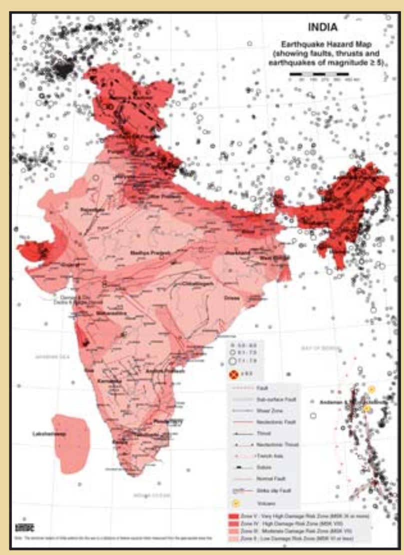
Guidelines

Improving Earthquake Resistance of Housing

2010



Building Materials & Technology Promotion Council
Ministry of Housing & Urban Poverty Alleviation
Government of India, New Delhi

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FOREWORD

I have great pleasure and pride in bringing out the BMTPC's latest publication entitled *Improving Earthquake Resistance of Housing: Guidelines* for the benefit of all the stakeholders involved in Earthquake risk management and mitigation. The publication is the updated version of BMTPC's earlier version with the same title which was drafted by Padmashree Prof. Anand S. Arya with the approval of expert group. The publication came in 2001. However, since then lot of changes have taken place as regards the strategy of Indian subcontinent is concerned towards combating earthquake risk . The Ministry of Home Affairs took over on Disaster related issues from Ministry of Agriculture and we have recognized that 3 Ps i.e. Prevention, Preparedness and Planning are more apt than being 3 Rs i.e. Reactive, Rescue and Recovery. Also, thanks to recurrent earthquakes in urban centers of India is last two decades, our knowledge in the area of earthquake resistant design and construction has increased manifold and several new concepts, technologies and tools have come up to understand the ever so complex behavior of structures under random earthquake loading. As a result, Indian standards have also gone through subtle changes such as obliterating Zone I (*now, we have only four zones starting from Zone II to Zone V*), bringing in more simplicity and new sections on irregularities, ductility, categorization of buildings etc. Therefore, it was felt obligatory to bring out newer version of our earlier published guidelines. The guidelines incorporate all the modifications being incorporated in Indian Standards. A separate annexure is added on Confined Masonry Technology as an alternative to the masonry construction reinforced at critical point being practiced in India for earthquake safe construction. This document would serve as an explanatory handbook on the various clauses of Indian Standards on Earthquakes which are important from the point of view of designing new buildings or improving resistance of existing building stock. The guidelines are in particular written keeping in mind that the major stock of buildings in India comprise of masonry buildings made up of mud burnt/un-burnt bricks and blocks & stones which are yet seldom designed for gravity and earthquake loads. Also, through these guidelines, we wish to pass on knowledge and expertise to our planners, engineers and architects and above all the common people of India to whom we owe, what have been learnt through all these menacing earthquakes. It has been endeavor of BMTPC to educate the masses and disseminate the knowledge in comprehensible lingo through its publication.

I place on record my deep and humble appreciation for Dr. Arya, Prof. Emeritus, IIT, Roorkee to take up the challenge and prepare the updated version of guidelines.

Let us build India as Earthquake Resilient Society.

8th Day of May, 2010
New Delhi

Dr. Shailesh Kr. Agrawal
Executive Director, BMTPC

PREFACE

The earthquake hazard has been well addressed by Committee CED:39 of Bureau of Indian Standards and the following standards are already printed:

1. IS:1983-(Part I) 2002 “Criteria for Earthquake Resistant Design of Structures (Fifth Revision)” 2002.
2. IS:13920-1993 “Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces - Code of Practice” November 1993*.
3. IS:4326-1993 “Earthquake Resistant Design and Construction of Buildings - Code of Practice (Second Revision)” October 1993*.
4. IS:13828-1993 “Improving Earthquake Resistance of Low Strength Masonry Buildings - Guidelines” August 1993.
5. IS:13827-1993 “Improving Earthquake Resistance of Earthen Buildings - Guidelines”, October 1993.
6. IS:13935-1993 “Repair and Seismic Strengthening of Buildings - Guidelines”, November 1993.*

