibre-reinforced polymers/plastics are a recently developed material for strengthening of reinforced concrete and masonry structure. This is an advanced material and most of the development in its application in structural retrofitting has taken place in the last two decades. It has been found to be a replacement of steel plate bonding. The main advantage of FRP is its high strength to weight ratio and high corrosion resistance. FRP plates can be 2 to 10 times stronger than steel plates, while their weight is just 20 percent of that of steel. These have to be glued to the walls or columns using epoxy mortars.

6 TECHNIQUES TO RESTORE ORIGINAL STRENGTH

6.1 General

While considering restoration of structural strength, it is important to realize that even fine cracks in load bearing members which are unreinforced like masonry and plain concrete reduce their resistance in a large measure. Therefore, all cracks must be located and marked carefully and the critical ones fully repaired structurally either by injecting strong cement or chemical grout or by providing external bandage. The techniques are described below along with other restoration measures.

6.2 Structural Repair of Minor and Medium Cracks

For the repair of minor and medium cracks (0.50 mm to 5 mm), the technique to restore the original tensile strength of the cracked element is by pressure injection of non-shrink cement polymer grout. The procedure is given in **6.2.1** (*see* Fig. 1A).

6.2.1 The external surfaces are cleaned of non-structural materials and plastic/aluminium injection ports are placed along the surface of the cracks on both sides of the member and are secured in place with polyester putty or 1 : 3 cement mortar. The centre-to-centre spacing of these ports may be approximately equal to the thickness of the wall element. After the sealant has cured, cracks are cleared using compressed air, and/or water. Thereafter the grout is injected into one port at a time beginning at the lowest part of the crack, in case it is vertical, or at one end of the crack, in case it is horizontal.

The grout is injected till it is seen flowing from the opposite sides of the member at the corresponding port or from the next higher port on the same side of member. The injection port should be closed at this stage and injection equipment moved to the next port and so on.

The finer the crack, higher is the pressure or more closely spaced should be the ports so as to obtain complete penetration of the appropriate grout material throughout the depth and width of member. Larger cracks will permit larger port spacing depending upon width of the member. This technique is appropriate for all types of structural elements — beams, columns, walls and floor units in masonry as well as concrete structures. In the case of loss of bond between reinforcing bar and concrete, if the concrete adjacent to the bar has been pulverized to a very fine powder (this powder will block the grout from penetrating the region). It should be cleaned properly by air or water pressure prior to injection of grout in the concrete.

6.3 Repair of Major Cracks and Crushed Concrete

For cracks wider than about 5 mm or for regions in which the concrete or masonry has crushed, a treatment other than injection is indicated. The procedures may be adopted as follows:

a) The loose material is removed and replaced with any of the materials mentioned earlier, that is, expansive cement mortar, quick setting cement (*see* Fig. 1B).
b) Where found necessary, additional shear or flexural reinforcement is provided in the region of repairs. This reinforcement could be covered by mortar to give further strength as well as protection to the reinforcement (*see* Fig. 1C).
c) In areas of very severe damage, replacement of the member or portion of member can be carried out.
d) In the case of damage to walls and floor diaphragms, steel mesh could be provided on the outside of the surface and nailed or bolted to the wall. Then it may be covered with plaster or micro-concrete (*see* Fig. 1C).

6.4 Fractured Excessively Yielded and Buckled Reinforcement

In the case of severely damaged reinforced concrete member it is possible that the reinforcement would have buckled or elongated or excessive yielding may have occurred. This element can be repaired by replacing the old portion of steel with new steel using butt welding or lap welding.

Splicing by overlapping will be risky. If repair has to be made without removal of the existing steel, the best approach would depend upon the space available in the original member. Additional stirrup ties are to be added in the damaged portion before concreting so as to confine the concrete and enclose the longitudinal bars to prevent their buckling in future.

In some cases, it may be necessary to anchor additional steel into existing concrete. A common technique for providing the anchorage uses the following procedure.

6.4.1 A hole larger than the bar is drilled. The hole is filled with epoxy expanding cement or other high strength grouting material. The bar is pushed into place and held there until the grout has set.

7 DAMAGEABILITY ASSESSMENT OF EXISTING MASONRY BUILDINGS

The assessment of possible grade of damage in an existing masonry building under earthquake occurrences will mainly depend on:

a) Probable maximum intensity of the earthquake,
b) Building typology,
c) Building configuration, and
d) Quality of construction and maintenance over time.

For initial quick assessment a rapid visual screening of the building may be carried out as given in Annex A. Besides the above factors, consideration of importance factor of 1.5 as per IS 1893 (Part 1) has also been considered by raising the earthquake Intensity for assessment of damageability. Based on assessed damageability of the building the further actions can be planned (*see* Table 1 and Table 3).

Table 1 Damageability Grades and Retrofitting Actions

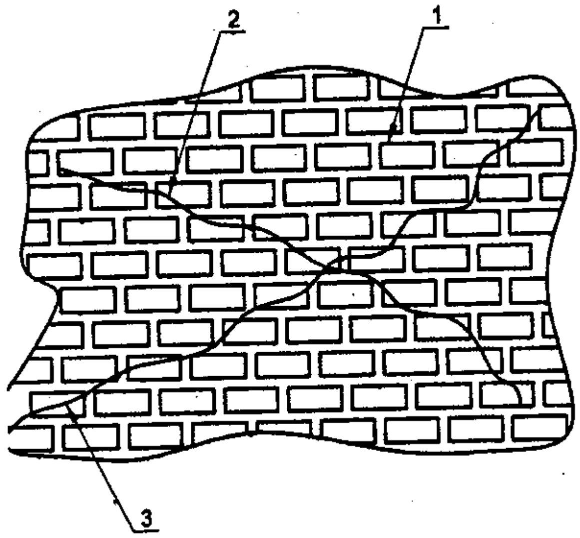
Sl No. (1)	Damage-ability Grade (2)	Suggested Actions Regarding Retrofitting (3)
i)	G1	Retrofitting not needed
ii)	G2	Structural retrofitting not needed, unstable non-structural elements to be stabilized
iii)	G3	a) Structural and non-structural elements to be retrofitted b) Global as well as element deficiencies to be evaluated and retrofitting to be suitably designed
iv)	G4	a) Structural and non-structural elements to be retrofitted b) Global as well as element deficiencies to be evaluated and retrofitting to be suitably designed
v)	G5	a) Structural and non-structural elements to be retrofitted b) Global as well as element deficiencies to be evaluated and retrofitting to be suitably designed c) Alternatively, replacement of the existing damaged building with a new earthquake resistant building to be considered

NOTES

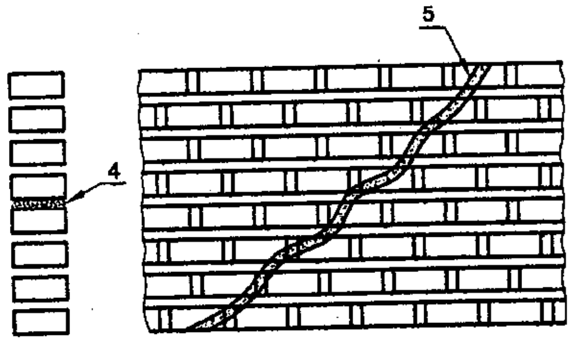
1 Detailed description of Grades G1 to G5 is given in Annex A.

2 In case of G4 and G5, the option of replacing the existing building with a new one may also be explored, particularly for old buildings.

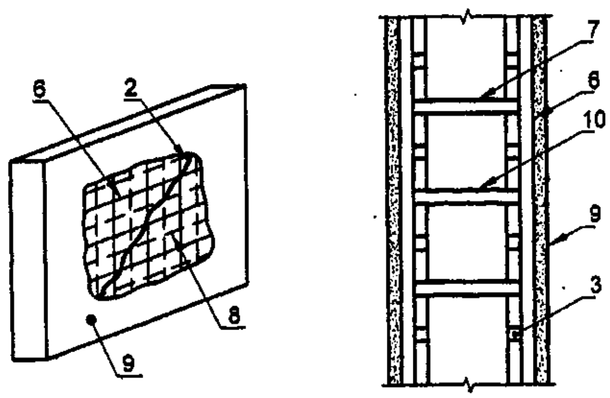
3 In case of G4 and G5, only non-collapse performance level may be aimed at. For example, in Seismic Zones IV and V, masonry buildings constructed using mud or weak lime mortar will fall in G4 and G5 categories and if they are retrofitted using the details of IS 13828 as for new buildings, non-collapse performance will be achieved.



1A Grout or Epoxy Injection in Cracks



1B Cement Mortar and Flat Chips in Wide Cracks



1C Cement Mortar and Wire Mesh in Cracks

1 Plaster removed
2 Cracks sealed after cleaning
3 Grout ports
4 V- Groove joints
5 Cement mortar and flat stone chips
6 Wire mesh on front face
7 Clamps
8 Wire mesh on back face
9 Cement plaster
10 Crack in wall

FIG. 1 STRUCTURAL RESTORATION OF CRACKED MASONRY WALLS

8 ASSESSMENT OF RETROFITTING REQUIREMENTS

8.1 Categorization of Buildings

In all cases of masonry buildings, ordinary as well as important [defined in IS 1893 (Part 1) with I = 1.5], the retrofitting requirements can directly be assessed by comparing the data of the building under consideration with the specified safety requirements in the IS 4326 and IS 13828 as the case may be. Tables 3, 4 and 5 illustrate this approach, in which the building categories B, C, D and E are to be taken as given in Table 2.

Table 2 Building Categories (for use with IS 4326 and IS 13828)

Sl No.	Building Use	Building Category in Seismic Zone			
		II	III	IV	V
(1)	(2)	(3)	(4)	(5)	(6)
i)	Ordinary	B	C	D	E
ii)	Important (I=1.5)	C	D	E	E

8.2 Special or Critically Important Buildings

Besides the Ordinary and Important buildings defined in IS 1893 (Part 1), there are some special buildings of monumental nature or of critical importance to the safety of the occupants, for example the Qutab Minar and Taj Mahal on the one hand and Rashrapati Bhawan, residences and offices of the VVIPS on the other hand. Such buildings will need to be rationally analysed using the seismic actions as per IS 1893 (Part 1) taken appropriately while adopting a higher importance factor, say 2.0, or the MCE zone factor Z, and fixing the performance criterion, that is suitable to the post-earthquake usability/repairability of the building. The strategy for retrofitting of such buildings will have to be chosen by the structural engineer concerned and proof checked by an expert appointed by the owner.

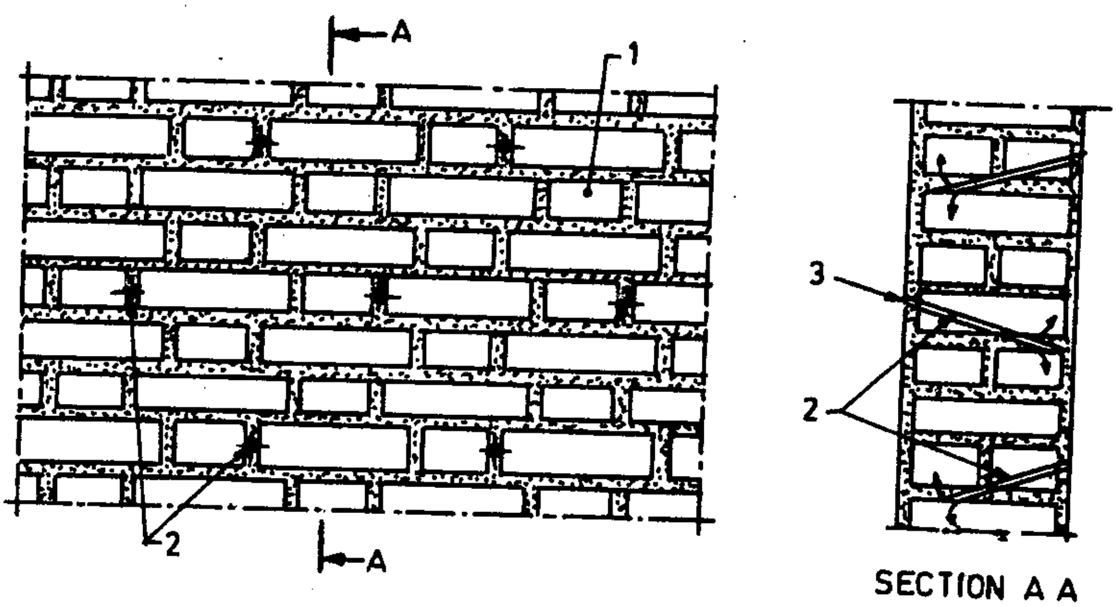
9 STRENGTHENING EXISTING WALLS

The lateral strength of buildings can be improved by increasing the strength and stiffness of existing individual walls, whether they are cracked or uncracked. This can be achieved:

a) by grouting,
b) by addition of vertical reinforced concrete coverings on the two sides of the wall, and
c) by prestressing wall.

9.1 Grouting

A number of holes are drilled in the wall (2 to 4 in each m^2) (*see* Fig. 2). First water is injected in order to wash the wall inside, and to improve the cohesion between the grouting mixture and the wall elements. Secondly, a cement water mixture (1 : 1) is grouted at low pressure (0.1 to 0.25 MPa) in the holes starting from the lower holes and going up.



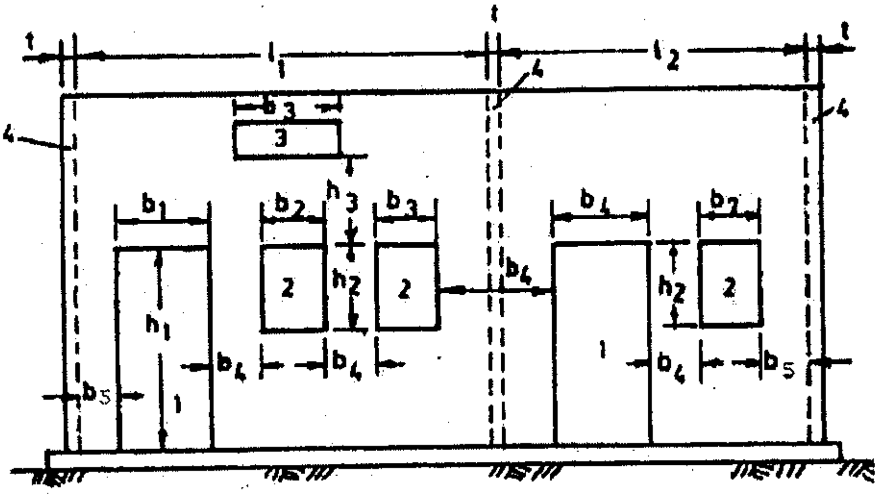
1 Brick or block wall
2 Injection holes
3 Crout mixture

FIG. 2 GROUT OR EPOXY INJECTION IN EXISTING WEAK WALLS

Table 3 Provisions in IS 4326 and Actions for Retrofitting

(*Clause* 8.1)

Sl No.	Item of Masonry	Requirement as per IS 4326 for Building Category				Action for Retrofitting, if Code Requirement not Found Satisfied
		B	C	D	E	
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Mortar	CLS-1 : 2 : 9 or CS-1 : 6		CLS-1 : 1 : 6 or CS-1 : 4		Change of mortar not feasible. Hollowness may be filled by grouting or walls may be strengthened by ferro-cement plating or fibre-wrapping
ii)	Door, Window openings: 1) b_5 minimum	0.0	230 mm	450 mm	450 mm	Increase by build-up or reinforce with belt
	$(b_1 + b_2 +.)/l$, *Max*:					Attain the limit by closing/ narrowing an opening or reinforce the opening by seismic belting
	a) one storey	0.6	0.55	0.5	0.5	
	b) two storey	0.50	0.46	0.42	0.42	
	c) three storey	0.42	0.37	0.33	0.33	
	d) four storey	0.42	0.37	0.33	4 storey building not allowed in Zone V	



1. Door 3. Ventilator 2. Window 4. Cross wall

Sl No.	Item of Masonry	B	C	D	E	Action for Retrofitting, if Code Requirement not Found Satisfied
	2) b_4 minimum	340 mm	450 mm	560 mm	560 mm	Increase by build-up or reinforce with belt
iii)	Length of wall between cross walls	—	Maximum length = 35 × thickness or 8 m whichever less			If length more, provide pilaster or buttress
iv)	Height of wall from floor to ceiling	—	Maximum = 15 times thickness or 4 m whichever less			If height more, add pilaster to increase effective thickness
v)	Random — Rubble walls	'Through' or Header stones, one each in 0.72 m² surface area of wall. Long stones at corners of walls, in each wall in every alternate course.				If not provide, install RC Headers in holes made by removing stone
vi)	Horizontal seismic Bands: a) Plinth Level	Needed if soft (Type III) soil at base				Provide seismic belt, if plinth height ≥ 90 cm
	b) Door window lintel level	Needed in all cases with varying reinforcement and thickness specified in each case				Provide seismic belt of equivalent strength on both sides of walls
	c) Ceiling or eave level	Needed in sloping roofs or floors or roofs of prefab. materials				Repeat
	d) Gable or ridge wall	Needed in case of pitched roofs				Repeat
	e) Window sill level or dowels	Not required	Not required	Required in 3 and 4 storeyed buildings only	Required in all buildings	Repeat
vii)	Vertical bar at each corner and T-junction of wall	Needed in only 4 storey building	Needed in 3 and 4 storey buildings	Needed in all buildings	Needed in all buildings, (4 storeys not permitted)	Install equivalent bars or vertical belts at corners and T-junctions
viii)	Vertical bar at jambs of windows and doors	Not needed	Repeat	Repeat	Repeat	Install equivalent seismic belts around the opening

Alternatively, polymeric mortars may be used for grouting. The increase of shear strength which can be achieved in this way is considerable. However, grouting can not be relied on as far as the improving or making a new connection between orthogonal walls is concerned.

NOTE — The pressure needed for grouting can be obtained by gravity flow from super elevated containers.

9.2 Strengthening with Wire Mesh

Masonry walls with concentration of multiple cracks in the same portion and appearing on both sides on the wall or weak wall regions may be repaired with a layer of cement mortar or micro concrete layer 20 to 40 mm thick on both sides, reinforced with galvanized steel wire fabric (50 mm × 50 mm size) forming a vertical plate bonded to the wall. The two plates on either side of the wall should be connected by galvanized steel rods at a spacing of about 300 to 400 mm (*see* Fig. 3).

9.3 Connection Between Existing Stone Walls

In stone buildings of historic importance, consisting of fully dressed stone masonry in good mortar, effective sewing of perpendicular walls may be done by drilling inclined holes through them inserting steel rods and injecting cement grout (*see* Fig. 4).

9.4 Making 'Through' Bond Elements in R. R. Stone Wall (*see* Fig. 5)

a) Select points where 'through' stones will be installed at horizontal and vertical distance of about one meter apart, with 500 mm horizontal stagger.
b) Remove the plaster from the surface exposing the stones. Remove the mortar around the stone to sufficient depth gently, not violently, so as to expose the stone on all sides.
c) Loosen the stone by means of gentle pushes side ways and up and down by means of a small crowbar, so that the other stones of the walls are not disturbed. Pull out the stone slowly, holding it by both hands.
d) Remove inner material gradually so that a 75 mm size hole can be made in the wall. Bigger hole is not needed.
e) Locate position of the opposite stone on the

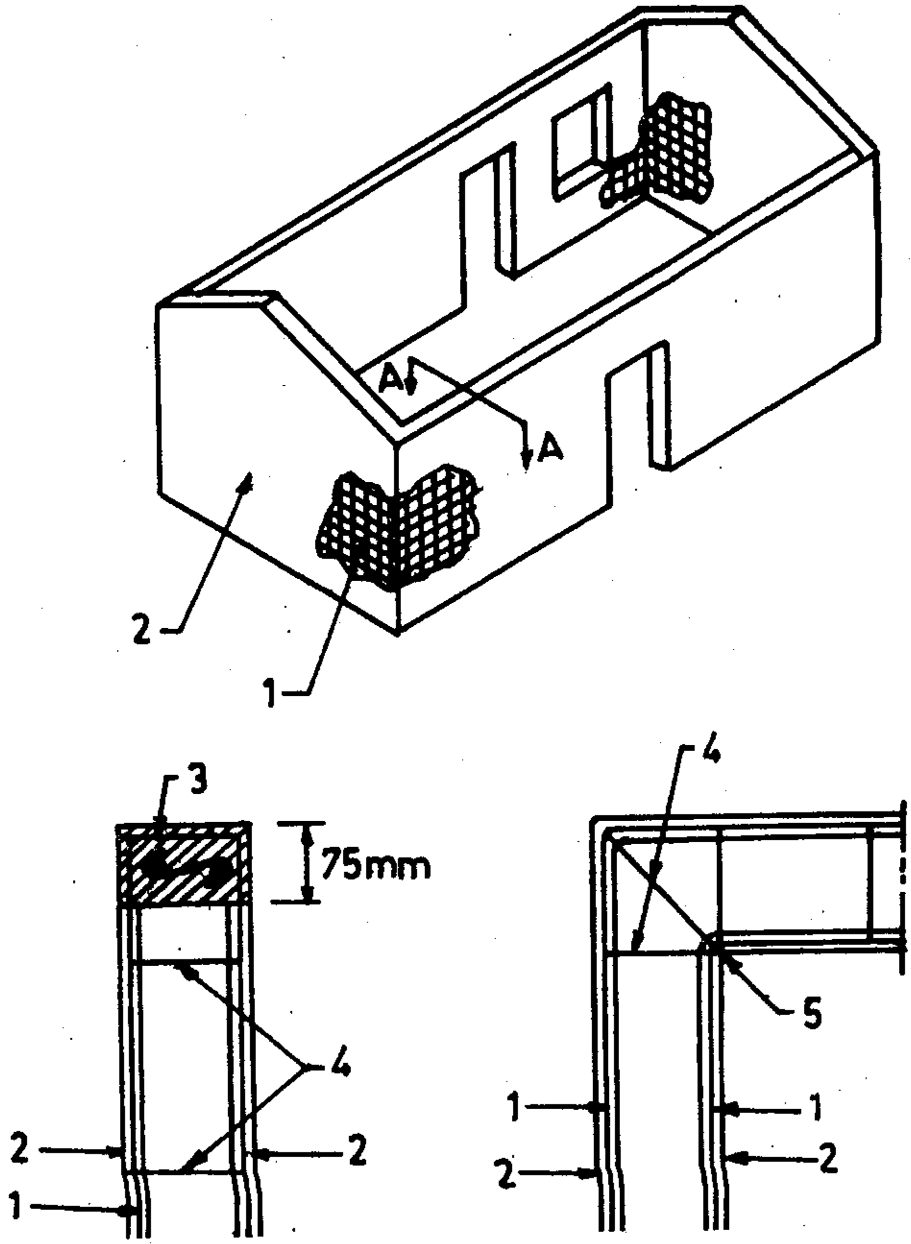
Table 4 Provisions for Roofs and Floors in IS 4326 and Actions for Retrofitting
(*Clause* 8.1)

Sl No.	Item of Roof/Floor	Requirement as per IS 4326 for Building Category B	C	D	E	Retrofitting Action, if Code Provision not Satisfied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Roof/floor with prefabricated/ pre-cast elements	Tie beam all round		All round tie beam and RC screed		Provide RC screed[1)] and seismic belt or band around
ii)	Roof/floor with wooden joists, various covering elements (brick, reeds, etc) and earth fill	—	All round seismic band and integration of units as a rigid horizontal diaphragm			Provide seismic belt around, interconnect beam ends through wooden planks and diagonal x-ties
iii)	Sloping roofs with sheet or tile coverings	—	i) Horizontal x-bracing at level of ties of the trusses ii) X-bracing in the planes of the rafters and purlins			Install the x-bracings, anchor trusses into walls and rafters into seismic belt at eave
iv)	Jack arch roof/floor	—	Connect the steel joists by horizontal ties at intervals to prevent spreading and cracking of the arches. Provide seismic band all round			Install steel flats as ties by welding them to the steel joists and provide seismic belt

[1)] RC screed — RC screed consists of minimum 14 mm concrete reinforcement with 6 mm dia bars @ 100 mm c/c both ways (single layer), covering the whole roof/floor.

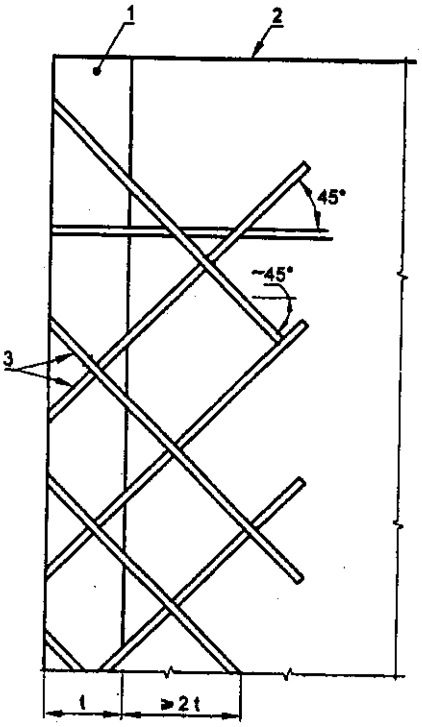
Table 5 Improvements Against Global Deficiencies
(*Clause* 8.1)

Sl No.	Item	B	C	D	E	Retrofitting Action if Code Provision not Satisfied
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Sloping raftered roofs	Preferably use full trusses				Convert rafters into A-frames or full trusses to reduce thrust on walls
ii)	Unsymmetrical plans	Symmetrical plans are suggested				Inserting new walls to reduce dissymmetry
iii)	Perpendicular walls not connected at corners and T-junctions	Perpendicular walls should be integrally constructed				Stitch the perpendicular walls using tie rods in drilled holes and grouted or with seismic belts



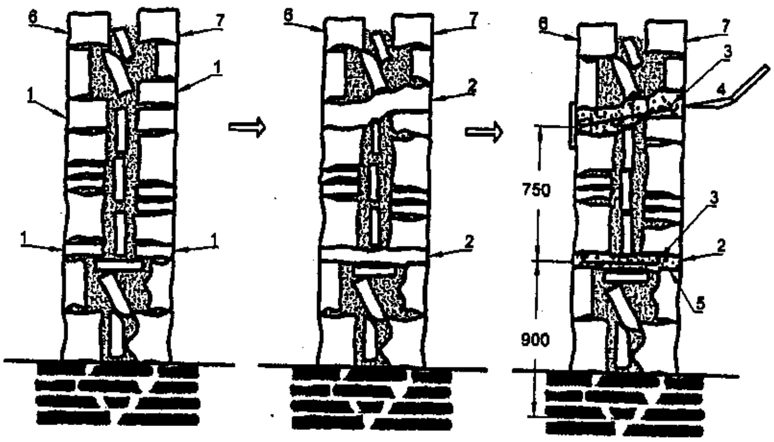
1 Welded wire mesh — 50 × 50 mm
2 Mortar or micro-concrete rendering
3 Concrete roof hand
4 Cross ties — 300 to 400 mm aparts
5 Corner bar ϕ 8

FIG. 3 STRENGTHENING WITH WIRE MESH AND MORTAR



1 Transverse wall
2 Longitudinal wall
3 Holes drilled through the junction of the two walls

FIG. 4 SEWING TRANSVERSE WALLS WITH INCLINED BARS



a) Existing Wall
b) Making Holes
c) Placing Bar and Filling Concrete

1 Stones removed to make through holes
2 Holes
3 Hooked bar
4 Chute for pouring concrete
5 Filled concrete
6 Internal wyhte
7 External wythe

FIG. 5 PROVIDING R.C. THROUGH ELEMENTS FOR STITCHING STONE WYTHE other face of the wall by gentle tapping in the hole. Remove the identified stone slowly by same gentle process.

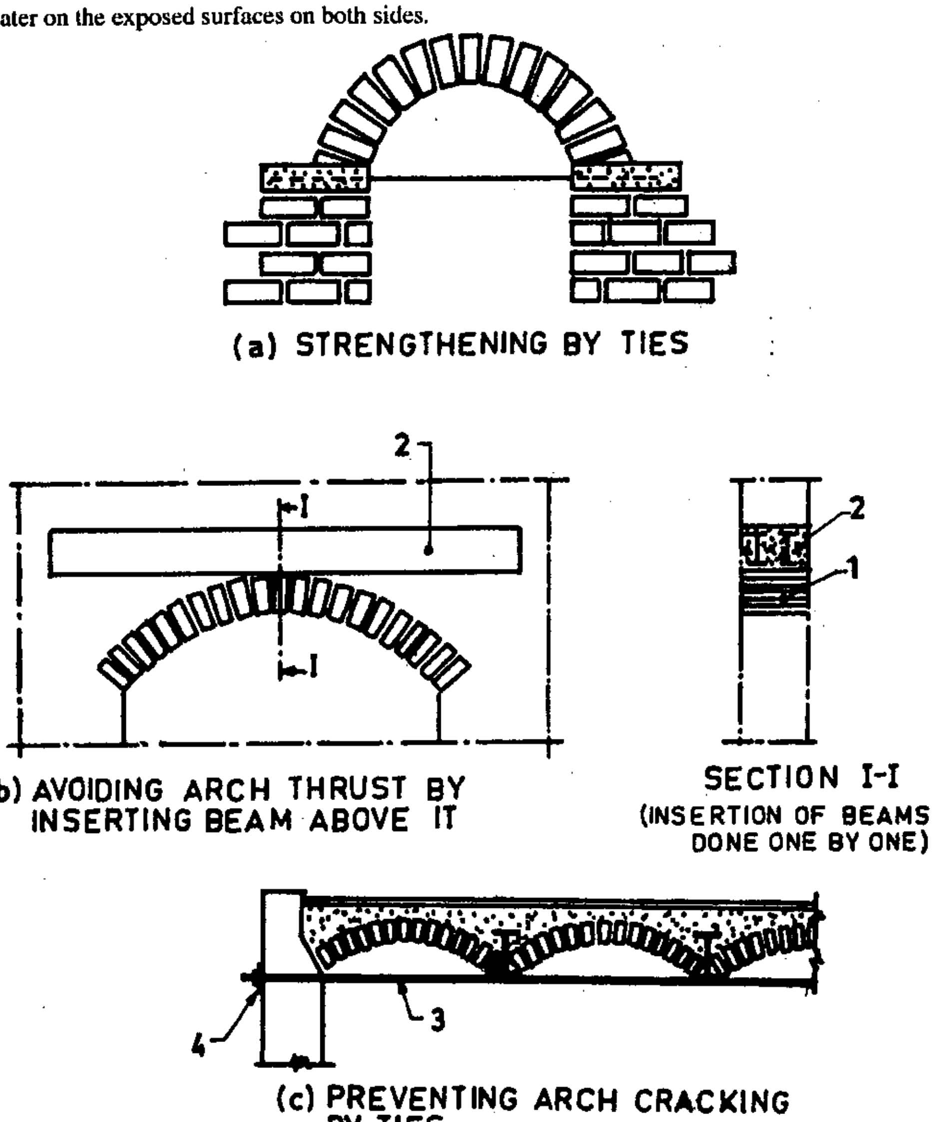
f) The hole so made through the wall may be bigger in size on both faces and narrower inside resembling a dumbbell shape. This is good. It does not matter if the hole is inclined instead of being horizontal.

g) Place concrete of 1 : 2 : 4 mix to fill half the depth of the hole from both sides and place 8 mm diameter hooked mild steel bar in the hole and fill the hole completely.

h) Cure for minimum 10 days by sprinkling water on the exposed surfaces on both sides.

9.5 Masonry Arches

If the walls have large arched openings in them, it will be necessary to install tie rods across them at springing levels or slightly above it by drilling holes on both sides and grouting steel rods in them [*see* Fig. 6(a)]. Alternatively, a lintel consisting of steel channels or I-shapes could be inserted just above the arch to take the load and relieve the arch as shown at Fig. 6(b). In jack-arch roofs, flat iron bars or rods shall be provided to connect the bottom flanges of I-beams connected by bolting or welding [*see* Fig. 6(c)].



1 Arch
2 Steel beam lintel
3 Flat iron or rod
4 Bearing plate

FIG. 6 STRENGTHENING AN ARCHED OPENING IN MASONRY WALL

10 SEISMIC BELTS AROUND DOOR/WINDOW OPENINGS

The jambs and piers between window and door openings require vertical reinforcement in the following situations:

a) In category D and E buildings for resistance against earthquake forces, and
b) For restoring the strength of the piers in any building category when badly damaged in an earthquake.

Where the above conditions specified in IS 4326 and IS 13828 are not satisfied, action has to be taken to close an opening or reduce its size.

The following mesh reinforcement is recommended to be used for covering the jamb area on both sides of an opening or for covering the pier between the consecutive openings (*see* Fig. 7):

a) *In category D and E buildings* — Mesh of gauge 10 with 8 wires in vertical direction spaced at 25 mm in a belt width of 200 mm or mesh of gauge 13 with wires @ 25 mm in a belt width of 250 mm may be used.
b) *In category C buildings* — Mesh of gauge 13 with 10 wires in vertical direction spaced at 25 mm in a belt width of 250 mm.

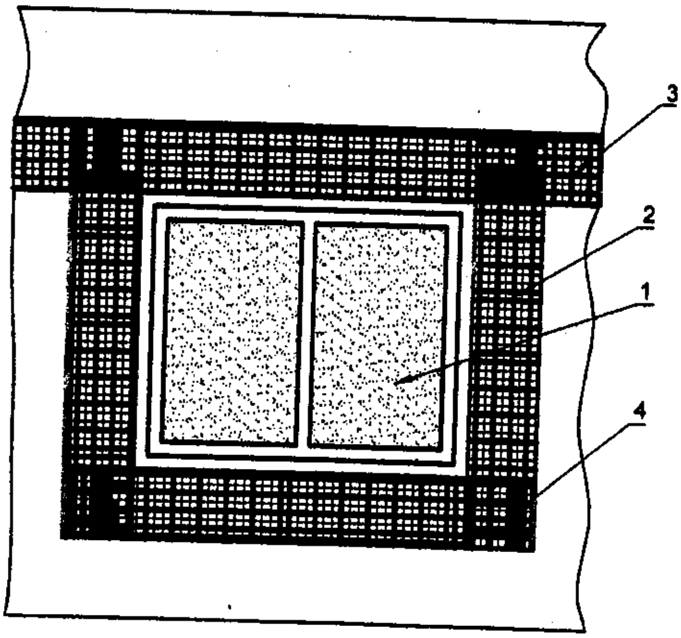
NOTE — For procedure of construction steps for seismic belt (*see* **11.3.3**).

11 ACHIEVING INTEGRAL BOX ACTION

The overall lateral strength and stability of bearing wall buildings is very much improved, if the integral box like action of room enclosures is ensured. This can be achieved by (a) use of pre-stressing, and (b) providing horizontal bands. Strength of shear walls is achieved by providing vertical steel at selected locations such as the corners and T-junction of walls.

11.1 Pre-stressing

A horizontal compression state induced by horizontal wires/bars can be used to increase the shear strength of walls. Moreover, this will also improve, considerably; the connections of orthogonal walls (*see* Fig. 8). The easiest way of affecting the pre-compression is to place two steel rods on the two sides of the wall and stretching them by turnbuckles.



1 Window
2 Mesh of ferro-cement
3 Seismic belt
4 Overlap of mesh

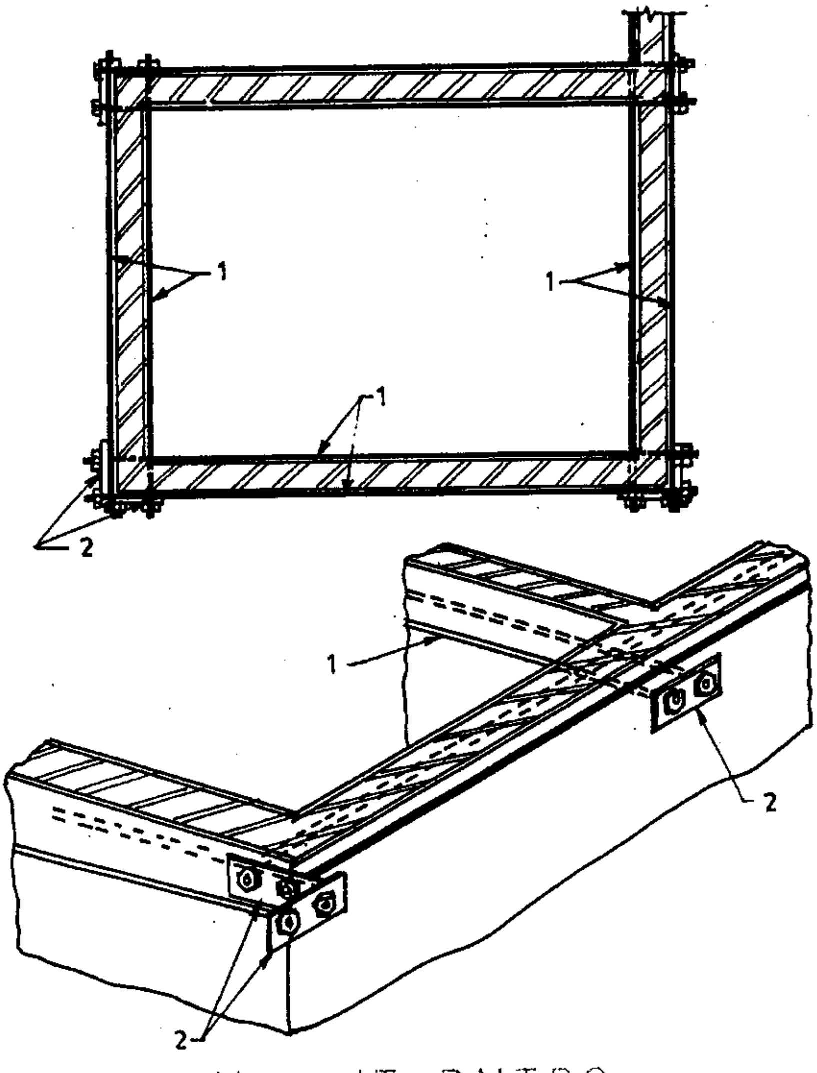
FIG. 7 REINFORCING AROUND OPENING

The vertical spacing of pre-stressing steel rods shall be 1/3 rd 2/3 rd of the height of the wall from bottom. Such pre-stressing elements may be provided near the top of wall in the case of sloping roof or roof consisting of pre-fabricated elements and an additional pre-stressing system at about mid height of the wall. In the case of RC slab roof such pre-stressing connection may be provided only near the mid height of the storey.

Note that, good effects can be obtained by slight horizontal pre-stressing (about 0.1 MPa) on the vertical section of the wall. Prestressing is also useful to strengthen spandrel beam between two rows of opening in the case no rigid slab exists. Opposite parallel walls can be held to internal cross walls by prestressing bars as illustrated above, the anchoring being done against horizontal steel channels instead of small steel plates. The steel channels running from one cross wall to the other will hold the walls together and improve the integral box like action of the walls.

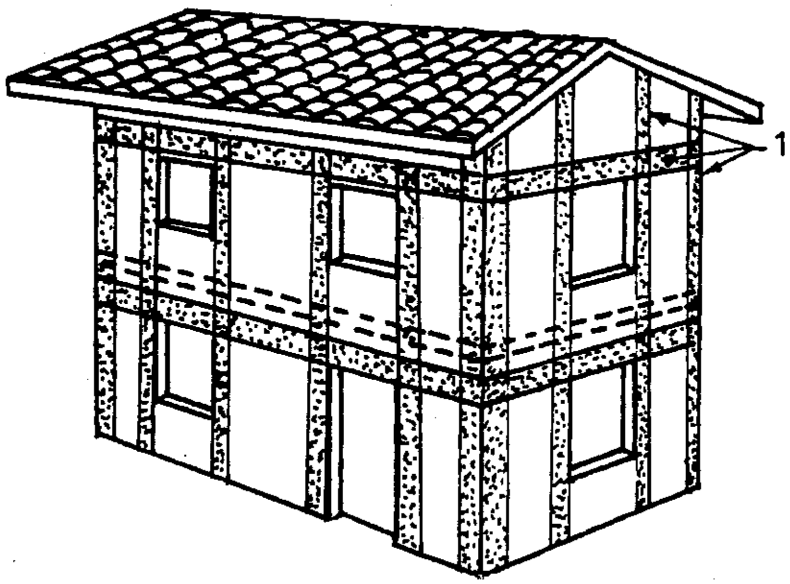
11.2 External Binding

The technique of covering the wall with steel mesh and mortar or micro-concrete may be used only on the outside surface of external walls but maintaining continuity of steel at the corners. This would strengthen the walls as well as bind them together. As a variation and for economy in the use of materials, the covering may be in the form of vertical splints located between the openings and horizontal 'bandages' formed over spandrel walls at suitable number of points only (*see* Fig. 9).



1 Steel rods for prestressing 2 Anchor plates

FIG. 8 STRENGTHENING OF WALLS BY PRE-STRESSING



1 Wire mesh with width ≥ 400 mm

FIG. 9 SPLINT AND BANDAGE STRENGTHENING TECHNIQUE

11.3 Providing Horizontal Seismic Belts

11.3.1 *Seismic Belt Locations*

a) Seismic belts are to be provided on all walls on both the faces just above lintels of door and window openings and below floor or roof,
b) The roof belt may be omitted if the roof or floor is of RCC slab,
c) Seismic belt is not necessary at plinth level, unless the plinth height is more than 900 mm, and
d) Install similar seismic belt at the eave level of sloping roof and near top of gable wall, below the roof.

NOTES

1 On small wall lengths in a room (less than 5 m) seismic band only on the outside face will suffice. In this case these should be connected by ties going across the rooms at about 2.5 m apart (*see* Fig. 10).

2 If the height of eave level above the top of door is less than 900 mm, only the eave level belt may be provided and lintel level belt may be omitted.

11.3.2 *Description of Reinforcement in Belt*

The reinforcement may be of mesh types as suggested in Table 6 or any other mesh of equivalent longitudinal wires. For example in Category D building with room length of 6 m, MW 21 weld mesh (with long wires 5 of 4.5 mm diameter spaced at 75 mm apart; cross wires

Table 6 Mesh Reinforcement in Seismic Belts in Various Building Categories

Sl No.	Length of Wall	Category B			Category C			Category D			Category E		
	M	Gauge	N	H	Gauge	N	H	Gauge	N	H	Gauge	N	H
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	≤ 5.0	g 14	9	250	g 13	9	250	g 12	9	250	g 10	10	280
ii)	6.0	g 13	9	250	g 12	9	250	g 10	10	280	g 10	14	380
iii)	7.0	g 12	9	250	g 10	10	280	g 10	14	380	g 10	18	460
iv)	8.0	g 10	9	250	g 10	14	380	g 10	18	460	g 10	23	580

NOTES

1 Gauges: g 10 = 3.25 mm, g 11 = 2.95 mm, g 12 = 2.64 mm, g 13 = 2.34 mm, g 14 = 2.03 mm.

2 N = Number of made longitudinal wires in the belt at spacing of 25 mm.

3 H = Height of belt on wall in micro-concrete, in mm.

4 The transverse wires in the mesh could be spaced upto 150 mm.

5 The mesh should be galvanized to save from corrosion.

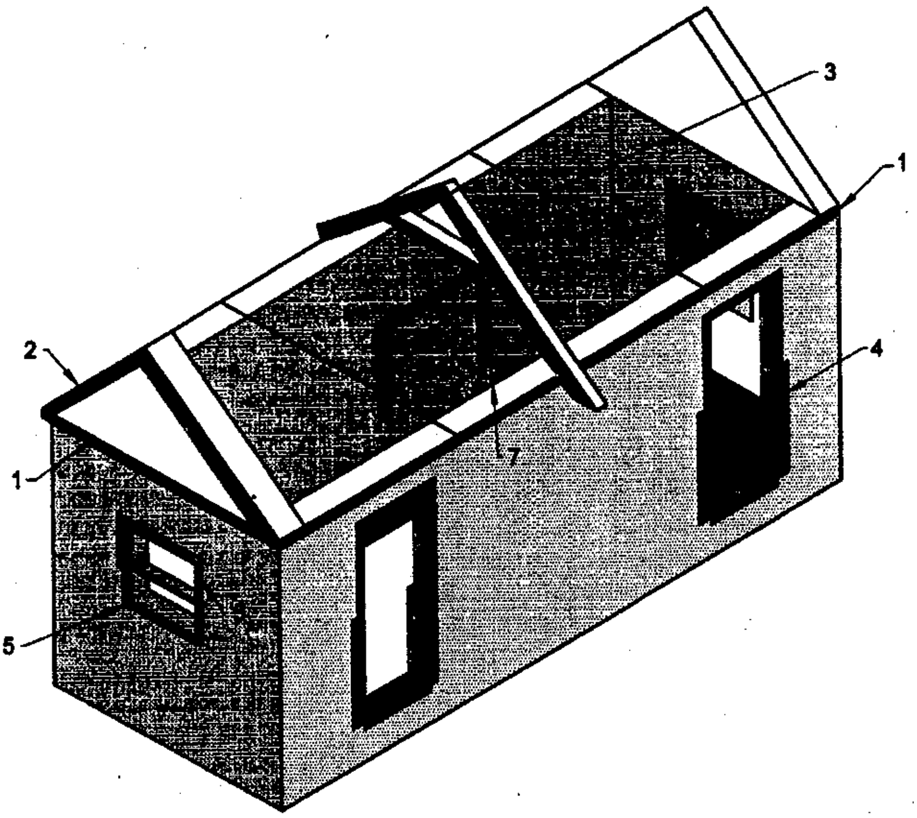


FIG. 10 OVERALL ARRANGEMENT OF SEISMIC BELTS

of 3.15 mm diameter placed at 300 mm apart) can be used, the height of the belt being kept as 375 mm.