**Presently under revision may be reprinted by end of 2010.*

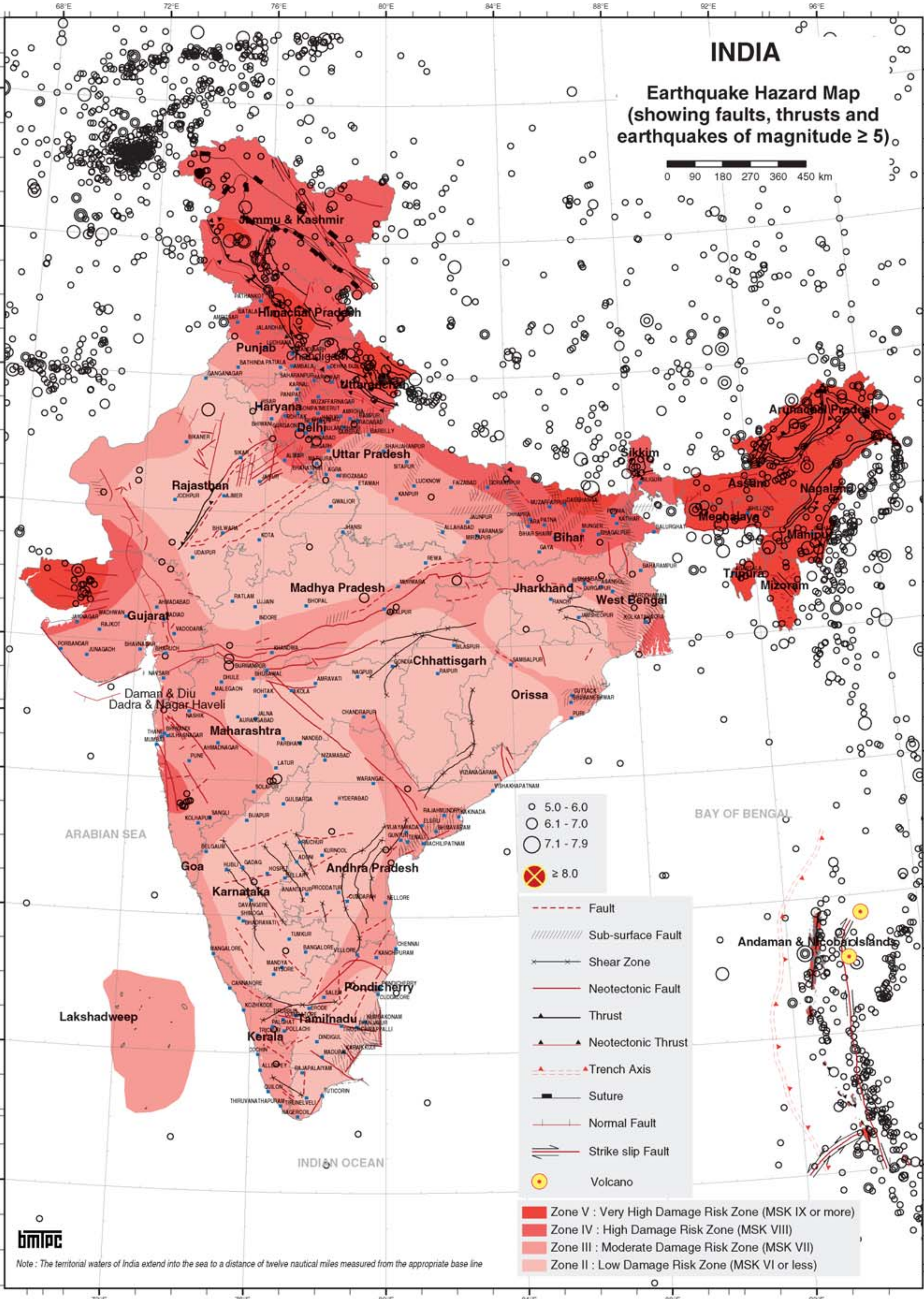
These standards taken together cover the professional design and construction requirements of buildings quite adequately. BMTPC, however, considered the need to prepare a brief guideline to explain the terms and principles underlying the occurrence of earthquakes, their effects on ground and buildings, and highlight the minimum safety provisions in buildings of various types commonly used for housing and built by people without involvement of engineering design and supervision. Obviously, the recommendations have to be made in line with those in the above standards and proper reference to them has to be made as necessary. Accordingly, these guidelines had first been prepared in 1998 and published in 1999-2002 for use by the people particularly those using the Vulnerability Atlas of India.

During the period 1999 to 2006, three major damaging earthquakes occurred in India namely, Chamoli (Uttranchal) 1999, Bhuj (Gujrat) 2001, and Kupwada (Jammu and Kashmir) 2005, as a result of which the main earthquake Code, IS: 1893 underwent major revision. The seismic zoning map was amended to reduce the number of Zones from 5 to 4 with many changes in the design seismic forces. Also, the categorization of the buildings was accordingly changed in IS: 4326. This first revision of the BMTPC guidelines incorporates all such modifications as a measure of updation. Besides, an earthquake safe construction technology, called as Confined Masonry Construction, has been included as an annexure, taking into view that inclusion of this technology is under consideration in the revision of IS: 4326.