NOTE — Weld mesh has to be provided continuously. If splicing is required, there shall be minimum overlap of 300 mm.

11.3.3 *Procedure for Construction of Seismic Belt*

It consists of a galvanized iron mesh fixed to the walls through nails or connector-links drilled through the wall thickness and the mesh is covered by rich mix of cement-sand mortar in the ratio of 1 : 3 to achieve good results, the following step-wise procedure is to be followed:

a) Mark the height or width of the desired belt based on the weld mesh number of longitudinal wires and the mesh size,

b) Cut the existing plaster at the edge by mechanical cutter for neatness, and remove the plaster,

c) Rake the exposed joints to a depth of 20 mm. Clean the joints with water jet,

d) Apply neat cement slurry and plaster the wall with 1.3 cement — coarse sand mix by filling all raked joints fully and covering the wall with a thickness of 15 mm. Make the surface rough for better bond with the second layer of plaster,

e) Fix the mesh to be plastered surface through 15 cm long nails driven into the wall at a spacing of 45 cm tying the mesh to the nails by binding wire,

f) Now apply the second layer of plaster with a thickness of 15 mm above the mesh. Good bonding will be achieved with the first layer of plaster and mesh if neat cement slurry is applied by a brush to the wall and the mesh just in advance of the second layer of plaster, and

g) Cure the plaster by sprinkling water for a minimum period of 10 days.

NOTE — Where the RC belt is provided on both faces of the wall, the nails should be replaced by anchors through drilled holes filled with mortar grout and tied to the meshes on both faces.

11.4 Vertical Seismic Belt at Corners

Vertical reinforcing is required at the corners of rooms and junctions of walls as per Table 7. The width of this belt on each side of the corner has to be kept 25 mm extra to the width of the mesh.

This reinforcement should be started 300 mm below the plinth level and continued into the roof/eave level horizontal belt (*see* Fig.11).

11.5 Providing Vertical Reinforcement at Corners, Junctions of Walls

The vertical reinforcement consisting of HSD bar as per Table 7 or equivalent shall be provided on the inside corner of room starting from 750 mm below the ground floor going upto the roof slab, passing through each middle floor through holes made in the slabs (*see* Fig.12). The reinforcement shall be connected to the walls by using L shape dowels of 8 mm TOR bar, the vertical leg of 400 mm length firmly tied to the vertical reinforcement bars and the horizontal leg of minimum 150 mm length embedded in the walls through 75 mm diameter holes drilled in the wall into which the 8 mm diameter leg of the dowel will be grouted using non-Shrink cement cum polymer grout. Such dowels will be provided, first one just above plinth level and then at about every 1 m distance apart. The corner reinforcement will be covered with 1 : 3 cement mortar or 1 : 1 ½ : 3 micro concrete fully bonded with the walls giving a minimum cover of 15 mm on the bar.

12 MODIFICATION OF ROOFS OR FLOORS

12.1 Slates and roofing tiles are brittle and easily dislodged. Where possible, they should be replaced with corrugated iron or asbestos sheeting.

12.2 False ceilings of brittle material are dangerous. Non-brittle material, like hessian cloth, bamboo matting or light ones of foam substances, may be substituted.

12.3 Roof truss frames should be braced by welding or clamping suitable diagonal bracing members in the vertical as well as horizontal planes.

12.4 Anchors of roof trusses to supporting walls should be improved and the roof thrust on walls should be eliminated.

Figures 13 and 14 illustrate one of the methods for pitched roofs without trusses.

12.5 Where the roof or floor consists of prefabricated units like RC rectangular T or channel units or wooden poles and joists carrying brick tiles, integration of such units is necessary. Timber elements could be connected to diagonal planks nailed to them and spiked to an all round wooden frame at the ends. Reinforced concrete elements may either have 40 mm cast-in-situ concrete topping with 6 mm diameter bars 150 mm c/c both ways or bounded by a horizontal cast-in-situ reinforced concrete ring beam all round into which the ends of reinforced concrete elements are embedded. Figure 15 shows one such detail.

Table 7 Vertical Bar or Mesh Reinforcement in Vertical Belt at Corners of Rooms
(*Clause* 11.4)

Sl No.	No. of Storeys	Storeys	Category B			Category C			Category D			Category E		
			Single Bar mm	Mesh (g 10)		Single Bar mm	Mesh (g 10)		Single Bar mm	Mesh (g 10)		Single Bar mm	Mesh (g 10)	
				N	B		N	B		N	B		N	B
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
i)	One	One	—	—	—	—	—	—	10	10	300	12	14	400
ii)	Two	Top	—	—	—	—	—	—	10	10	300	12	14	400
iii)		Bottom	—	—	—	—	—	—	12	14	400	16	—	—
iv)	Three	Top	—	—	—	10	10	300	10	10	300	12	14	400
v)		Middle	—	—	—	10	10	300	12	14	400	16	25	650
vi)		Bottom	—	—	—	12	14	400	12	14	400	16	25	650

NOTES

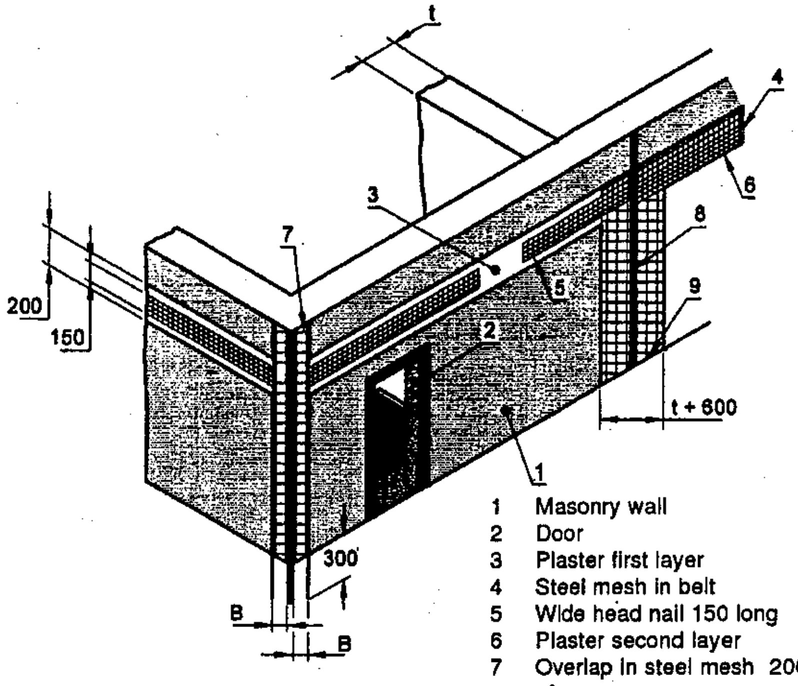
1 Gauge 10 (3.25 mm dia) galvanized mesh with 25 mm spacing of wires shall be used.

2 Single bar, if used, shall be HSD or TOR type. If two bars are used at a T-junction, the diameter can be taken as follows. For one of 10 or 12 mm take 2 of 8 mm, and for one of 16 mm take 2 of 12 mm.

3 N = Number of longitudinal wires in the mesh.

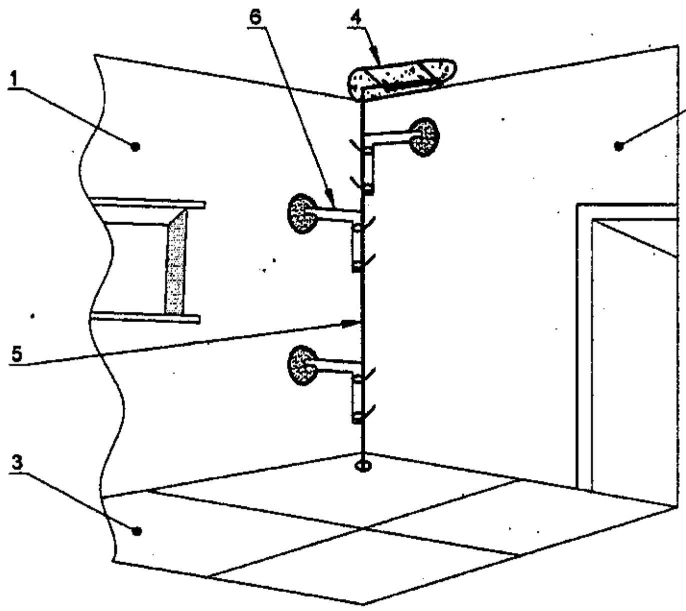
4 B = Width of the micro concrete belt, half on each wall meeting at the corner or T-junction.

5 The transverse wires in the mesh could be at spacing up to 150 mm.



1 Masonry wall
2 Door
3 Plaster first layer
4 Steel mesh in belt
5 Wide head nail 150 long
6 Plaster second layer
7 Overlap in steel mesh 200 mm each side of corner
8 Vertical seismic belts junction (going across horizontal belt)
9 Spikes for fixing mesh

FIG. 11 VERTICAL SEISMIC BAND AT CORNER AND JUNCTIONS



1 Wall
2 Perpendicular wall
3 Floor
4 Slas
5 Vertical bar
6 Dowel

DETAIL AT A

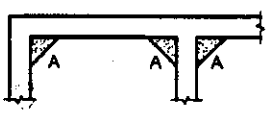
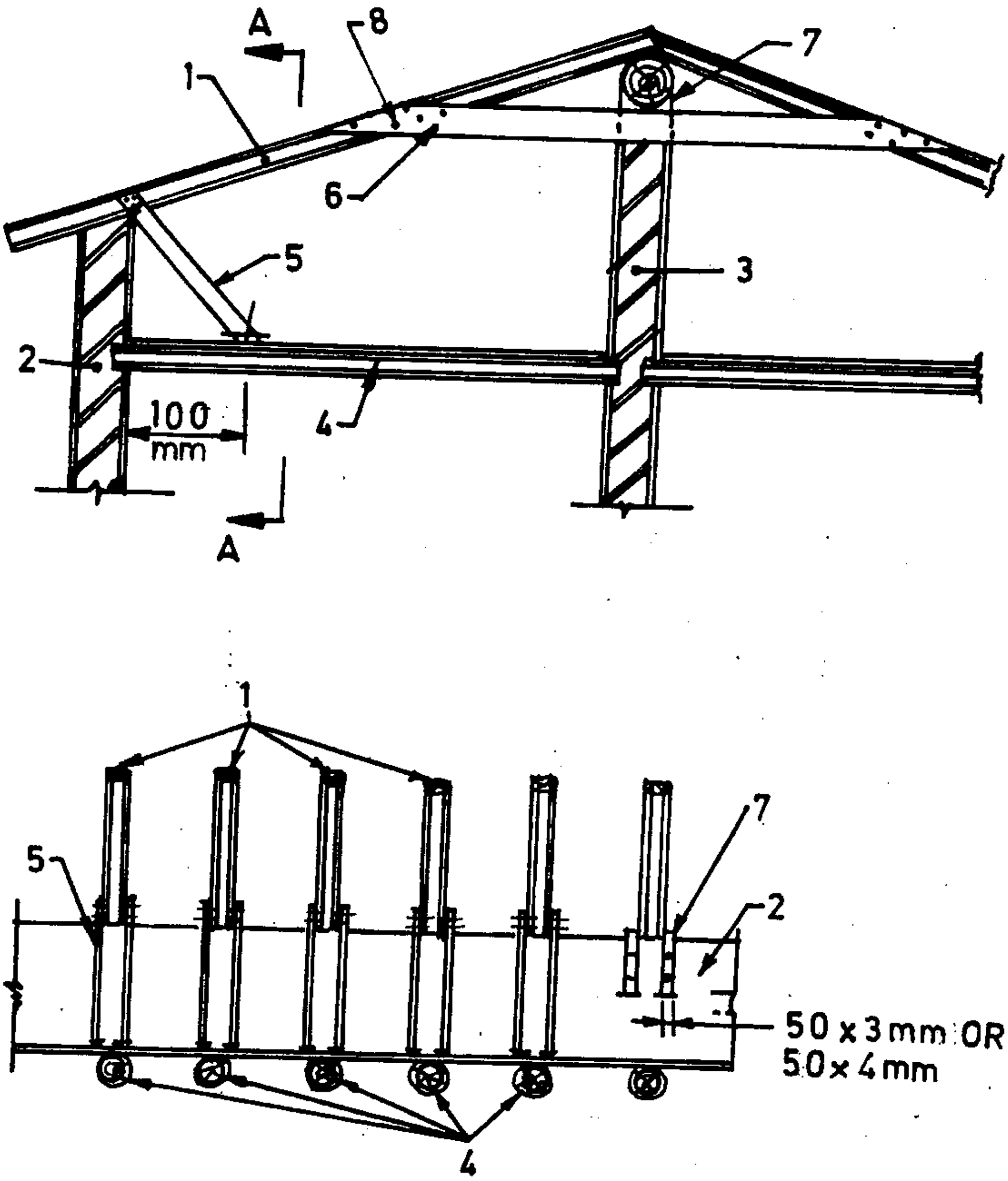


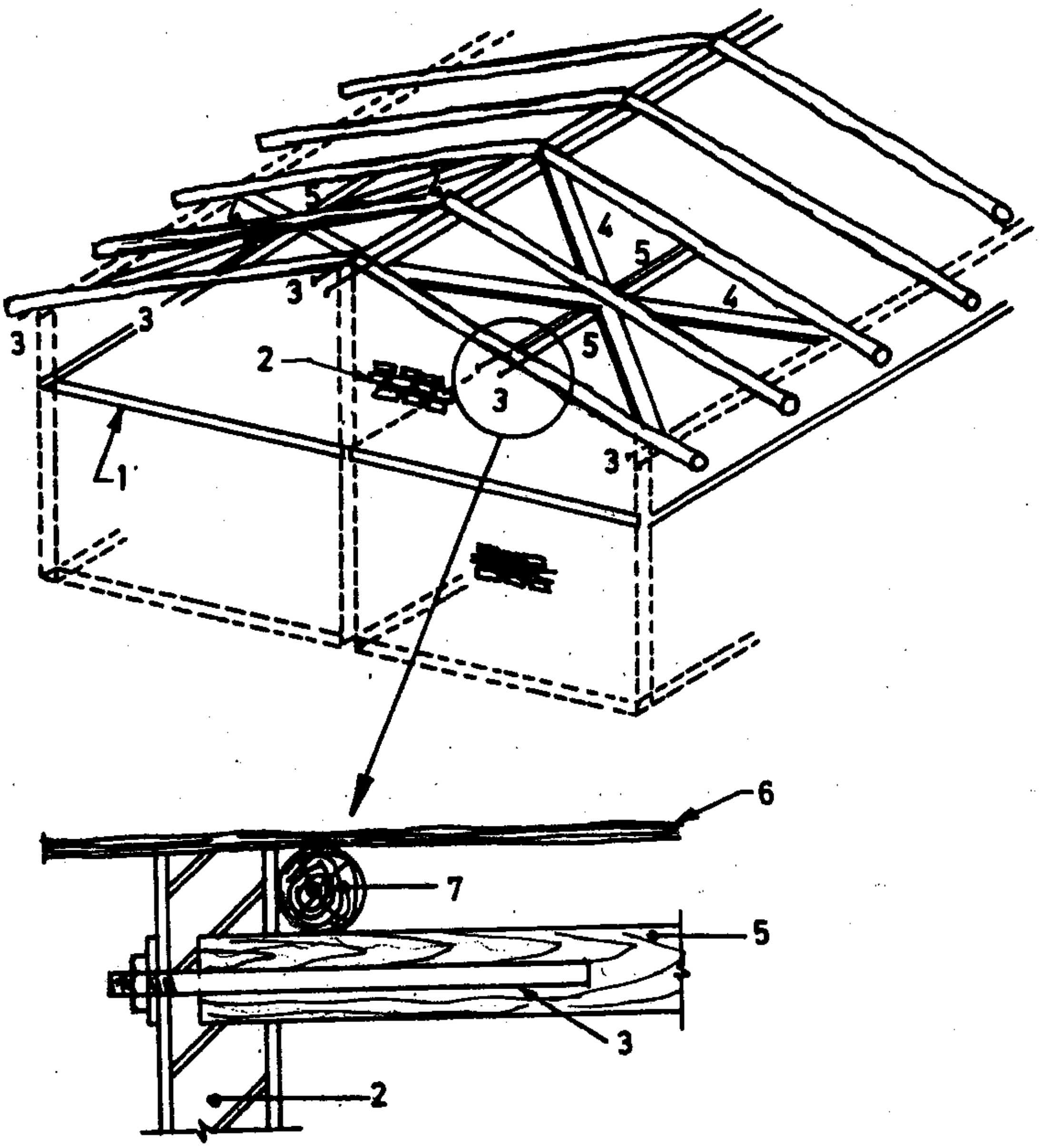
FIG. 12 VERTICAL BAR AT INSIDE CORNER



SECTION AA

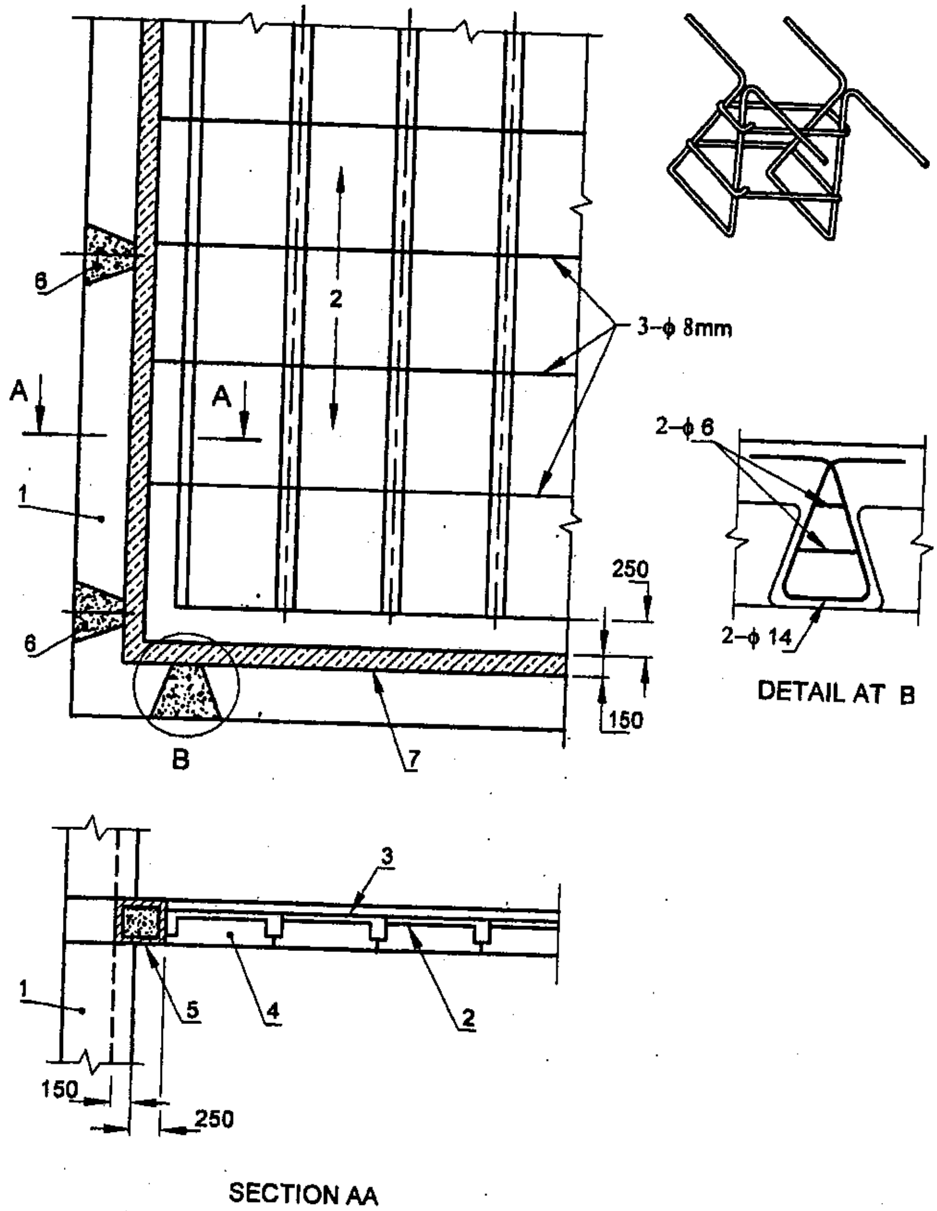
1 Existing rafters
2 Existing outer wall
3 Existing inner wall
4 Existing floor beam
5 New planks 200 × 40 mm nailed at ends
6 New planks 200 × 40 mm nailed at ends to take rafter thrust
7 U-Steel anchor clamp bolted to existing wall at 3 to 4 m apart
8 Nails

FIG. 13 ROOF MODIFICATION TO REDUCE THRUST OF WALLS



1 Existing floor
2 Existing gable wall (tympanum)
3 Steel strips bolted to new ties (5) and wall (2)
4 New planks, diagonal bracing
5 New planks, ties
6 Roof covering
7 Existing roof rafters

FIG. 14 DETAILS OF NEW ROOF BRACING



1 Existing wall
2 Existing or new pre-fabrication floor
3 Slab topping with reinforcement
4 Prefab slab units
5 R.C. band
6 Key connecting new floor to existing wall @ 3M
7 Grooves cut in wall

All dimensions in millimetres.

FIG. 15 INTEGRATION AND STIFFENING OF AN EXISTING FLOOR

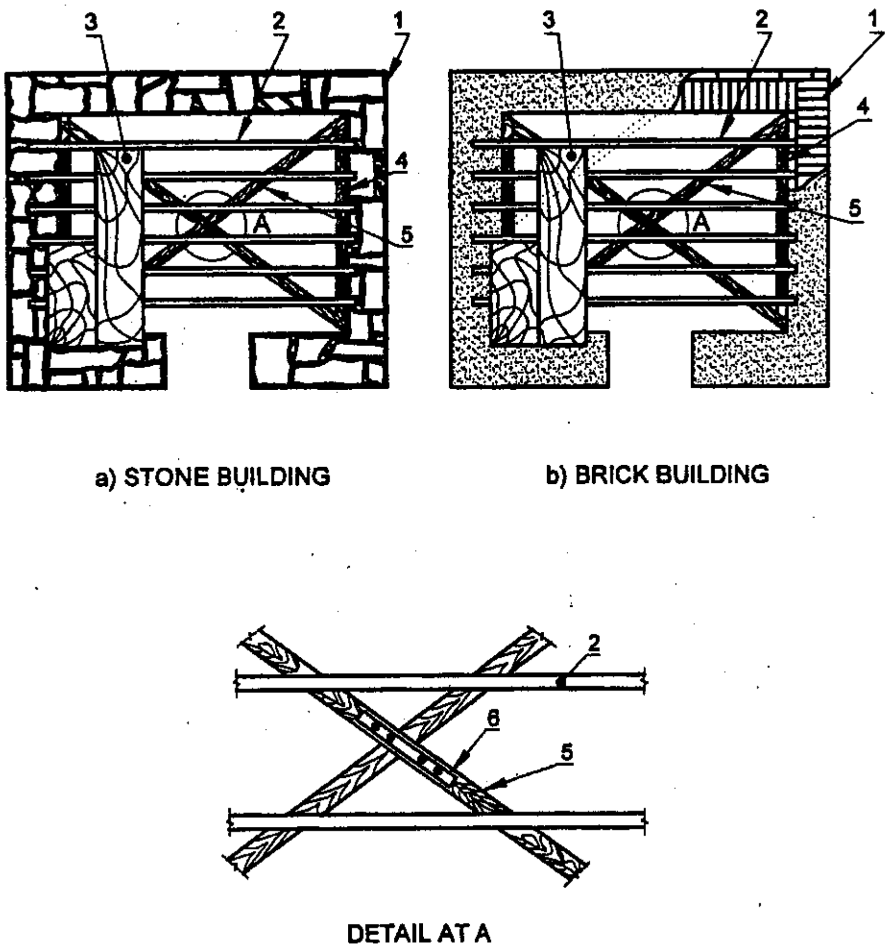
12.6 Roofs or floors consisting of steel joists flat or segmental arches must have horizontal ties holding the joists horizontally in each arch span so as to prevent the spreading of joists. If such ties do not exist, these should be installed by welding or clamping.

12.7 Stiffening the Flat Wooden Floor/Roof

Many of the houses have flat floor or roof made of wood joists covered with wooden planks and earth. For making such roof/floor rigid, long planks 100 mm wide and 25 mm thick should be nailed at both ends of the joists from below. Additionally, similar planks or galvanized metal strips 1.5 mm thick 50 mm wide should be nailed diagonally also (*see* Fig. 16).

12.8 Stiffening the Sloping Roof Structure

Most of the sloping roofs are made of rafters, purlins, and burnt clay tiles on top. Similarly AC or CGI sheet roofs are made using wooden purlins resting on gable walls or main rafters. But trusses were not formed which require the use of ties. Such roofs push the walls outward during earthquakes.



1 Wall
2 Wood joist
3 Wood plank
4 Tile plank under ends of joist
5 Diagonal ties
6 Joint by nailling through 3 mm flat iron

Fig. 16 Stiffening Flat Wooden Floor/Roof

For stiffening such roofs, the rafters should be tied with the seismic belt as in Notes 1 and 2 under **11.3.1**, and the opposite rafters, on both sides of the ridge need to be connected near about mid-height of the roof through cross ties nailed to the rafters (*see* Fig.17).

13 INSERTING NEW WALLS

13.1 Unsymmetrical buildings which may produce dangerous torsional effects during earthquakes the centre of masses can be made coincident with the center of stiffness by separating parts of buildings thus achieving individual symmetric units and/or inserting new vertical resisting elements such as new masonry or reinforced concrete walls either internally as shear walls or externally as buttresses. Insertion of cross wall will be necessary for providing transverse supports to longitudinal walls of long barrack-type buildings used for various purposes such as schools and dormitories.

13.2 The main problem in such modifications is the connection of new walls with old walls. Figures 18, 19 and 20 show three examples of connection of new walls to existing ones. The first two cases refer to a T-junction

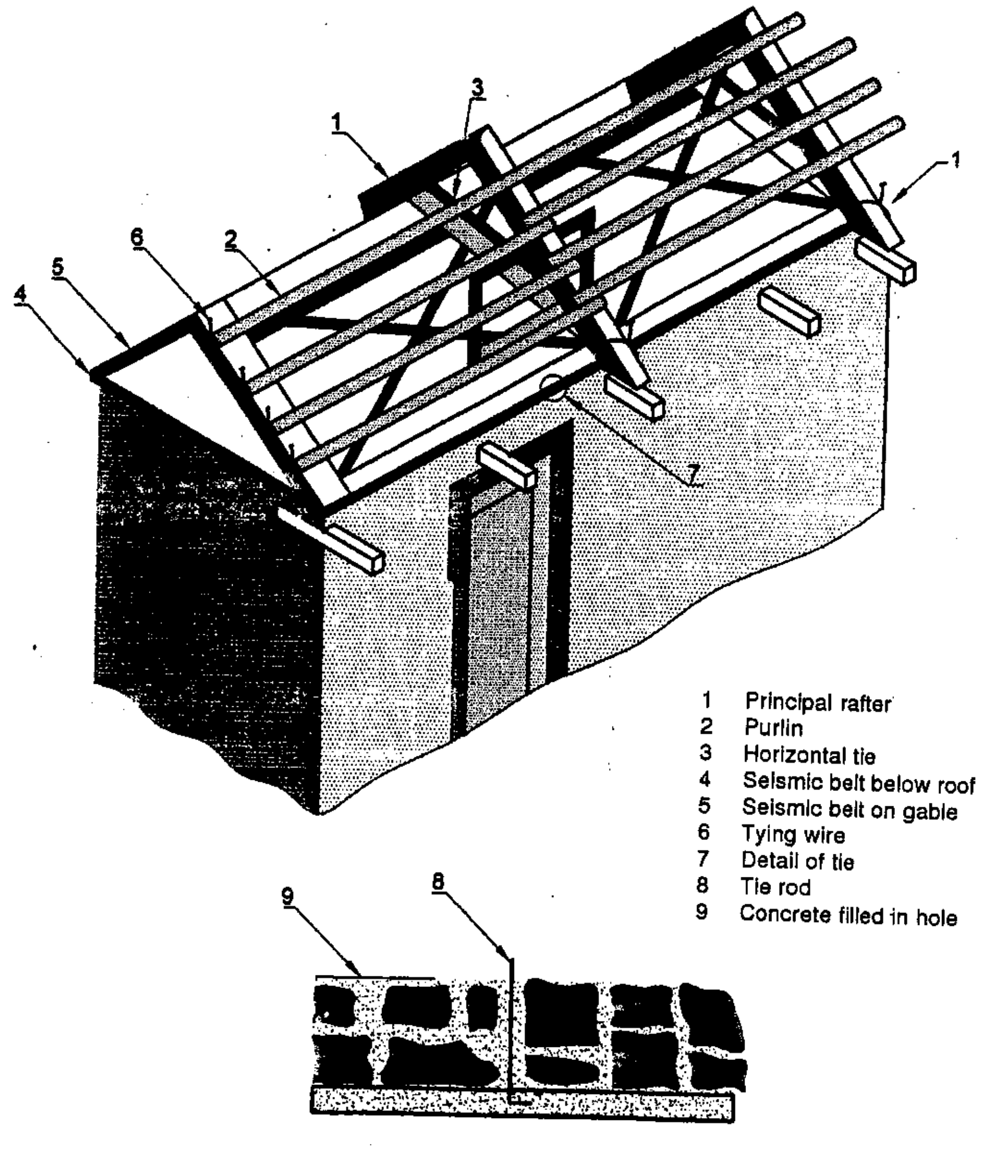
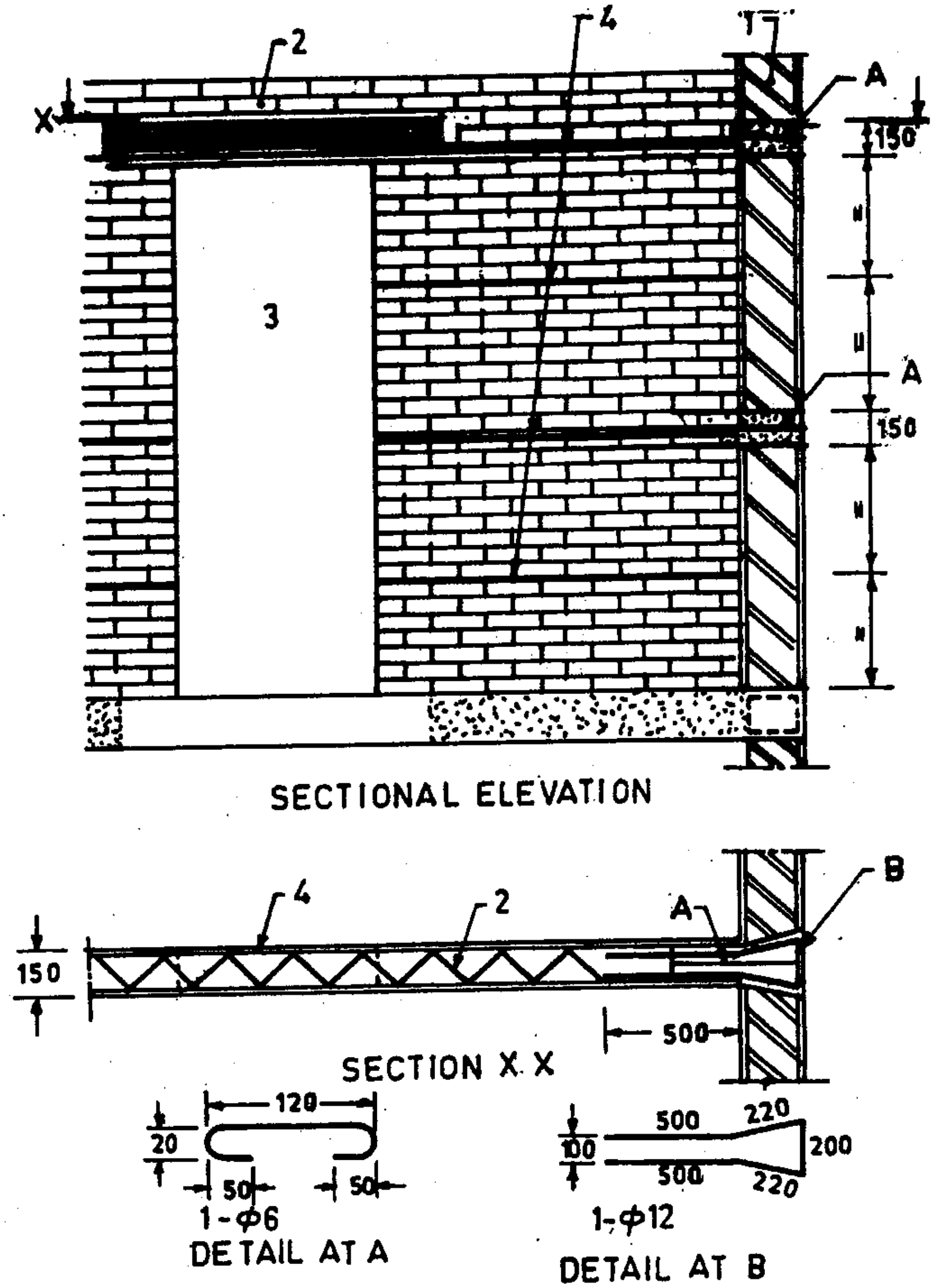


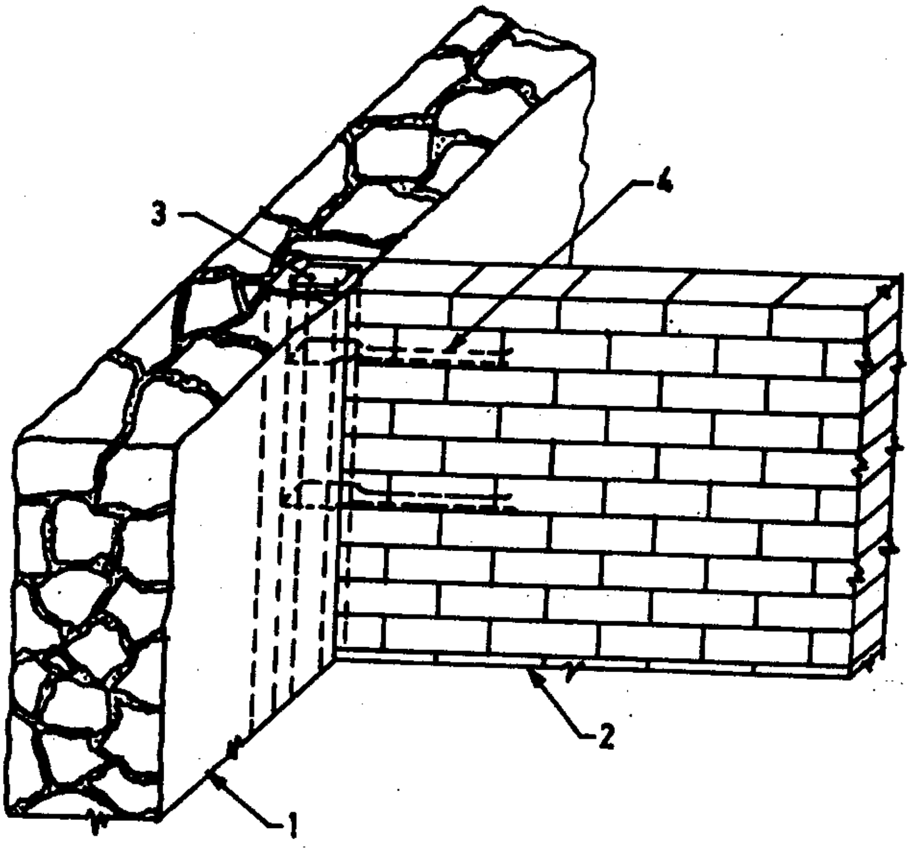
FIG. 17 STIFFENING OF SLOPING ROOF STRUCTURE



1 Existing wall
2 New wall
3 Door opening
4 Horizontal reinforcement (example of truss system shown)

All dimensions in millimetres.

FIG. 18 CONNECTION OF NEW AND OLD BRICK WALLS



1 Existing old wall
2 New wall
3 Concrete column for bonding
4 Connecting ties of steel, every fourth course

FIG. 19 CONNECTION OF NEW BRICK WALL WITH EXISTING STONE WALLS

whereas the third to a corner junction. In all cases the link to the old walls is performed by means of a number of keys made in the old walls. Steel is inserted in them and local concrete infilling is made. In the second case, however, connection can be achieved by a number of steel bars inserted in small length drilled holes filled with fresh cement-grout which substitute keys.

14 STRENGTHENING OF FOUNDATIONS

Strengthening of foundations before or after the earthquake is the most involved task since it may require careful underpinning operations. Some alternatives are given below for preliminary consideration of the strengthening scheme:

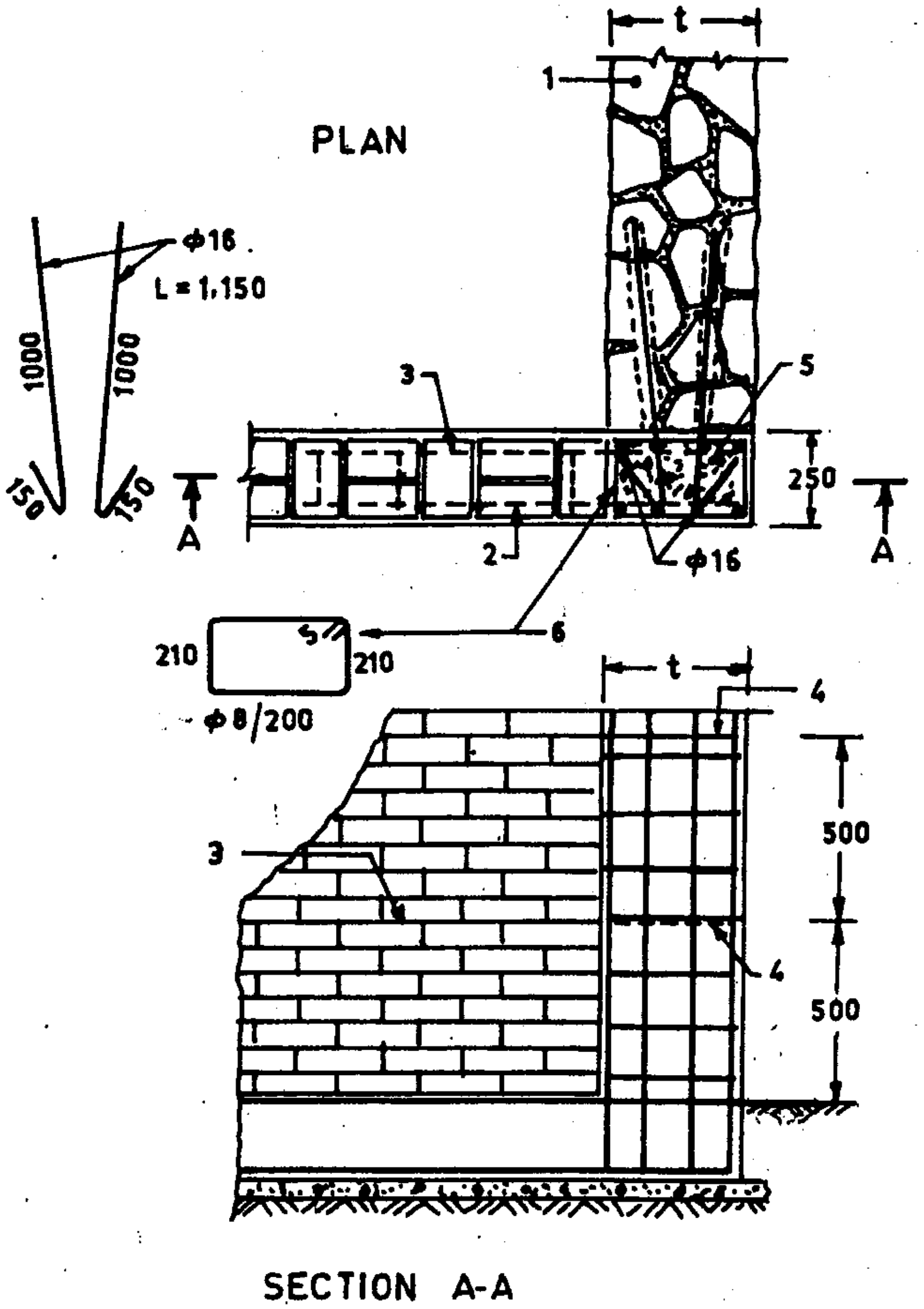
a) Introducing new load bearing members including foundations to relieve the already loaded members. Jacking operations may be needed in this process.

b) Improving the drainage of the area to prevent saturation of foundation soil to obviate any problems of liquefaction which may occur because of poor drainage.

c) Providing apron around the building to prevent soaking of foundation directly and draining off the water.

d) Adding strong elements in the form of reinforced concrete strips attached to the existing foundation part of the building. These will also bind the various wall footings and may be provided on both sides of the wall (*see* Fig. 21) or only one side of it. In any case, the reinforced concrete strips and the wall have to be linked by a number of keys inserted into the existing footing.

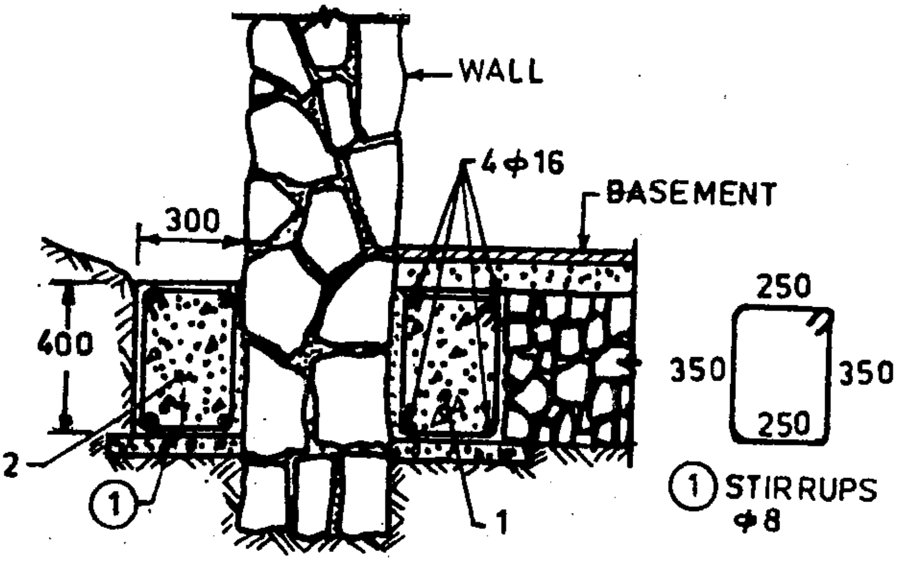
NOTE — To avoid disturbance to the integrity of the existing wall during the foundation strengthening process proper investigation and design is called for.