Acknowledgement : These Guidelines were initially drafted by and now revised by the author. He will like to acknowledge heartily the assistance provided by the BMTPC staff in general and Dalip Kumar in particular. He feels indebted to Shri T.N. Gupta, the then E. D., and Dr. Shailesh Agrawal, the present Executive Director, BMTPC for providing the opportunity and the facilities of BMTPC for carrying out this work in the service of the people.

May 25, 2010

Dr. Anand S. Arya
Professor Emeritus, IIT Roorkee



BMTPC : Vulnerability Atlas - 2nd Edition (2006); Peer Group, MoH&UPA; Map is based on digitised data of SOI, GOI; Seismic Zones of India IS:1893 - 2002, BIS, GOI, Seismotectonic Atlas of India and its Environs, GSI, GOI

CONTENTS

IMPROVING EARTHQUAKE RESISTANCE OF BUILDINGS: GUIDELINES

1 INTRODUCTION

On the subject of earthquake resistant design and construction of buildings, various Indian Standards criteria, codes and guidelines are already framed and published by the Bureau of Indian Standard. The list of references given in the end contains all these along with others which are relevant in the design of buildings.

It will, therefore, be prudent that in the design and construction of buildings in relation to earthquake safety, whether by architects and engineers or in the informal sector, reference should be made to the basic code **IS:1893 (Part-1)-2002**. For engineered buildings, additional reference will be needed to **IS:4326-1993** and **IS:13920-1993**. Two codes, namely, **IS:13827-1993** and **IS:13828-1993** are specific to the low strength masonry and earthen buildings respectively. Such buildings are mostly in the rural and the semi-urban areas in the informal sector. The principles and details given in **IS:13935-1993** are for improving the seismic resistance of existing buildings through repair, and seismic strengthening or retrofitting techniques which are applicable to various building types, as per the nature and detail of the building.

2 SCOPE

2.1. The scope of these Guidelines is to highlight the essential and most important features of earthquake occurrence, its effects on habitat, and principles of earthquake resistant design and construction, which would achieve 'non-collapse' protection of buildings from the onslaught of probable maximum earthquakes,as covered in the codes and guidelines referred above in **1**.

2.2 All buildings from reinforced concrete to the earthen houses are covered herein. The methodology adopted is to state the essential principles and make a reference as necessary to the relevant Indian Standard for detailed treatment. The most important features only are presented here.

2.3 The reference to the clauses of a particular standard is written as **4326-1.2** which will mean clause 1.2 of **IS:4326-1993**, (which deals with Mortar to be used in Masonry constructions in seismic zones).

2.4 These guidelines cover the following features from earthquake safety view point:

a. Siting and foundations
b. Architectural design features
c. Structural analysis and design
d. Construction and strengthening features in load bearing walls.
e. Construction and strengthening features of roofs and floors
f. Repair and strengthening of damaged buildings
g. Seismic retrofitting of existing weak buildings

3 TERMINOLOGY

Some of the technical terms used herein are defined below:

3.1 Band

A reinforced concrete, reinforced brick or timber runner provided in the walls to tie them together and to impart horizontal bending strength in the walls. Plinth band, lintel band, roof band, eave level band and gable band are names used for the band depending on the level of the building where the band is provided.

3.2 Centre of Mass

The centre of gravity of all the masses of roofs/floors and the walls above any storey of the building.

3.3 Concrete Grades

28 day crushing strength of concrete cubes of 150 mm size, in MPa; for example, for Grade M15 of **IS:456-2000**, the concrete strength will be equal to 15 MPa.

3.4 Design Horizontal Seismic Coefficient

The value of horizontal seismic coefficient computed taking into account the seismic zone factor Z (**1893 - Tabel-2**) the importance factor I, (**1893 - Table-6**) the response reduction factor R, (**1893 - Table-7**) and the average response acceleration efficient, S_a/g (**Fig.2**) based on hardness of the base soil as specified in **1893-6.4.2.**

3.5 Engineered Buildings

Buildings designed by architects and/or engineers and properly supervised by engineering staff during construction such as reinforced concrete and steel framed buildings.

3.6 Non-engineered Buildings

Those buildings which are spontaneously and traditionally built by masons and carpenters without inputs from architects or engineers involved in design or construction, such as houses built using traditional materials namely, stone, burnt-brick, clay mud or adobe, wood and other bio-mass materials.