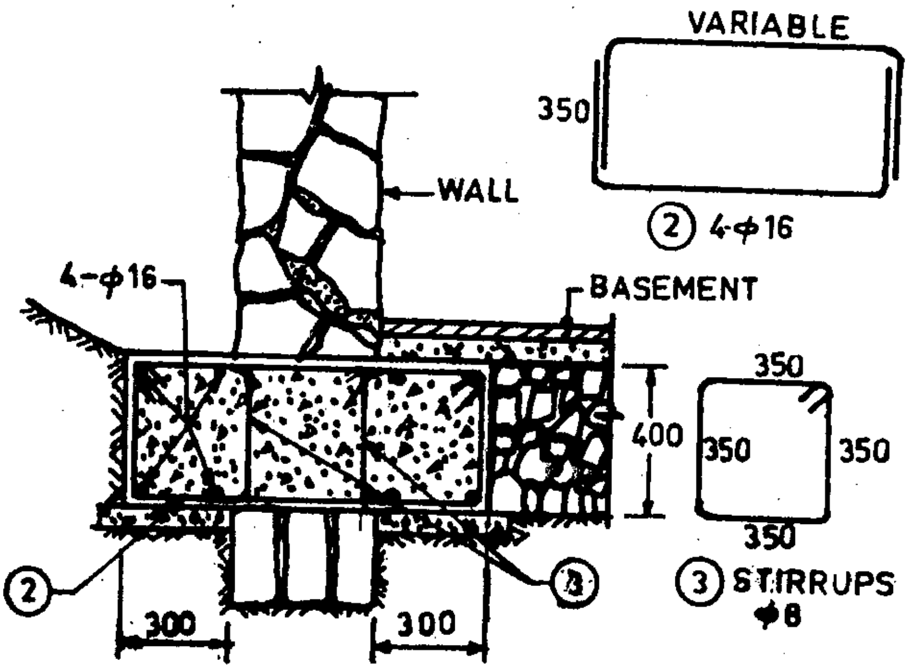
1 Existing wall, thickness, T
2 New wall
3 Horizontal reinforcement with links
4 Connection steel grouted in drilled holes
5 Concrete in column and footing
6 Stirrups

All dimensions in millimetres.

FIG. 20 CONNECTION OF NEW AND OLD WALLS (CORNER JUNCTIONS)



21A



21B

All dimensions in millimetres.

FIG. 21 STRENGTHENING EXISTING FOUNDATION (R. C. STRIP ON BOTH SIDES)

ANNEX A

(*Clause* 7)

RAPID VISUAL SCREENING OF MASONRY BUILDINGS

A-1 RVS PROCEDURE, OBJECTIVES AND SCOPE

The rapid visual screening method is designed to be implemented without performing any structural calculations. The procedure utilizes a damageability grading system that requires the evaluator to (a) identify the primary structural lateral load-resisting system, and (b) identify building attributes that modify the seismic performance expected for this lateral load-resisting system along with non-structural components. The inspection, data collection and decision-making process typically occurs at the building site, and is expected to take couple of hours for a building, depending on its size.

The screening is based on code based seismic intensity, building type and damageability Grade as observed in past earthquake and covered in MSK/European macro-intensity.

A-2 USES OF RVS RESULTS

The main uses of this procedure in relation to seismic upgrading of existing buildings are:

a) to identify, if a particular building requires further evaluation for assessment of its seismic vulnerability;
b) to assess the seismic damageability (structural vulnerability) of the building and seismic rehabilitation needs;
c) to identify simplified retrofitting requirements for the building (to collapse prevention level) where further evaluations are not considered necessary or not found feasible.

A-3 SEISMIC HAZARD IN INDIA

As per IS 1893 (Part 1), India has been divided into 4 seismic hazard zones. The details of different seismic zones are given below:

a) *Zone II* — Low seismic hazard (damage during earthquake may be of MSK Intensity VI or lower)
b) *Zone III* — Moderate seismic hazard (maximum damage during earthquake may be upto MSK Intensity VII)
c) *Zone IV* — High seismic hazard (maximum damage during earthquake may be upto MSK Intensity VIII)
d) *Zone V* — Very high seismic hazard (maximum damage during earthquake may be of MSK Intensity IX or greater)

When a particular damage Intensity occurs, different building types experience different levels of damage depending on their inherent characteristics. For carrying out the rapid visual screening, all four hazard zones have been considered.

A-4 BUILDING TYPES CONSIDERED IN RVS PROCEDURE

A wide variety of construction types and building materials are used in urban and rural areas of India. These include local materials such as mud, straw and wood, semi-engineered materials such as burnt brick and stone masonry and engineered materials such as concrete and steel. The seismic vulnerability of the different building types depends on the choice of building materials and construction technology adopted. The building vulnerability is generally highest with the use of local materials without engineering inputs and lowest with the use of engineered materials and skills.

The basic vulnerability class of a building type is based on the average expected seismic performance for that building type. All buildings have been divided into Type A to Type D based on the MSK Scale. The buildings in Type A have the highest seismic vulnerability while the buildings in Type D have the lowest seismic vulnerability. A building of a given type, however, may have its vulnerability different from the basic class defined for that type depending on the condition of the building, presence of earthquake resistance features, architectural features, number of storeys etc. It is therefore possible to have a damageability range for each building type considering the different factors affecting its likely performance. Some variations in building type are therefore defined as A, A$^+$, B, B$^+$, etc.

The RVS procedure presented in the standard has considered different building types, based on the building materials and construction types that are most commonly found in India. Masonry buildings are presented in Table 8. The likely damages to buildings have been categorized in different Grades depending on the seismic impact on the strength of the building.

A-5 GRADES OF DAMAGEABILITY

Five grades of damageability from G1 to G5 are specified in MSK and European Intensity Scale as described in Table 9 and Table 10.

Table 8 Masonry Load Bearing Wall Buildings
(*Clause* A-4)

Sl No. (1)	Building Type (2)	Description (3)
i)	A	a) Rubble (Field stone) in mud mortar or without mortar usually with sloping wooden roof
		b) Mud walls. Adobe walls of two storeys
		c) UCR masonry without adequate through stones
		d) Masonry with rounded (undresses) stones
ii)	A+	a) Adobe (unburnt block or brick) walls of single storey
		b) Rammed earth / Pise construction
iii)	B	a) Semi-dressed, rubble, brought to courses, with through stones and long corner stone unreinforced brick walls with country type wooden roofs; unreinforced CC block wal constructed in mud mortar or weak lime mortar
		b) Earthen walls (Adobe, Rammed earth) with horizontal wooden elements
iv)	B+	a) Unreinforced brick masonry in mud mortar with vertical wood posts or horizontal wood elemen or seismic band (IS 13828)
		b) Unreinforced brick masonry in lime mortar
v)	C	a) Unreinforced masonry walls built from fully dressed (Ashlar) stone masonry or CC block or burn brick using good lime or cement mortar, either having RC floor/roof or sloping roof having eav level horizontal bracing system or seismic band
		b) As at B(a) with horizontal seismic bands (IS 13828)
vi)	C+	Like C(a) type but having horizontal seismic bands at lintel level of doors and windows (*see* IS 4326)
vii)	D	Masonry construction as at C(a) but reinforced with bands and vertical reinforcement, etc (*see* IS 4326 or confined masonry using horizontal and vertical reinforcing of walls.

NOTE — In rural areas, there are huts or shacks made from bio-mass and metal sheets etc. Their vulnerability to earthquakes is very low.

Table 9 Grades of Damageability of Masonry Buildings
(*Clause* A-5)

Sl No. (1)	Classification of Damage to Masonry Buildings (2)	
i)	**Grade 1 Negligible to slight damage (no structural damage, slight non-structural damage)**	
	a) *Structural*:	Hair-line cracks in very few walls.
	b) *Non-structural*:	Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.
ii)	**Grade 2 Moderate damage (Slight structural damage, moderate non-structural damage)**	
	a) *Structural*:	Cracks in many walls, thin cracks in RC slabs and A.C. sheets.
	b) *Non-structural*:	Fall of fairly large pieces of plaster, partial collapse of smoke chimneys on roofs. Damage to parapets, chajjas. Roof tiles disturbed in about 10 percent of the area. Minor damage in under structure of sloping roofs.
iii)	**Grade 3 Substantial to heavy damage (moderate structural damage, heavy non-structural damage)**	
	a) *Structural*:	Large and extensive cracks in most walls. Wide spread cracking of columns and piers.
	b) *Non-structural*:	Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).
iv)	**Grade 4 Very heavy damage (heavy structural damage, very heavy non-structural damage)**	
	Structural:	Serious failure of walls (gaps in walls), inner walls collapse; partial structural failure of roofs and floors.
v)	**Grade 5 Destruction (very heavy structural damage)**	
		Total or near total collapse of the building.

A-6 RELATIONSHIP OF SEISMIC INTENSITY, BUILDING TYPE AND DAMAGE GRADES

Table 10 provides guidance regarding likely performance of the building in the event of design-level earthquake intensity postulated in the seismic zone. This information has been used in the survey forms to decide if there is necessity of further evaluation of the building using higher level procedures. It can also be used to identify need for retrofitting, and to recommend simple retrofitting techniques for ordinary buildings where more detailed evaluation is not feasible.

The Indicative quantities Few, Many and Most as defined in MSK intensity scales are as follows:

a) Few : 5-15 percent
b) Many : 50 percent
c) Most : 75 percent

NOTES

1 Table 10 is generally based on MSK descriptions.

2 As per european intensity scale the average values of these terms may be taken as:

a) Few—less than 15 ± 5 percent,

b) Many—Between 15 ± 5 to 55 ± 5 percent,

c) Most—Between 55 ± 5 to100 percent

A-7 RVS SURVEY FORMS — SPECIAL POINTS

The RVS survey forms (Form 1 to 4) are developed here for all the seismic Zones II to V based on the probable earthquake Intensities, building types and damageability grades as described above. Some special cases included therein are described below:

A-7.1 Importance of Building/Structure

As per IS 1893 (Part 1), an important factor I is defined for enhancing the seismic strength of buildings and structures, as follows:

> *Important Buildings* (*see* Note) — Hospitals, Schools, monumental structures; emergency buildings like telephone exchange, television, radio stations, railway stations, fire stations, large community halls like cinemas, assembly halls and subway stations, power stations, Important Industrial establishments, VIP residences and Residences of Important Emergency person.

For these important buildings the value of I is specified as 1.5, by which the design seismic force is increased by a factor of 1.5. Now the seismic zone factors for zone II to V are as follows:

Zone	II	III	IV	V
Zone factor	0.10	0.16	0.24	0.36

It is seen that one Unit change in seismic zone Intensity increases the Zone Factor 1.5 times.

Hence to deal with the damageability of important buildings in any zone, they should be checked for one unit higher zone. The assessment forms are designed accordingly.

NOTE — Any building having more than 100 occupants may be treated as Important for the purpose of RVS.

A-7.2 Special Hazards

There are some special hazardous conditions to be considered:

A-7.2.1 *Liquefiable Condition*

Normal loose sands submerged in high water table are susceptible to liquefaction under moderate to high ground accelerations; building founded on such soils will require special evaluation and treatment.

A-7.2.2 *Land Slide Prone Area*

If the building is situated on a hill slope which is prone to land slide/land slip or rock-fall under monsoon and/or earthquake, special evaluation of the site and treatment of the building will be needed.

A-7.2.3 *Irregular Buildings*

Irregularities in buildings are defined in **7.1** of IS 1893 (Part 1) under the following sub-heads:

Table 10 Damageability Grades of Masonry Buildings
(*Clause* A-5)

Sl No.	Type of Building	Zone II (MSK VI or less)	Zone III (MSK VII)	Zone IV (MSK VIII)	Zone V (MSK IX or More)
(1)	(2)	(3)	(4)	(5)	(6)
i)	A and A+	Many of grade 1 Few of grade 2 (rest no damage)	Most of grade 3 Few of grade 4 (rest of grade 2 or 1)	Most of grade 4 Few of grade 5 (rest of grade 3, 2)	Many of grade 5 (rest of grade 4 and
ii)	B and B+	Few of grade 1 (rest no damage)	Many of grade 2 Few of grade 3 (rest of grade 1)	Most of grade 3 Few of grade 4 (rest of grade 2)	Many of grade 4 Few of grade 5 (rest of grade 3)
iii)	C and C+	Few of grade 1 (rest no damage)	Many of grade 1 Few of grade 2 (rest of grade 1)	Most of grade 2 Few of grade 3 (rest of grade 1)	Many of grade 3 Few of grade 4 (rest of grade 2)
iv)	D	—	Few of grade 1	Few of grade 2	Many of grade 2 Few of grade 3 (rest of grade 1)

NOTES

1 As per MSK scale, few, many and most may be taken as: Few: (5-15) percent, Many: 50 percent and Most: 75 percent.

2 Buildings having vertical irregularity may undergo severe damage in seismic Zones III, IV and V if not specifically designed. Hence they will require special evaluation. Also buildings sited in liquefiable or landslide prone areas will require special evaluation for seismic safety.

3 Buildings having plan irregularity may undergo a damage of one grade higher in Zones III, IV and V. The surveyor may recommend re-evaluation.

a) *Plan irregularities* — These are defined in Table 4 of the Code as follows:

    1) Torsion irregularity,
    2) Re-entrant corners,
    3) Diaphragm discontinuity,
    4) Out of plane offsets, and
    5) Non-parallel systems.

    The geometric irregularities in building plans which can be easily identified are shown in Fig. 22.

    These irregularities enhance the overall damage (increased grade of damage for example at re-entrant corners). Such a building may be recommended for detailed evaluation.

b) *Vertical irregularities* — These are defined in Table 5 of IS 1893 (Part 1). The following vertical irregularities may be seen in masonry buildings (*see* Fig. 23).

    1) Mass irregularity
    2) Vertical geometric irregularity
    3) In-plane discontinuity in vertical elements resisting lateral forces.

If any of these irregularities are noticed, the building should be recommended for detailed evaluation.

A-7.2.4 *Falling Hazard*

Where such hazards are present, particularly in Zones IV and V, recommendations should make reference to these in the survey report as indicated.

A-7.2.5 *Type of Foundation Soil*

IS 1893 (Part 1) defines three soil types hard/stiff, medium and soft. No effect of these is seen in the design spectra of short period buildings, $T < 0.4$, covering all masonry buildings, hence the effect may be considered not so significant.

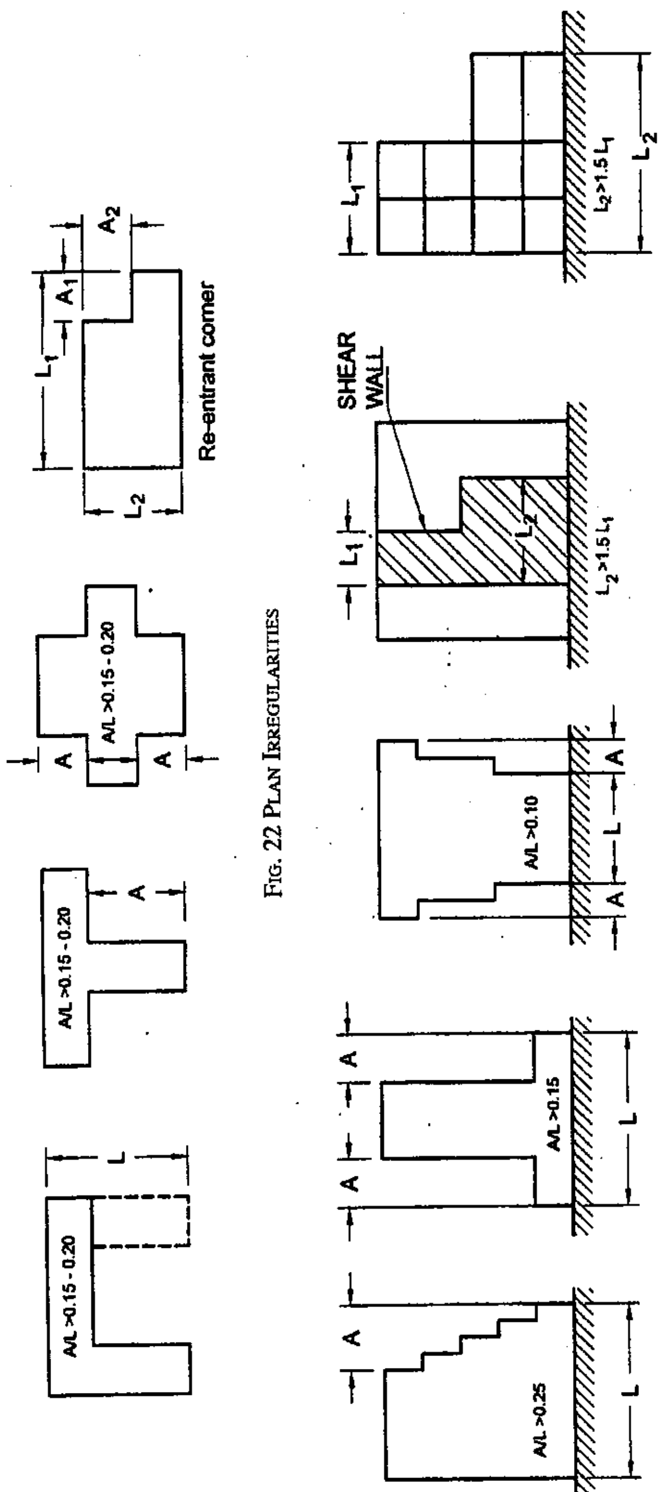


FIG. 22 PLAN IRREGULARITIES

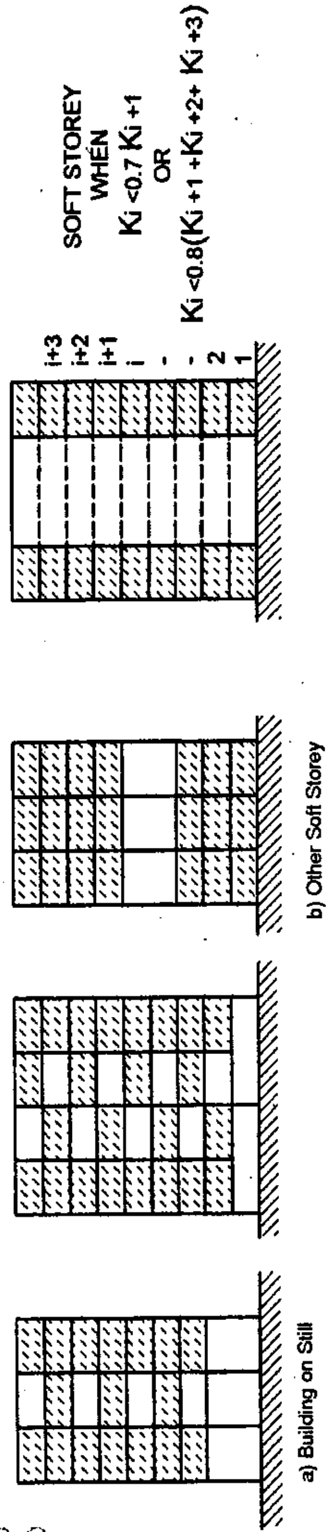


FIG. 23 VERTICAL IRREGULARITIES

FORM 1

RAPID VISUAL SCREENING OF MASONRY BUILDINGS FOR SEISMIC HAZARDS

Photograph

Sketch Plan with Length and Breadth

Seismic Zone II Ordinary Building

1.1 Building Name: ____________

1.2 Use: ____________

1.3 Address: ____________ Pin ____________

1.4 Other Identifiers: ____________

1.5 No. of Stories ____________ **1.6** Year Built ____________

1.7 Total Covered Area; all floors (sq.m) ____________

1.8 Ground Coverage (Sq.m): ____________

1.9 Soil Type: ____________ **1.10** Foundation Type ____________

1.11 Roof Type: ____________ **1.12** Floor Type ____________

1.13 Structural Components: ____________

1.13.1 Wall Type: BB* ☐ Earthen ☐ UCR* ☐ CCB* ☐

1.13.2 Thickness of wall ____________ **1.13.3** Slab thickness ___

1.13.4 Mortar Type: Mud ☐ Lime ☐ Cement ☐

1.13.5 Vert. R/F bars: Corners ☐ T-junctions ☐ Jambs ☐

1.13.6 Seismic bands: Plinth ☐ Lintel ☐ Eaves ☐ Gable ☐

*BB — Burnt Brick, *UCR — Uncoursed Random Rubble
*CCB: Cement Concrete Block

2.0 OCCUPANCY	3.0 SPECIAL HAZARD	4.0 FALLING HAZARD
2.1 ***Important buildings:*** Hospitals, Schools, monumental structures; emergency buildings like telephone exchange, television, radio stations, railway stations, fire stations, large community halls like cinemas, assembly halls and subway stations, power stations, Important Industrial establishments, VIP residences and Residences of Important Emergency person. **Any building having more than 100 Occupants may be treated as Important.*	**3.1** High Water Table (within 1m) and if sandy soil, then liquefiable site indicated. Yes No **3.2** Land Slide Prone Site Yes No **3.3** Severe Vertical Irregularity Yes No	**4.1** Chimneys **4.2** Parapets **4.3** Cladding
2.2 ***Ordinary buildings:*** Other buildings having occupants <100	**3.4** Severe Plan Irregularity Yes No	**4.4** Others

RECOMMENDED ACTION:

☐ Ensure adequate maintenance.

☐ If any Special Hazard 3.0 found, re-evaluate for possible retrofitting.

5.0 Probable Damageability in Few/Many Buildings

Building Type	5.1 Masonry Building			
Damage-ability in Zone II	A /A+	B/B+	C/C+	D
	G2/G1	G1/–	–/–	–/–

Note: +sign indicates higher strength hence somewhat lower damage expected as stated. Also average damage in one building type in the area may be lower by one grade point than the probable damageability indicated.

Surveyor will identify the Building Type; encircle it, also the corresponding damage grade.

Surveyor's sign: ____________

Name: ____________

Executive Engineer's Sign: ____________

Date of Survey: ____________

FORM 2

RAPID VISUAL SCREENING OF MASONRY BUILDINGS FOR SEISMIC HAZARDS

Seismic Zone III Ordinary Building
(Also Seismic Zone II Important Building)

Photograph

Sketch Plan with Length and Breadth

1.1	Building Name: ________
1.2	Use: ________
1.3	Address: ________ Pin ________
1.4	Other Identifiers: ________
1.5	No. of Stories ________ 1.6 Year Built ________
1.7	Total Covered Area; all floors (sq.m) ________
1.8	Ground Coverage (Sq.m): ________
1.9	Soil Type: ________ 1.10 Foundation Type ________
1.11	Roof Type: ________ 1.12 Floor Type ________
1.13	Structural Components: ________
1.13.1	Wall Type: BB* ☐ Earthen ☐ UCR* ☐ CCB* ☐
1.13.2	Thickness of wall ________ 1.13.3 Slab thickness ________
1.13.4	Mortar Type: Mud ☐ Lime ☐ Cement ☐
1.13.5	Vert. R/F bars: Corners ☐ T-junctions ☐ Jambs ☐
1.13.6	Seismic bands: Plinth ☐ Lintel ☐ Eaves ☐ Gable ☐

*BB — Burnt Brick, *UCR — Uncoursed Random Rubble
*CCB: Cement Concrete Block

2.0 OCCUPANCY	3.0 SPECIAL HAZARD	4.0 FALLING HAZARD
2.1 *Important buildings*: Hospitals, Schools, monumental structures; emergency buildings like telephone exchange, television, radio stations, railway stations, fire stations, large community halls like cinemas, assembly halls and subway stations, power stations, Important Industrial establishments, VIP residences and Residences of Important Emergency person. **Any building having more than 100 Occupants may be treated as Important.*	3.1 High Water Table (within 3 m) and if sandy soil, then liquefiable site indicated. Yes No	4.1 Chimneys
	3.2 Land Slide Prone Site Yes No	4.2 Parapets
	3.3 Severe Vertical Irregularity Yes No	4.3 Cladding
2.2 *Ordinary buildings*: Other buildings having occupants <100	3.4 Severe Plan Irregularity Yes No	4.4 Others

RECOMMENDED ACTION:

- ☐ Ensure adequate maintenance.
- ☐ Detailed evaluation of B type for need for retrofitting.
- ☐ Detailed evalution of A, A+ types for need for reconstruction or possible retrofitting
- ☐ If any Special Hazard 3.0 found, re-evaluate for possible retrofitting.
- ☐ If any of the falling hazard is present, either remove it or strengthen against falling.

Surveyor's sign: ________

Name: ________

Executive Engineer's Sign: ________

Date of Survey: ________

5.0 Probable Damageability in Few/Many Buildings

Building Type	5.1 Masonry Building			
Damage-ability in Zone III	A /A+	B/B+	C/C+	D
	G4/G3	G3/G2	G2/G1	G1

Note: +sign indicates higher strength hence somewhat lower damage expected as stated. Also average damage in one building type in the area may be lower by one grade point than the probable damageability indicated.

Surveyor will identify the Building Type; encircle it, also the corresponding damage grade.

FORM 3

RAPID VISUAL SCREENING OF MASONRY BUILDINGS FOR SEISMIC HAZARDS

Seismic Zone IV Ordinary Building
(Also Seismic Zone III Important Building)

Photograph

Sketch Plan with Length and Breadth

1.1 Building Name: ____
1.2 Use: ____
1.3 Address: ____ Pin ____
1.4 Other Identifiers: ____
1.5 No. of Stories ____ 1.6 Year Built ____
1.7 Total Covered Area; all floors (sq.m) ____
1.8 Ground Coverage (Sq.m): ____
1.9 Soil Type: ____ 1.10 Foundation Type ____
1.11 Roof Type: ____ 1.12 Floor Type ____
1.13 Structural Components: ____
1.13.1 Wall Type: BB* ☐ Earthen ☐ UCR* ☐ CCB* ☐
1.13.2 Thickness of wall ____ 1.13.3 Slab thickness ____
1.13.4 Mortar Type: Mud ☐ Lime ☐ Cement ☐
1.13.5 Vert. R/F bars: Corners ☐ T-junctions ☐ Jambs ☐
1.13.6 Seismic bands: Plinth ☐ Lintel ☐ Eaves ☐ Gable ☐

*BB — Burnt Brick, *UCR — Uncoursed Random Rubble
*CCB: Cement Concrete Block

2.0 OCCUPANCY	3.0 SPECIAL HAZARD	4.0 FALLING HAZARD
2.1 ***Important buildings:*** Hospitals, Schools, monumental structures; emergency buildings like telephone exchange, television, radio stations, railway stations, fire stations, large community halls like cinemas, assembly halls and subway stations, power stations, Important Industrial establishments, VIP residences and Residences of Important Emergency person. **Any building having more than 100 Occupants may be treated as Important.*	**3.1** High Water Table (within 5 m) and if sandy soil, then liquefiable site indicated. Yes No **3.2** Land Slide Prone Site Yes No **3.3** Severe Vertical Irregularity Yes No	**4.1** Chimneys **4.2** Parapets **4.3** Cladding
2.2 ***Ordinary buildings:*** Other buildings having occupants <100	**3.4** Severe Plan Irregularity Yes No	**4.4** Others

RECOMMENDED ACTION:

☐ A, A+ or B: Evaluate in detail for need of reconstruction or possible retrofitting to achieve type C or D

☐ B+, C: Evaluate in detail for need for retrofitting

☐ If any Special Hazard 3.0 found, re-evaluate for possible prevention/retrofitting.

☐ If any of the falling hazard is present, either remove it or strengthen against falling.

5.0 Probable Damageability in Few/Many Buildings

Building Type	5.1 Masonry Building			
Damageability in Zone III	A/A+	B/B+	C/C+	D
	G5/G4	G4/G3	G3/G2	G2

Note: +sign indicates higher strength hence somewhat lower damage expected as stated. Also average damage in one building type in the area may be lower by one grade point than the probable damageability indicated.

Surveyor will identify the Building Type; encircle it; also the corresponding damage grade.

Surveyor's sign: ____

Name: ____

Executive Engineer's Sign: ____

Date of Survey: ____

FORM 4

RAPID VISUAL SCREENING OF MASONRY BUILDINGS FOR SEISMIC HAZARDS

Seismic Zone V All Building
(Also Seismic Zone IV Important Building)

Photograph

Sketch Plan with Length and Breadth

1.1 Building Name: __________
1.2 Use: __________
1.3 Address: __________ Pin __________
1.4 Other Identifiers: __________
1.5 No. of Stories __________ **1.6** Year Built __________
1.7 Total Covered Area; all floors (sq.m) __________
1.8 Ground Coverage (Sq.m): __________
1.9 Soil Type: __________ **1.10** Foundation Type __________
1.11 Roof Type: __________ **1.12** Floor Type __________
1.13 Structural Components: __________
1.13.1 Wall Type: BB* ☐ Earthen ☐ UCR* ☐ CCB* ☐
1.13.2 Thickness of wall __________ **1.13.3** Slab thickness ____
1.13.4 Mortar Type: Mud ☐ Lime ☐ Cement ☐
1.13.5 Vert. R/F bars: Corners ☐ T-junctions ☐ Jambs ☐
1.13.6 Seismic bands: Plinth ☐ Lintel ☐ Eaves ☐ Gable ☐

*BB — Burnt Brick, *UCR — Uncoursed Random Rubble
*CCB: Cement Concrete Block

2.0 OCCUPANCY	3.0 SPECIAL HAZARD	4.0 FALLING HAZARD
2.1 ***Important buildings:*** Hospitals, Schools, monumental structures; emergency buildings like telephone exchange, television, radio stations, railway stations, fire stations, large community halls like cinemas, assembly halls and subway stations, power stations, Important Industrial establishments, VIP residences and Residences of Important Emergency person.	**3.1** High Water Table (within 5 m) and if sandy soil, then liquefiable site indicated. Yes No	**4.1** Chimneys
	3.2 Land Slide Prone Site Yes No	**4.2** Parapets
**Any building having more than 100 Occupants may be treated as Important.*	**3.3** Severe Vertical Irregularity Yes No	**4.3** Cladding
2.2 ***Ordinary buildings:*** Other buildings having occupants <100	**3.4** Severe Plan Irregularity Yes No	**4.4** Others

5.0 Probable Damageability in Few/Many Buildings

Building Type	5.1 Masonry Building			
Damage-ability in Zone V	A /A+	B/B+	C/C+	D
	G5/G4	G5/G4	G4/G3	G3

Note: +sign indicates higher strength hence somewhat lower damage expected as stated. Also average damage in one building type in the area may be lower by one grade point than the probable damageability indicated.

Surveyor will identify the Building Type, encircle it, also the corresponding damage grade.

RECOMMENDED ACTION:

- ☐ A, A⁺ or B, B⁺: evaluate in detail for need of reconstruction or possible retrofitting to achieve type C⁺ or D
- ☐ C: evaluate in detail for need for retrofitting to achieve type C⁺, D
- ☐ Wood: evaluate in detail for retrofitting
- ☐ If any Special Hazard 3.0 found, re-evaluate for possible prevention/ retrofitting.
- ☐ If any of the falling hazard is present, either remove it or strengthen against falling.

Surveyor's sign: __________

Name: __________

Executive Engineer's Sign: __________

Date of Survey: __________

ANNEX A

(Foreword)

COMMITTEE COMPOSITION

Earthquake Engineering Sectional Committee, CED 39

Organization	*Representative(s)*
National Institute of Disaster Management, New Delhi	Dr A. S. Arya (***Chairman***)
Atomic Energy Regulatory Board, Mumbai	Dr P. C. Basu Shri Roshan A. D. (*Alternate*)
Bharat Heavy Electrical Limited, New Delhi	Shri N. C. Addy Shri A. K. Singh (*Alternate*)
Bharat Heavy Electricals Limited, Hyderabad	Dr C. Kameshwara Rao (*Alternate*)
Building Materials & Technology Promotion Council, New Delhi	Shri J. K. Prasad Shri Pankaj Gupta (*Alternate*)
Central Building Research Institute, Roorkee	Shri Achal Mittal Shri Ajay Chaurasia (*Alternate*)
Central Public Works Department, New Delhi	Shri R. K. Duggal Shri A. V. Kujur (*Alternate* I) Dr A. K. Mittal (*Alternate* II)
Central Soils and Materials Research Station, New Delhi	Shri Nakul Dev Dr. K. S. Krishna Murthy (*Alternate*)
Central Water & Power Research Station, Pune	Shri I. D. Gupta Shri S. G. Chaphalkar (*Alternate*)
Central Water Commission, New Delhi	Director Embankment (NW&S) Director, BCD (N&W&NWS) (*Alternate*)
Delhi College of Engineering, Delhi	Dr (Smt) P. R. Bose Shri Alok Verma (*Alternate*)
Department of Atomic Engery, Kalpakkam	Shri S. Ramanujam Shri R. C. Jain (*Alternate*)
Directorate General of Border Roads, New Delhi	Dr V. K. Yadav
Engineer-in-Chief's Branch, New Delhi	Shri J. B. Sharma Shri. N. K. Jain (*Alternate*)
Engineers India Limited, New Delhi	Shri V. Y. Salpekar Shri Arvind Kumar (*Alternate*)
Gammon India Limited, Mumbai	Shri V. N. Haggade Shri J. N. Desai (Alternate)
Geological Survey of India, Lucknow	Shri P. Pandey Dr. Y. P. Sharda (*Alternate*)
Housing & Urban Development Corporation Ltd, New Delhi	Smt. Bindu Jeswani Shri Surinder Gera (*Alternate*)
Indian Concrete Instutute, Chennai	Dr A. R. Santhakumar
Indian Institute of Technology Bombay, Mumbai	Dr Ravi Sinha Dr A. Goyal (*Alternate*)
Indian Institute of Technology Kanpur, Kanpur	Dr S. K. Jain Dr C. V. R. Murty (*Alternate* I) Dr. Durgesh C. Rai (*Alternate* II)
Indian Institute of Technology Madras, Chennai	Dr A. Meher Prasad
Indian Institute of Technology Roorkee, Roorkee	Dr D. K. Paul Dr S. K. Thakkar (*Alternate* I) Prof Ashok Jain (*Alternate* II)
Indian Meteorological Department, New Delhi	Dr Sant Prasad Shri R. S. Dattatrayam (*Alternate*)

Organization	*Representative(s)*
Indian Road Congress, New Delhi	SECRETARY GENERAL DIRECTOR (*Alternate*)
Indian Society of Earthquake Technology, Roorkee	PROF D. K. PAUL PROF H. R. WASON (*Alternate*)
Maharashtra Engineering Research Centre, Nasik	EXECUTIVE ENGINEER ASSISTANT RESEARCH OFFICER (*Alternate*)
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In personal capacity (*403 Krishi Lok, Sector 45, Faridabad, Haryana*)	SHRI P. L. NARULA
BIS Directorate General	Shri A. K. SAINI, Scientist 'F' & Head (Civ Eng) [Representing Director General (*Ex-officio*)]

Member Secretary
SHRI S. CHATURVEDI
Scientist 'E' (Civ Eng), BIS

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Indian Standard

SEISMIC EVALUATION AND STRENGTHENING OF EXISTING REINFORCED CONCRETE BUILDINGS — GUIDELINES

ICS 91.120.25

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

February 2013

Price Group 8

Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

This standard is intended to reduce the risk of death and injury that may result from the damaging effects of earthquake on building which predate the current seismic codes [IS 1893 (Part 1) : 2002 'Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings', IS 4326 : 1993 'Code of practice for earthquake resistant design and construction of buildings' and IS 13920 : 1993 'Ductile detailing of reinforced concrete structures subjected to seismic forces — Code of practice'] or have not been designed for earthquake forces.

This standard describes a set of key steps and procedures for the assessment of the expected seismic performance of existing building in the event of a design level earthquake and where found necessary, strengthening of existing structural systems and elements for improved seismic performance.

Seismic forces for evaluation criteria of existing buildings are different from those meant for the design of new buildings. Appropriate modifications are made to address the issues of reduced serviceable life and acceptable risk for higher importance. Further, to account for uncertainty in the reliability of available information about the existing structure and the condition of structure components, strength calculations need to be suitably modified.

For deficient buildings, a broad outline for the design seismic strengthening measures has been developed and the interface with current design codes in general terms has been identified.

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

ATC 33.03 Guidelines for seismic evaluation of existing buildings, Applied Technology Council, CA.

Eurocode 8 Design provisions for earthquake resistance of structures : Part 3, CEN, Brussels, 2001.

FEMA 178 NEHRP Handbook for the seismic evaluation of existing buildings, Building Seismic Safety Council, Washington, D.C., 1992.

FEMA 154 Rapid visual screening of buildings for potential seismic hazards: A Handbook, Fedral Emergency Management Agency, Washington DC, USA, 1998.

FEMA 310 Handbook for the seismic evaluation of buildings: A Prestandard, Federal Emergency Management Agency, Washington DC, USA, 20C.

FEMA 356 Prestandard and commentary for the seismic rehabilitation of building, Federal Emergency Management Agency, Washington DC, USA, 20C.

The assessment and improvement of the structural performance of earthquake risk buildings — Draft for General Release, New Zealand National Society for Earthquake Engineering for Building Industry Authority, New Zealand, 1996T.

ASCE 31-03 Seismic evaluation of existing buildings. American Society of Civil Engineers, Reston, VA, 2003.

ASCE 41-06 Seismic rehabilitation of existing buildings. American Society of Civil Engineers, Reston, VA,2006.

Seismic assessment and retrofit of reinforced concrete buildings. International Federation of structural Concrete (Fib), Laussance, Switzerland 2003.

Uniform code for building conservation, International Conference of Building Officials, Whittier, CA, USA, 1991.

Post-earthquake damage evaluation and strength assessment of buildings, under seismic conditions, Volume 4, UNDP/UNIDO, Vienna, 1985.

International existing building code (IBC). International Code Council, Illinois, 2006.

(*Continued on third cover*)

Indian Standard

SEISMIC EVALUATION AND STRENGTHENING OF EXISTING REINFORCED CONCRETE BUILDINGS — GUIDELINES

1 SCOPE

1.1 This stand[illegible] particularly concerned with the sei[illegible] and strengthening of existing buildings and it is intended to be used as a guideline.

1.2 This standard prov[illegible] method to assess the ability of [illegible] each an adequate level of [illegible] life-safety of occupants. The [illegible] is on identification of unfavourable characteristics of the building that could result in damage [illegible] part of a building or the entire structure.

2 REFERENCES

The fo[illegible] isions, which [illegible] provision of the [illegible] publication, the editions indicated were valid. All standards are subject to revision and parties to agreements based on this stan[illegible] the possibility of [illegible] the standards [illegible]

IS No.	*Title*
456 : 2000	Code of practice for plain and reinforced conc[illegible] *fourth revision)*
1893 (Part 1) : 2002	Criteria [illegible] quake resistant [illegible]es: Part 1 General [illegible]uildings
139[illegible]	[illegible]ng of reinforced concrete structures subjected to seismic forces — Code of practice

3 T[illegible]MINOLOGY

For the [illegible] of this standard, the definitions given in IS 1893 (Part 1), IS 13920 and the following shall apply.

3.1 Acceptance Criteria — Limiting values of properties such as drift, strength demand, and inelastic deformation used to determine the acceptability of a component.

3.2 Action — An internal moment, shear, torque, axial load, developed in a member due to externally applied load/displacement on the structure.

3.3 Capacity — The permissible strength or deformation of a structural member or system.

3.4 Column (or Beam) Jacketing — A method in which a concrete column or beam is covered with a steel or reinforced concrete *jacket* in order to strengthen and/or repair the member by confining the concrete.

3.5 Components — The basic structural members that constitute a building including beams, columns, slabs, braces, walls, piers, coupling beams and connections.

3.6 Deformation — Relative displacement or rotation at the ends of a component or element or node.

3.7 Demand — The amount of force or deformation imposed on an element or component.

3.8 Displacement — The total movement, typically horizontal, of a component or element or node.

3.9 Flexible Diaphragm — A floor diaphragm shall be considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.5 times the average displacement of the entire diaphragm. Diaphragms of wood construction and of similar material or elements which are not connected together for seismic loading are considered as flexible diaphragms. Cast-*in-situ* RC floor systems are usually not flexible diaphragms.

3.10 Infill — A panel of masonry placed within a steel or concrete frame. Panels separated from the surrounding frame by a gap are termed isolated infills. A panel in tight contact with a frame around its full perimeter is termed a shear infill.

3.11 Knowledge Factor — A factor to represent the uncertainty about the reliability of the available information about the structural configuration and present condition of materials and components of the existing building.

3.12 Lateral Force Resisting System — The collection of frames, shear walls, bearing walls, braced frames and interconnecting horizontal diaphragms that provide earthquake resistance to a building.

3.13 Life Safety Performance Level — Building performance that includes significant damage to both structural and non-structural components during a design earthquake, where at least some margin against either partial or total structural collapse remains. Injuries may occur, but the level of risk for life-threatening injury and entrapment is low.

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

3.15 Load Path — The path that seismic forces acting anywhere in the building, take to the foundation of the structure and, finally, to the soil. Typically, the load travels from the diaphragm through connections to the vertical lateral-force-resisting elements, and then proceeds to the foundation.

3.16 Masonry — The assemblage of masonry units, mortar, and possibly grout and/or reinforcement. Types of masonry are classified herein with respect to the type of the masonry units, such as brick/clay-unit masonry or concrete masonry.

3.17 Non-structural Component — Architectural, mechanical or electrical components of a building that are permanently installed in, or are an integral part of a building.

3.18 Out-of-Plane Wall — A wall that resists lateral forces applied normal to its plane.

3.19 Overturning — An action resulting when the moment produced at the base of a vertical lateral-force-resisting element is larger than the resistance provided by the foundation's uplift resistance and building weight.

3.20 Plan Irregularity — Horizontal irregularity in the layout of vertical lateral-force-resisting elements, producing a mismatch between the center-of-mass and center-of-rigidity that typically results in significant torsional demands on the structure.

3.21 Pounding — Two adjacent buildings impacting during earthquake excitation because they are too close together.

3.22 Primary Element — An element that is essential to the ability of the structure to resist earthquake-induced deformations.

3.23 Probable or Measured Nominal Strength — The strength of a structure or a component to resist the effects of loads, as determined by: (a) computations using specified material strengths and dimensions, and formulas derived from accepted principles of structural mechanics; or (b) strength field tests or laboratory tests of scaled models, allowing for modelling effects and differences between laboratory and field conditions.

3.24 Redundancy — Provision of alternative load paths in a structure by which the lateral forces are resisted, allowing the structure to remain stable following the failure of any single element.

3.25 Required Member Resistance (or Required Strength) — Load effect acting on an element or connection, determined by structural analysis, resulting from the factored loads and the critical load combinations.

3.26 Rigid Diaphragm — A floor diaphragm shall be considered to be rigid, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is less than 1.5 times the average displacement of the entire diaphragm. Reinforced concrete monolithic slab-beam floors or those consisting of pre-fabricated/pre-cast elements with adequate topping reinforced screed can be taken as rigid diaphragms.

3.27 Secondary Element — An element that does not affect the ability of the structure to resist earthquake-induced deformations. They may or may not actually resist any lateral force.

3.28 Seismic Demand — Seismic hazard level and commonly expressed in the form of a ground shaking response spectrum. Structural actions (force)/ deformation in members of the building are computed due to design earthquake.

3.29 Seismic Evaluation — An approved process or methodology of evaluating deficiencies in a building which prevent the building from achieving life safety objective.

3.30 Short Column — The reduced height of column due to surrounding parapet, infill wall, etc, is less than five times the dimension of the column in (a) the direction of parapet, infill wall, etc, or (b) 50 percent of the nominal height of the typical columns at that level.

3.31 Strength — The maximum axial force, shear force, or moment that can be resisted by a component.

3.32 Strengthening Measures — Modifications to existing components, or installation of new components, that correct deficiencies identified in a seismic evaluation as part of a strengthening scheme.

3.33 Strengthening Method — A procedural methodology for the reduction of earthquake vulnerability of the building.

3.34 Strengthening Strategy — A technical approach for developing strengthening measures for a building to reduce its earthquake vulnerability.

3.35 Strong Column-Weak Beam — The capacity of the column in any moment frame joint must be greater than that of the beams, to ensure inelastic action in the beams.

3.36 Vertical Irregularity — A discontinuity of strength, stiffness, geometry, or mass in one storey with respect to adjacent stories.

4 SYMBOLS

The symbols and notations given below shall apply to the provisions of this standard:

A_c = total cross-sectional area of columns
A_g = gross area of the reinforced concrete section
A_s = steel to be provided in the jacket
A_{vf} = area of shear transfer reinforcement
A_{vf} = cross-section area of a single bar
A_w = area of shear wall
A_{wall} = total area of shear walls in the direction of loading
b_f = width of flange
d_h = diameter of stirrup
E_c = modulus of concrete
f_{ck} = characteristic strength of concrete
F_o = axial force due to overturning
f_y = yield strength of steel
H = total height
I_g = gross moment of intertia of reinforced concrete section
K = knowledge factor
L = length of the building
L_d = development length of bar in tension
M = moment
n_c = total number of columns
n_f = total number of frames in the direction of loading
P = axial load
P_{ac} = strength in axial compression
P_y = minimum yield strength in tension for the braces
t_f = thickness of flange
T_{rem} = remaining useful life of the building
T_{des} = design useful life of the building
τ_{col} = average shear stress in concrete columns
τ_{wall} = average shear stress in walls
t_j = thickness of jacket
U = useable life factor
μ = coefficient of friction
η = efficiency factor
V = total shear capacity of reinforced concrete beam
V_{con} = shear contribution of concrete
V_{FRP} = shear contribution of FRP sheet
V_B = base shear
V_s = shear force contribution of steel in a reinforced concrete beam
V_j = storey shear at level j
V_u = allowable shear force

5 EVALUATION CRITERIA

5.1 General

The seismic performance of existing buildings is evaluated in relation to the performance criteria in use for new buildings. This section defines the minimum evaluation criteria for the expected performance of life safety of existing buildings with appropriate modification to IS 1893 (Part 1) seismic force which is applicable for the seismic design of new buildings.

5.2 Since the provisions of this standard are strongly correlated with the design criteria of new buildings contained in IS 1893 (Part 1), reference shall always be made to the current edition of IS 1893 (Part 1). All existing structural elements must be able to carry full other non-seismic loads in accordance with the current applicable standards related to loading and material strengths.

5.3 Basic inputs for determination of seismic forces such as seismic zone, building type, response reduction factor are to be taken directly from IS 1893 (Part 1). Alternatively, a site-specific seismic design criteria developed along the principles described in IS 1893 (Part 1) may be used. Modification to seismic forces as given in IS 1893 (Part 1) and to material strengths will be applicable to both preliminary and detailed assessments described in this standard.

5.4 Lateral Load Modification Factor

The lateral force determined for strength related checks needs to be modified for reduced useable life. The useable life factor U, is to be multiplied to the lateral force (base shear) for new building as specified in IS 1893 (Part 1). U will be determined as

$$U = (T_{rem}/T_{des})^{0.5}$$

where

T_{rem} = remaining useful life of the building; and
T_{des} = design useful life of the building.

U will not be taken less than 0.7 in any case.

NOTES

1 By comparing the requirements of the revisions of IS 1893 of 2002 with 1984, 1975, 1966 and 1962 revisions, it is seen that buildings designed accordingly from time to time, will be found deficient to some extent.

2 It may be mentioned that buildings designed as per IS 1893 will in general not need retrofitting except those on stilts (soft first story) and those using 230 mm or thinner columns will need retrofitting.

3 Buildings designed to earlier code revisions of IS 1893 may be found deficient to a small extent. Engineer incharge may use his discretion in regard to retrofitting decision.

4 Building designed to earlier code revisions of IS 1893, unless over designed and those not designed for earthquake forces will generally need retrofitting.

5 Factor U may be applied in all cases (except in a building of critical safety, if desired U may be taken as 1.0).

5.5 Modified Material Factor

Strength capacities of existing building components shall be based on the probable material strengths in the building. Probable or measured nominal strengths are best indicator of the actual strength and may only be obtained by field or lab tests on a series of samples. It is recommended that probable strengths are either based on actual tests or the default values given in the subsequent clauses. These may also be assessed from the values given in the original building documents. However, they all need to be further modified for the uncertainty regarding the reliability of available information, and present condition of the component. The probable material strengths need to be multiplied with a Knowledge Factor, K as defined in Table 1.

Table 1 Knowledge Factor, K

Sl No. (1)	Description of Building (2)	K (3)
i)	Original construction documents available, including post-construction activities, such as modification to structure or materials testing undertaken of existing structure	1.00
ii)	Documentation as in Sl No. (i) but no testing of materials, that is using originally specified values for materials	0.90
iii)	Documentation as in Sl No. (i) no testing of, that is originally specified values for materials and minor deterioration of original condition	0.80
iv)	Incomplete but useable original construction documents and no testing	0.70
v)	Incomplete or no documents available but extensive testing and inspection done to establish current strength of load resisting members	0.70
vi)	Documentation as in Sl No. (iv) and limited inspection, and verification of structural members, or materials test results with large variation	0.60
vii)	Little knowledge of details of a component	0.50

5.6 Evaluation Process

Existing buildings not designed in accordance with the principles and philosophies and requirements of current seismic standards as described in the following clauses shall be assessed.

5.6.1 A preliminary evaluation of building is carried out. This involves broad assessment of its physical condition, robustness, structural integrity and strength of structure, including simple calculations.

5.6.2 If the results of preliminary evaluation for strength, overall stability and integrity are acceptable, no further action is required. Else a detailed evaluation is required unless exempted.

NOTE — Single or two storey buildings (not housing essential services required for post-earthquake emergency response) of total floor areas less than 300 sq. m may be exempted from detailed evaluation even when a preliminary evaluation indicates deficiencies and where seismic retrofitting is carried out to remedy those deficiencies.

5.6.3 A detailed evaluation includes numerical checks on stability and integrity of the whole structure as well as the strength of each member. Conventional design calculations for these checks shall use modified demands and strengths. A flow diagram summarizing various steps of the evaluation process is shown in Fig. 1.

6 PRELIMINARY EVALUATION

6.1 General

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that may affect its seismic vulnerability. It is a very approximate procedure based on conservative parameters to identify the potential earthquake risk of a building and may be used to screen buildings for detailed evaluation. Method is primarily based on observed damage characteristics in previous earthquakes coupled with some simple calculations.

6.2 Site Visit

A site visit shall be conducted by the design professional to verify available existing building data or collect additional data, and to determine the condition of the building and its components. The following information either needs to be confirmed or collected during the visit:

a) *General information* — Number of storeys and dimensions, year of construction.
b) *Structural system description* — Framing vertical lateral force-resisting system, floor and roof diaphragm connection to walls, basement and foundation system.
c) Building type and site soil classification as in IS 1893 (Part 1).
d) Building use and nature of occupancy.
e) Adjacent buildings and potential for pounding and falling hazards.
f) *General conditions* — Deterioration of

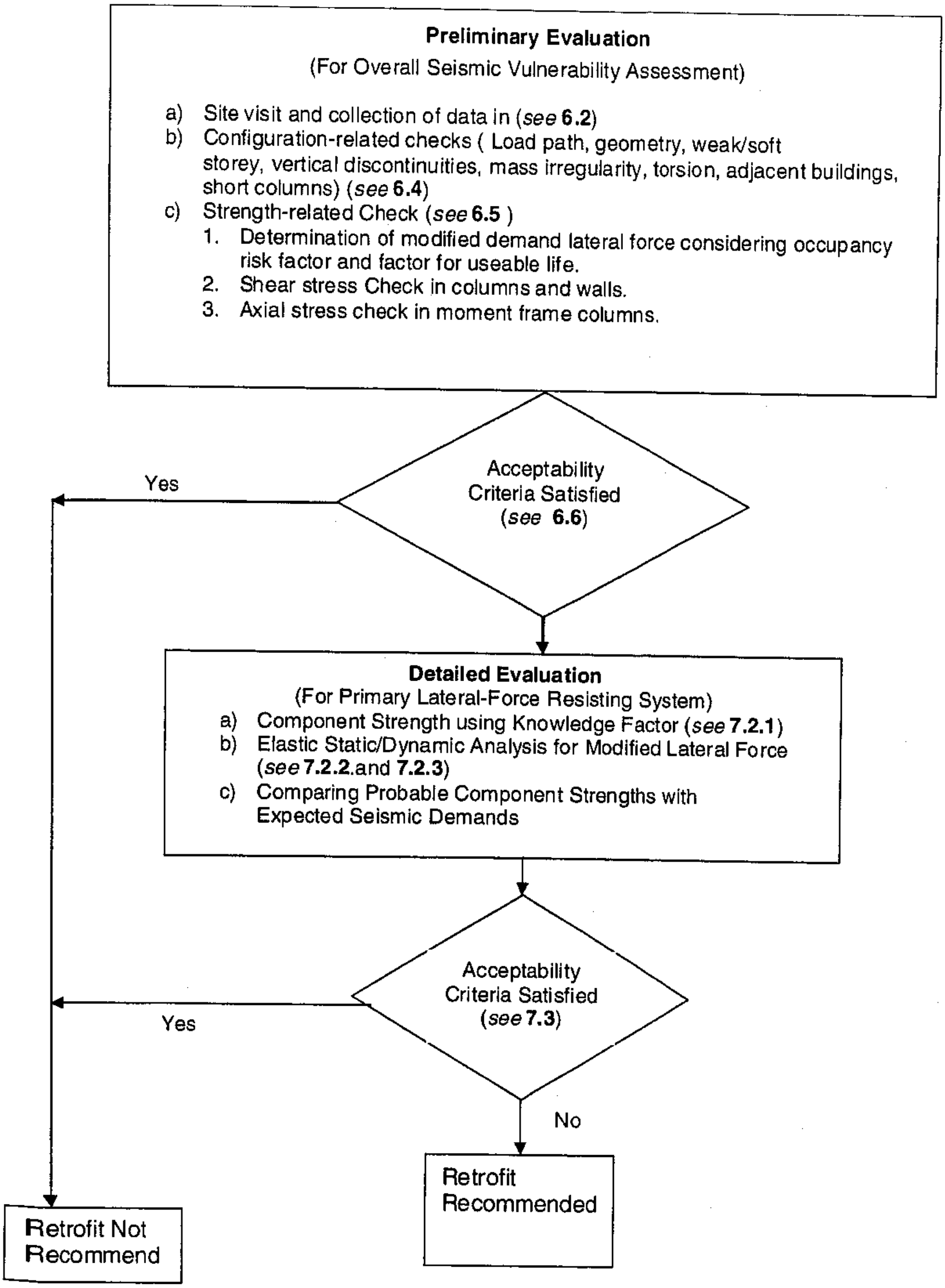


FIG. 1 FLOW CHART SUMMARIZING EVALUATION PROCESS

materials, damage from past earthquakes, alterations and additions that could affect earthquake performance.

g) Architectural features that may affect earthquake performance, especially location of masonry infill walls.

h) Geological site hazards and foundation conditions: Susceptibility for liquefaction and conditions for slope failure and surface fault rupture.

j) Special construction anomalies and conditions.

6.3 Acceptability Criteria

A building is said to be acceptable, if it meets all the configuration-related checks as well as global level checks on axial and shear stress as outlined in the following clauses.

6.4 Configuration-Related Checks

6.4.1 *Load Path*

The structure shall contain at-least one rational and complete load path for seismic forces from any horizontal direction so that they may transfer all inertial forces in the building to the foundation.

6.4.2 *Redundancy*

The number of lines of vertical lateral load resisting elements in each principal direction shall be greater than or equal to 2. In the case of moment frames, the number of bays in each line shall be greater than or equal to 2. Similarly, the number of lines of shear walls in each direction shall be greater than or equal to 2.

6.4.3 *Geometry*

No change shall be made in the horizontal dimension of lateral force resisting system of more than 50 percent in a storey relative to adjacent stories, excluding penthouses and mezzanine floors.

6.4.4 *Weak Storey*

The strength of the vertical lateral force resisting system in any storey shall not be less than 70 percent of the strength in an adjacent storey.

6.4.5 *Soft Storey*

The stiffness of vertical lateral load resisting system in any storey shall not be less than 60 percent of the stiffness in an adjacent storey or less than 70 percent of the average stiffness of the three storeys above.

6.4.6 *Vertical Discontinuities*

All vertical elements in the lateral force resisting system shall be continuous from the root to the foundation.

6.4.7 *Mass*

There shall be no change in effective mass more than 100 percent from one storey to the next. Light roofs, penthouses, and mezzanine floors need not be considered, in mass irregularity.

6.4.8 *Torsion*

The estimated distance between a storey center of mass and the storey centre of stiffness shall be less than 30 percent of the building dimension at right angles to the direction of loading considered.

6.4.9 *Adjacent Buildings*

The clear horizontal distance between the building under consideration and any adjacent building shall be greater than 4 percent of the height of the shorter building, except for buildings that are of the same height with floors located at the same levels. The gap width specified in **7.11.3** of IS 1893 (Part 1).

6.4.10 *Short Columns*

The reduced height of a column due to surrounding parapet, infill wall, etc, shall not be less than five times the dimension of the column in the direction of parapet, infill wall, etc, or 50 percent of the nominal height of the typical columns in that storey.

6.4.11 *Mezzanines/Loft/Sub-floors*

Interior mezzanine/loft/sub-floor levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure.

6.5 Strength-Related Checks

Approximate and quick checks shall be used to compute the strength and stiffness of building components. The seismic base shear and storey shears for the building shall be computed in accordance with IS 1893 (Part 1) and the requirements of **5**.

6.5.1 *Shear Stress in Reinforced Concrete Frame Columns*

The average shear stress in concrete columns, τ_{col}, computed in accordance with the following equation shall be lesser of,

a) 0.4 MPa; and

b) $0.10\sqrt{f_{ck}}$, f_{ck} is characteristic cube strength of concrete:

$$\tau_{col} = \left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_i}{A_c}\right)$$

where

n_c = total number of columns;

n_f = total number of frames in the direction of loading;

V_j = storey shear at level j; and

A_c = total cross-sectional area of columns.

6.5.2 *Shear Stress in Shear Walls*

Average shear stress in concrete and masonry shear walls, τ_{wall}, shall be calculated as per the following equation:

$$\tau_{wall} = \left(\frac{V_j}{A_{wall}}\right)$$

where

V_j = storey shear at level j; and

A_{wall} = total area of shear walls in the direction of the loading.

NOTES

1 For concrete shear walls, τ_{wall} shall be less than 0.40 MPa.

2 For unreinforced masonry load bearing wall buildings, the average shear stress, τ_{wall} shall be less than 0.10 MPa.

6.5.3 *Shear Stress Check for Reinforced Concrete Masonry Infill Walls*

The shear stress in the reinforced masonry shear walls shall be less than 0.30 MPa and the shear stress in the unreinforced masonry shear walls shall be less than 0.10 MPa.

6.5.4 *Axial Stress in Moment Frames*

The maximum compressive axial stress in the columns of moment frames at base due to overturning forces alone (F_0) as calculated using the following equation shall be less than $0.25f_{ck}$.

$$F_0 = \frac{2}{3}\left(\frac{V_B}{n_f}\right)\left(\frac{H}{L}\right)$$

where

n_f = total number of frames in the direction of loading,

V_B = base shear,

H = total height, and

L = length of the building.

6.6 Recommendation for Detailed Evaluation

A building is recommended to undergo a detailed evaluation as described in 6, if any of the following conditions are met:

a) Building fails to comply with the requirements of the preliminary evaluation;
b) A building is 6 storeys and higher;
c) Buildings located on incompetent or liquefiable soils and/or located near (less than 15 km) active faults and/or with inadequate foundation details; and
d) Buildings with inadequate connections between primary structural members, such as poorly designed and/or constructed joints of pre-cast elements.

7 DETAILED EVALUATION

7.1 General

The detailed evaluation procedure is based on determining the probable strength of lateral load resisting elements and comparing them with the expected seismic demands. The probable strengths determined from conventional methods and applicable codes shall be modified with appropriate knowledge factor K given in 5. An assessment of the building for its present condition of its components and strength of materials is required. Further, seismic demand on critical individual components shall be determined using seismic analysis methods described in IS 1893 (Part 1) for lateral forces prescribed therein with modification for (reduced) useable life factor, described in 5.

7.1.1 *Condition of the Building Components*

The building shall be checked for the existence of some of the following common indicators of deficiency:

a) *Deterioration of concrete* — There shall be no visible deterioration of the concrete or reinforcing steel in any of the vertical or lateral force resisting elements.
b) *Cracks in boundary columns* — There shall be no existing diagonal cracks wider than 3 mm in concrete columns that encase masonry infills.
c) *Masonry units* — There shall be no visible deterioration of masonry units.
d) *Masonry joints* — The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.
e) *Cracks in infill walls* — There shall be no existing diagonal cracks in infill walls that extend throughout a panel, are greater than 3 mm, or have out-of-plane offsets in the bed joint greater than 3 mm.

7.1.2 *Condition of the Building Materials*

An evaluation of the present day strength of materials shall be performed using on-site non-destructive testing and laboratory analysis of samples taken from the building. Field tests are usually indicative tests and therefore shall be supplemented with proper laboratory facilities for accurate quantitative results.

7.2 Evaluation Procedure

The key steps of this evaluation procedure are as follows:

7.2.1 *Probable Flexure and Shear Demand and Capacity*

Estimate the probable flexural and shear strengths of the critical sections of the members and joints of vertical lateral force resisting elements. These calculations shall be performed as per respective codes for various building types and modified with knowledge factor K.

7.2.2 *Design Base Shear*

Calculate the total lateral force (design base shear) in accordance with [IS 1893 (Part 1)] and multiply it with U, a factor for the reduced useable life (equal to 0.70).

7.2.3 *Analysis Procedure*

Perform a linear equivalent static or a dynamic analysis of the lateral load resisting system of the building in accordance with IS 1893 (Part 1) for the modified base shear determined in the previous step and determine resulting member actions for critical components.

a) *Mathematical model* — Mathematical model of the physical structure shall be such as to represent the spatial distribution of mass and stiffness of the structure to an extent that is adequate for the calculation of significant features of its distribution of lateral forces. All concrete as well as masonry elements shall be included in the model.

b) *Component stiffness* — Component stiffness shall be determined based on some rational procedure. Some standard values are given in Table 2.

7.2.4 *Demand-Capacity Ratio*

Evaluate the acceptability of each component by comparing its probable strength with the member actions.

7.2.5 *Inter-storey Drift*

Calculate whether the inter-storey drifts and decide whether it is acceptable in terms of the requirements of IS 1893 (Part 1).

7.3 Acceptability Criteria

A building is said to be acceptable if either of the following two conditions are satisfied along with supplemental criteria for a particular building type described in **7.4**:

a) All critical elements of lateral force resisting elements have strengths greater than computed actions and drift checks are satisfied.

b) Except a few elements, all critical elements of the lateral force resisting elements have strengths greater than computed actions and drift checks are satisfied. The engineer has to ensure that the failure of these few elements shall not lead to loss of stability or initiate progressive collapse. This needs to be verified by a non-linear analysis such as pushover analysis, carried out upto the collapse load.

7.4 Ductility and Detailing Related Evaluation

In addition to the general evaluation (*see* **7.2**) for buildings which addresses only strength issues more criteria need to be considered which relate to ductility and detailing of structural components. These criteria address certain special features affecting the lateral load-behaviour which are specific to each building type.

7.4.1 *Moment Resisting Reinforced Concrete Frame Buildings*

For RC moment frame buildings designed using response reduction factor R [*see* IS 1893 (Part 1)] equal to 5 the following supplemental criteria need to be satisfied. Any deficiency should be considered in suitably reducing the value of R.

Table 2 Some Effective Stiffness Values

(*Clause* 7.2.3)

Sl No. (1)	Component (2)	Flexural Rigidity (3)	Shear Rigidity (4)	Axial Rigidity (5)
i)	Beam, non pre-stressed	$0.5\ E_c I_g$	—	—
ii)	Beam, pre-stressed	$1.0\ E_c I_g$	—	$E_c A_g$
iii)	Column in compression($P > 0.5 f_c' A_g$)	$0.7\ E_c I_g$	$0.4\ E_c A_w$	$E_c A_g$
iv)	Column in compression ($P \geq 0.5 f_c' A_g$)	$0.5\ E_c I_g$	—	$E_c A_g$
v)	Walls — Uncracked	$0.8\ E_c I_g$	—	$E_c A_g$
vi)	Walls — Cracked	$0.5\ E_c I_g$	—	$E_c A_g$
vii)	Flat slab	To be determined based on rational procedure		$E_c A_g$

a) *No shear failures* — Shear capacity of frame members shall be adequate to develop the moment capacity at the ends, and shall be in accordance with provisions of IS 13920 for shear design of beams and columns.

b) *Concrete columns* — All concrete columns shall be adequately anchored into the foundation from top face of pedestal of base slab.

c) *Strong column/weak beam* — The sum of the moment of resistance of the columns shall be at least 1.1 times the sum of the moment of resistance of the beams at each frame joint.

d) *Beam bars* — At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25 percent of the longitudinal bars located at the joints for either positive or negative moment shall be continuous throughout the length of the members.

e) *Column-bar splices* — Lap splices shall be located only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be located over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall preferably be spliced at one section. If more than 50 percent of the bars are spliced at one section, the lap length shall be 1.3 L_d where L_d is the development length of bar in tension as per IS 456.

f) *Beam-bar splices* — Longitudinal bars shall be spliced only if hoops are located over the entire splice length, at a spacing not exceeding 150 mm. The lap length shall not be less than the bar development length in tension. Lap splices shall not be located (1) within a joint; (2) within a distance of $2d$ from joint face; and (3) within a quarter length of the member near supports where flexural yielding may occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.

g) *Column-tie spacing* — The parallel legs of rectangular hoop shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, the provision of a cross tie should be there. Alternatively, a pair of overlapping hoops may be located within the column. The hooks shall engage peripheral longitudinal bars.

h) *Stirrup spacing*—The spacing of stirrups over a length of $2d$ at either end of a beam shall not exceed (1) $d/4$, or (2) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50 mm from the joint face. In case of beams vertical hoops at the same spacing as above shall also be located over a length equal to 2d on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding $d/2$.

j) *Joint reinforcing*— Beam-column joints shall have ties spaced at or less than 150 mm.

k) *Stirrup and tie hooks* — The beam stirrups and column ties shall preferably be anchored into the member cores with hooks of 135°.

7.4.2 *Concrete Shear Wall Buildings*

Concrete shear wall buildings can be either the ordinary reinforced type or ductile shear wall type. Some of the provisions mentioned below are applicable to both types of shear walls while some are applicable only for ductile shear walls. Applicable provisions shall indicate the suitable choice for the response reduction factor R.

7.4.2.1 *Thickness*

The thickness of any part of an ordinary shear wall shall preferably, not be less than 100 mm while for ductile shear wall it shall not be less than 150 mm. In case of coupled shear walls, the thickness of the walls shall be at least 200 mm.

7.4.2.2 *Overturning*

All shear walls shall have aspect ratio less than 4 to 1, else the foundation system shall be investigated for its adequacy to resist overturning moments. Wall piers need not be considered.

7.4.2.3 *Reinforcement*

a) Shear walls shall be provided with reinforcement in the longitudinal and transverse directions in the plane of the wall to resist bending moment and to prevent premature shear failure. The minimum reinforcement ratio for ordinary shear walls shall be 0.001 5 of the gross area in each direction. For ductile shear walls this value is increased to 0.002 5 in the horizontal direction. This reinforcement shall be distributed uniformly across the cross-section of the wall.

b) The stirrups in all coupling beams over openings for doors, passages, staircases, etc, shall be spaced at or less than $d/2$ and shall be anchored into the core with hooks of 135° or more. The shear and flexural demand on coupling beams which are non-compliant are calculated using analysis procedure of **7.2** and their adequacy is checked. If they are found inadequate then their adequacy is checked as if they were independent.

7.4.2.4 *Opening in walls*

Total length of openings shall not be greater than 75 percent of the length of any perimeter wall.

The adequacy of remaining wall for shear and overturning resistances shall be evaluated according to **7.2**. Shear transfer connection between the diaphragm and walls shall also be evaluated and checked for adequacy.

7.4.3 *Reinforced Concrete Frames with Masonry Infill Walls*

The provisions of **7.4.1** also apply to reinforced concrete frames with masonry infill walls. In addition, the infill walls shall be checked for the following additional criteria:

a) *Wall connections* — All infill walls shall have a positive connection to the frame to resist out-of-plane forces.
b) *Out of plane stability* — The unreinforced masonry wall height-to-thickness ratios shall be less than as given in Table 3. The frame element beams are assumed to provide necessary lateral support for the unreinforced masonry wall in out-of-plane direction.

Table 3 Allowable Height-to-Thickness Ratios of Unreinforced Masonry Walls

Sl No.	Wall Type	Zone II and III	Zone IV	Zone V
(1)	(2)	(3)	(4)	(5)
i)	Top storey of multi-storey building	14	14	9
ii)	First storey of multi-storey building	18	16	15
iii)	All other conditions	16	16	13

c) *Unreinforced masonry parapets* — The maximum height of an unsupported unreinforced masonry parapet shall not exceed the height-to-thickness ratio as shown in Table 4. If the required parapet height exceeds this maximum height, a bracing system designed for the forces determined as per non-structural elements specified in **8.5.2.2**, shall support the top of the parapet. The minimum height of a parapet above any wall anchor shall be 300 mm. If a reinforced concrete beam is provided at the top of the wall, the minimum height above the wall anchor may be 150 mm.

Table 4 Maximum Allowable *h/t* Ratio for Parapets

Unreinforced Masonry Parapets	Zone V	All Other Zones
Maximum allowable height-to-thickness ratio	1.5	2.5

8 SEISMIC STRENGTHENING

8.1 General

This clause outlines seismic strengthening options and strategies at a general level, and describes a methodology for the design of the strengthening measures as modifications to correct reduce seismic deficiency identifying during the evaluation procedure given in **7**.

8.2 Seismic Strengthening Options and Strategies

Seismic strengthening for improved performance in the future earthquakes shall be achieved by one of several options given in this clause. The chosen seismic strengthening scheme shall increase the redundancy of lateral load resisting elements to avoid collapse and overall instability.

8.2.1 *Strengthening at Member Level*

a) Existing buildings with a sufficient level of strength and stiffness at the global level may have some members (or components), which lack adequate strength, stiffness or ductility. If such deficient members are small in number, an economical and appropriate strategy is to modify these deficient members alone while retaining the existing lateral-force resisting system.
b) Member level modification shall be undertaken to improve strength, stiffness and/or ductility of deficient members and their connections strengthening measures shall include such as jacketing columns or beams.
c) Member level strengthening measures that enhance ductility of the member without significantly increasing its strength/stiffness are often useful when analysis indicates that a few members of the lateral-load resisting system are deficient. One such measure is jacketing of reinforced concrete columns, which improves the member level ductility by increased confinement.

8.2.2 *Eliminating or Reducing Structural Irregularities*

a) Irregularities related to distribution of strength, stiffness and mass result in poor seismic performance. Often these irregularities exist because of discontinuity of structural members. Simple removal of such discontinuities may reduce seismic demand on other structural components to acceptable levels.

b) An effective measure to correct vertical irregularities such as weak and/or soft storey is the addition of shear walls and braced frames within the weak/soft storey. Braced frames and shear walls may also be effectively used to balance stiffness and mass distribution within a storey to reduce torsional irregularities. Shear wall shall be placed such that it forms an integral part of load flow path for lateral loads. Minimum two shear wall shall be constructed in each orthogonal direction in opposite side of shear centre away from centre as far as possible to add better torsional resistance to the entire structure. The stiffness centre of the complete structure at a floor level after adding shear wall shall be such that eccentricity with respect to centre of gravity of mass is reduced to a minimum.

c) Seismic gaps (or movement joints) shall be created between various parts of a building with irregular plan geometry to separate it into a number of regular independent structures. However, care shall be exercised to provide sufficiently wide gaps to avoid the problem of pounding.

8.2.3 *Strengthening at Structural Level*

In structures where more than a few critical members and components do not have adequate strength and ductility, an effective way is to strengthen the structure so that the overall displacement demands shall be reduced. It may enhance force demands on some other elements, which may require further strengthening. Braced frames and shear walls are an effective means of adding stiffness and strength.

8.3 Alternative Strengthening Options

8.3.1 *Supplemental Damping and Isolation*

Seismic isolation and supplemental damping are rapidly evolving strategies for improving the seismic performance of structures. Base isolation reduces the demands on the elements of the structure. This technique is most effective for relatively stiff buildings with low profiles and large mass compared to light, flexible structures.

Energy dissipation helps in the overall reduction in displacements of the structure.

This technique is most effective in structures that are relatively flexible and have some inelastic deformation capacity.

8.4 Methods of Analysis and Design for Strengthening

8.4.1 *Design Criteria*

The performance criteria for the design of strengthening measures shall be same as for evaluation process as defined in **5**.

8.4.2 *Member Capacities*

Member capacities of existing elements shall be based on the probable strengths as defined in **5** and also used for detailed evaluation.

8.4.3 *Analysis Options*

The engineer may choose to perform the same analysis as performed during the evaluation process.

8.5 Strengthening Options for Reinforced Concrete Framed Structures

8.5.1 *Jacketing*

The deficient frame members and joints are identified during detailed evaluation of building. Members requiring strengthening or enhanced ductility shall be jacketed by reinforced concrete jacketing, steel profile jacketing, and steel encasement or wrapping with FRPs where possible, the deficient members shall first be stress relieved by propping.

NOTES

1 Reinforced concrete jacketing involves placement of new longitudinal reinforcement and transverse reinforcement bars in the new concrete overlay around existing member.

2 Steel profile jacketing shall be done through steel angle profiles placed at each corner of the existing reinforced concrete member and connected together as a skeleton with transverse steel straps. Another way is by providing steel encasement. Steel encasement is the complete covering of the existing member with thin plates.

3 Retrofitting using FRPs involves placement of composite material made of continuous fibres with resin impregnation on the outer surface of the reinforced concrete member.

8.5.1.1 *Reinforced concrete jacketing of columns*

Reinforced concrete jacketing improves column flexural strength and ductility. Closely spaced transverse reinforcement provided in the jacket improves the shear strength and ductility of the column. The procedure for reinforced concrete jacketing is as follows:

a) The seismic demand on the columns, in terms of axial load P and moment M is obtained.

b) The column size and section details are estimated for P and M as determined above.

c) The existing column size and amount of reinforcement is deducted to obtain the amount of concrete and steel to be provided in the jacket.

d) The extra size of column cross-section and reinforcement is provided in the jacket.

e) Increase the amount of concrete and steel actually to be provided as follows to account for losses. $A_c = (3/2)A_c'$ and $A_s = (4/3)A_s'$

where

A_c and A_s = actual concrete and steel to be provided in the jacket; and

A_c' and A_s' = concrete and steel values obtained for the jacket after deducting the existing concrete and steel from their respective required amount.

f) The spacing of ties to be provided in the jacket in order to avoid flexural shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = \frac{f_y}{\sqrt{f_{ck}}} \frac{d_h^2}{t_j}$$

where

f_y = yield strength of steel,

f_{ck} = cube strength of concrete,

d_h = diameter of stirrup, and

t_j = thickness of jacket.

g) If the transfer of axial load to new longitudinal steel is not critical then friction present at the interface shall be relied on for the shear transfer, which shall be enhanced by roughening the old surface.

h) Dowels which are epoxy grouted and bent into 90° hook shall also be employed to improve the anchorage of new concrete jacket.

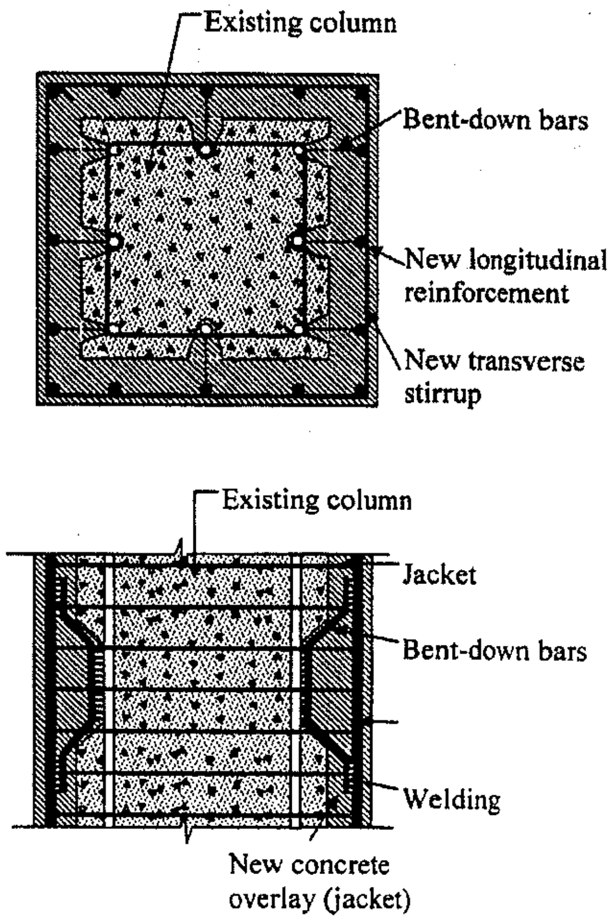


FIG. 2 REINFORCED CONCRETE JACKETING

8.5.1.2 The minimum specifications for jacketing columns are:

a) Strength of the new materials shall be equal or greater than those of the existing column. Concrete strength shall be at least 5 MPa greater than the strength of the existing concrete.
b) For columns where extra longitudinal reinforcement is not required, a minimum of 12ϕ bars in the four corners and ties of 8ϕ @ 100 c/c should be provided with 135° bends and 10ϕ leg lengths.
c) Minimum jacket thickness shall be 100 mm.
d) Lateral support to all the longitudinal bars shall be provided by ties with an included angle of not more than 135°.
e) Minimum diameter of ties shall be 8 mm and not less than one-third of the longitudinal bar diameter.
f) Vertical spacing of ties shall not exceed 200 mm, whereas the spacing close to the joints within a length of ¼ of the clear height shall not exceed 100 mm. Preferably, the spacing of ties shall not exceed the thickness of the jacket or 200 mm whichever is less.

8.5.1.3 *Fibre jacketing of a beam*

Dimensions of FRP jacket is determined assuming composite action between fiber and existing concrete. The rupture strength of FRP is used as its limiting strength.

Limit state moment capacity of FRP retrofitted member is given by:

Ultimate flexure strength is determined based on the assumption that compressive concrete reaches a strain of 0.003 5 and FRP reaches its maximum strain.

Shear strength of a beam after strengthening:

$$V = V_{con} + V_s + V_{FRP}$$

where

$V_{con} = T_c \times b \times D$

$V_s = 0.87 \times f_y \times A_{sv} \times (d/s_v)$

$V_{FRP} = A_f f_f \left(\frac{d}{s}\right)$

V_{con} = shear contribution of concrete;

V_s = shear contribution of steel; and

V_{FRP} = shear contribution of FRP sheet.

8.5.2 *Addition of New Structural Elements*

One of the strengthening methods includes adding new structural elements to an existing structure to increase its lateral force capacity. Shear walls and steel bracing shall be added as new elements to increase the strength and stiffness of the structure.

8.5.2.1 *Addition of reinforced concrete shear wall*

Addition of new reinforced concrete shear walls provides the best option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures. The design of shear walls shall be done as per IS 13920.

a) Where vertical shear walls are inserted between existing columns shear transfer reinforcement (dowel bars), perpendicular to the shear plane, is given by:

$$A_{vf} = \frac{V_u}{f_y \mu}\eta$$

where

V_u = allowable shear force not greater than $0.2 f_{ck} A_c$ or $5.5 A_c$ (A_c is the area of concrete section resisting shear transfer);

μ = coefficient of friction;

= 1.0 for concrete placed against hardened concrete with surface intentionally roughened;

= 0.75 for concrete anchored to as-rolled structural steel by headed studs or by reinforcing bars; and

η = efficiency factor = 0.5

b) The number of bars required for resisting shear at the interface are given by:

$$n = \frac{A_{vf}}{A'_{vf}}$$

where

A'_{vf} = cross-section area of a single bar.

c) The minimum anchorage length of the grouted-in longitudinal and transverse reinforcement of the shear wall in to the existing components of the building shall not be less than 6 times the diameter of the bars (*see* Fig. 3).
d) Wherever thickness of column is 250 mm or less, shear wall shall encase the column by wrapping shear wall reinforcement around column after roughening reinforced concrete column surface. In case where shear wall spans perpendicular to the larger dimension of column, the transverse reinforcement of shear wall shall be anchored and wrapped around the column surface as shown in the sketch.

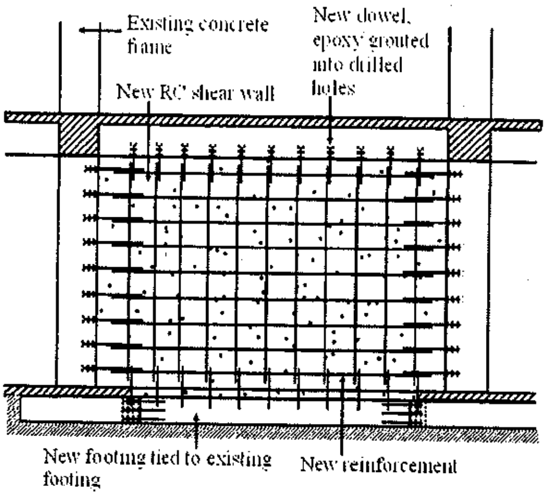


FIG. 3 ADDING NEW SHEAR WALLS

8.5.2.2 *Addition of steel bracing*

Steel diagonal braces shall be added to existing concrete frames. Braces shall be arranged so that their centre line passes through the centres of the beam-column joints. Angle or channel steel profiles shall be used. Some of the design criteria for braces are given below:

a) Slenderness of bracing member shall be less or equal to $2\,500/\sqrt{f_y}$.

b) The width-thickness ratio of angle sections for braces shall not exceed $136/\sqrt{f_y}$. For circular sections the outside diameter to wall thickness ratio shall not exceed $8\,960/f_y$, and rectangular tubes shall have an out-to-out width to wall thickness ratio not exceeding $288/\sqrt{f_y}$.

c) In case of Chevron (inverted-V) braces, the beam intersected by braces shall have adequate strength to resist effects of the maximum unbalanced vertical load applied to the beam by braces. This load shall be calculated using a minimum of yield strength P_y for the brace in tension and a maximum of 0.3 times of load capacity for the brace in compression P_{ac}.

d) The top and bottom flanges of the beam at the point of intersection of V-braces shall be designed to support a lateral force equal to 2 percent of the beam flange strength $f_y b_f t_f$.

e) The brace connection shall be adequate against out-of-plane failure and brittle fracture. Typical connection detail is shown in Fig. 4.

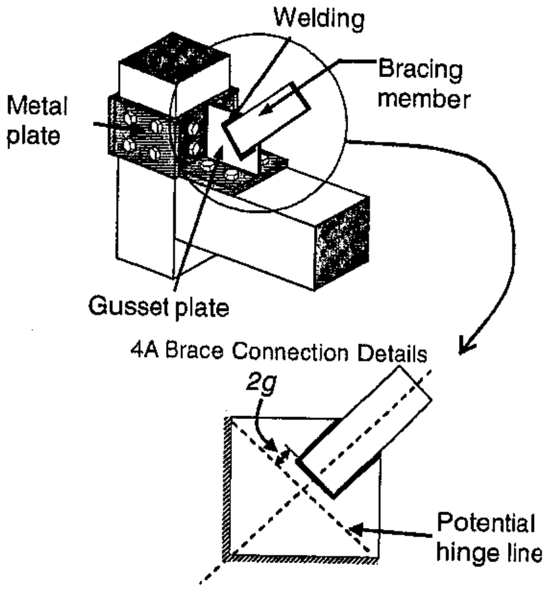
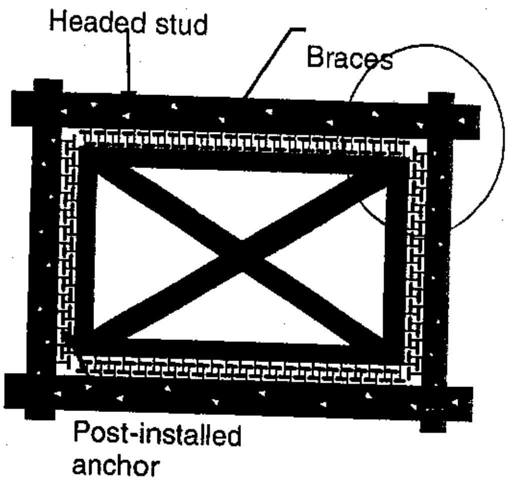
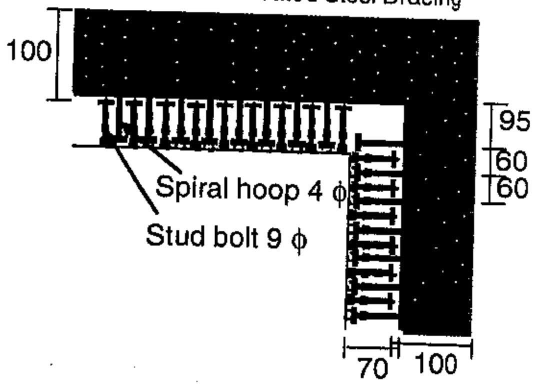


FIG. 4 BRACE CONNECTIONS

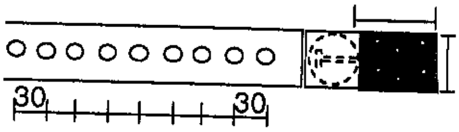
8.5.2.3 Pre-fabricated steel bracing sub assemblages as shown in Fig. 5 may be used, for ease of construction, Braces in X-, V-and inverted V-shall be arranged inside a heavy rectangular steel frame, which is then placed in frame bay and firmly connected.



5A Pre-fabricated Steel Bracing



5B Detailing of Corner View of Fig. 5A



5C Detailing of Corner View of Fig. 5A

FIG. 5 DETAILING OF PRE-FABRICATED STEEL BRACING

ANNEX A
(Foreword)
COMMITTEE COMPOSITION

Earthquake Engineering Sectional Committee, CED 39

Organization	*Representative(s)*
Building Materials & Technology Promotion Council, New Delhi	Dr A. S. Arya (***Chairman***)
Association of Consulting Engineers, Bangalore	Shri Umesh B. Rao Shri B. V. Ravindra Nath (*Alternate*)
Atomic Energy Regulatory Board, Mumbai	Dr P. C. Basu Shri Roshan A. D. (*Alternate*)
Bharat Heavy Electrical Limited, New Delhi	Shri Ravi Kumar Dr C. Kameshwara Rao (*Alternate*)
Building Materials & Technology Promotion Council, New Delhi	Shri J. K. Prasad Shri Pankaj Gupta (*Alternate*)
Central Building Research Institute, Roorkee	Shri Navjeen Saxena Shri Ajay Chaurasia (*Alternate*)
Central Public Works Department, New Delhi	Shri Bhagwan Singh Shri S. P. Lokhande (*Alternate*)
Central Soils and Materials Research Station, New Delhi	Shri N. P. Honkandavar Shri S. L. Gupta (*Alternate*)
Central Water & Power Research Station, Pune	Shri I. D. Gupta Shri S. G. Chaphalkar (*Alternate*)
Central Water Commission, New Delhi	Director, CMDD (E & NE) Director, Embankment (*Alternate*)
DDF Consultants Pvt Ltd, New Delhi	Dr (Shrimati) Pratima R. Bose
Delhi College of Engineering, Delhi	Shri Alok Verma
Department of Atomic Energy, Kalpakkam	Shri S. Ramanujam Shri R. C. Jain (*Alternate*)
Directorate General of Border Roads, New Delhi	Shri A. K. Dixit
Engineer-in-Chief's Branch, New Delhi	Brig B. D. Pandey Shri Ravi Sinha (*Alternate*)
Engineers India Limited, New Delhi	Shri Vinay Kumar Shrimati Ila Dass (*Alternate*)
Gammon India Limited, Mumbai	Shri V. N. Haggade Shri J. N. Desai (*Alternate*)
Geological Survey of India, Lucknow	Shri Harsh Gupta Dr Kiran Mazumdar (*Alternate*)
Housing & Urban Development Corporation Ltd, New Delhi	Shrimati Bindu Jeswani Shri Surinder Gera (*Alternate*)
Indian Concrete Institute, Chennai.	Dr A. R. Santhakumar
Indian Institute of Technology Bombay, Mumbai	Dr Ravi Sinha Dr Alok Goyal (*Alternate*)
Indian Institute of Technology Hyderabad, Hyderabad	Dr C. V. R. Murty
Indian Institute of Technology Kanpur, Kanpur	Dr Durgesh C. Rai
Indian Institute of Technology Madras, Chennai	Dr A. Meher Prasad
Indian Institute of Technology Roorkee,	Prof Ashok Jain
Indian Institute of Technology Roorkee, Roorkee	Dr D. K. Paul
Indian Institute of Technology, Gandhinagar	Dr S. K. Jain
Indian Meterological Department, New Delhi	Shri Surya Bali Jaiswar Shri Rajesh Prakash (*Alternate*)
Indian Road Congress, New Delhi	Secretary General Director (*Alternate*)

Organization	*Representative(s)*
Indian Society of Earthquake Technology, Roorkee	PROF D. K. PAUL PROF H. R. WASON (*Alternate*)
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This standard was originally formulated as part of project entitled 'Review of Building Codes and Preparation of Commentary and Handbooks' awarded to IIT Kanpur by the Gujarat State Disaster Management Agency (GSDMA) Gandhinagar, through World Bank finances.

The composition of the Committee responsible for the formulation of this standard is given in Annex A.

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 '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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