Note 1: Reinforced concrete or steel column-beam construction carried out by masons without proper analysis and design for lateral seismic loads will also fall in the category of non-engineered buildings.

3.7 Pre-Engineered Buildings

Those non-engineered buildings which comply with the provisions in **IS:4326, IS:13827, IS:13828** and **IS:13935** in their construction and seismic strengthening could be termed as pre-engineered buildings.

3.8 Seismic Zone

The seismic zones II to V as classified in **IS:1893:2002 (Fig.1).**

3.9 Semi-Engineered Buildings

Buildings which have certain elements structurally designed such as roof slabs and foundations but certain elements not properly designed such as walls of masonry buildings, and in which the

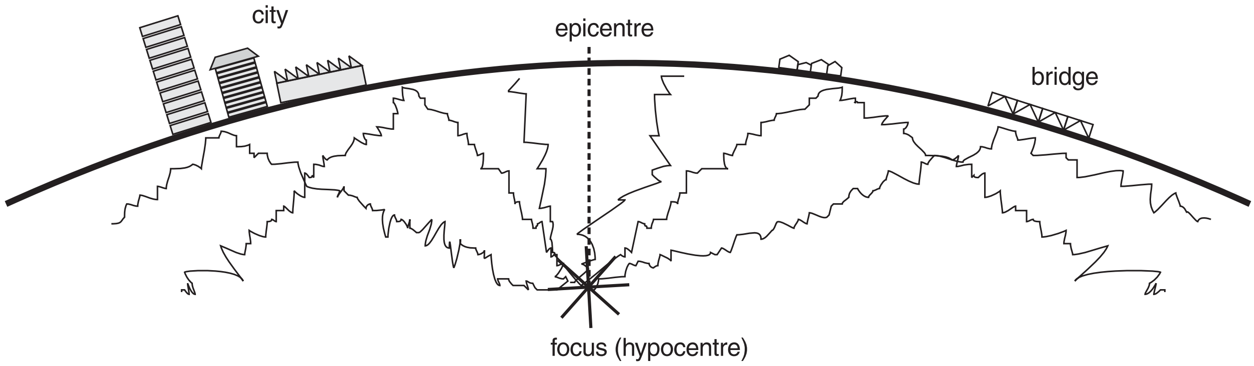


Fig. 1
Earthquake occurrence

supervision may be through engineering staff or otherwise. Many buildings of this type are planned by architects and built by private parties through petty contractors without efficient supervision.

4 UNDERSTANDING EARTHQUAKES

4.1 Earthquake Vibrations

Vibrations of earth's surface caused by waves coming from a source of disturbance inside the earth are described as 'Earthquake'.

By far the most important earthquakes from an engineering standpoint are of tectonic origin, that is, those associated with large scale strains in the crust of the earth. The theory describing this phenomenon is termed as Elastic Rebound Theory, according to which the strain energy that accumulates due to deformation in the earth mass, gets released when the resilience of the storing rock is exceeded. The energy released through rupture is propagated in the form of waves which impart it to the earth mass through which they pass and vibrate the structures standing on it *(Fig.1).*

A major tectonic earthquake is generally preceded by small 'foreshocks' caused either by small ruptures or plastics deformations, and is followed by 'aftershocks' due to fresh ruptures or the readjustments of the fractured mass. A shock may result from a rupture of rock over a length of few to hundreds of kilometres and several kilometres wide and thick. The bigger is the mass that ruptures at one time, the bigger is the earthquake.

Small earthquakes are also caused by volcanic eruptions, rock-bursts or subsidence in mines, blasts, impounding of reservoirs, pumping of oil, etc. They may cause considerable damage in small areas, but vast areas are shaken only by tectonic movements across active faults as explained above.

4.2 Measurement of Ground Motion

Two types of instruments are in vogue to record earthquake ground motions. The instruments called *seismographs* are generally sensitive and meant for recording weak motions of earth. For engineering measurements, the instruments, *accelerographs* generally operate when the ground motion exceeds a threshold value of acceleration which is pre-adjusted, and are expected to record the strongest acceleration of ground motion. Both types are complementary to each other and provide useful data in earthquake engineering.

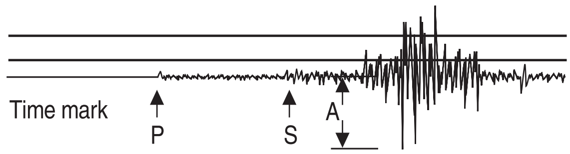


Fig. 2
A typical earthquake record

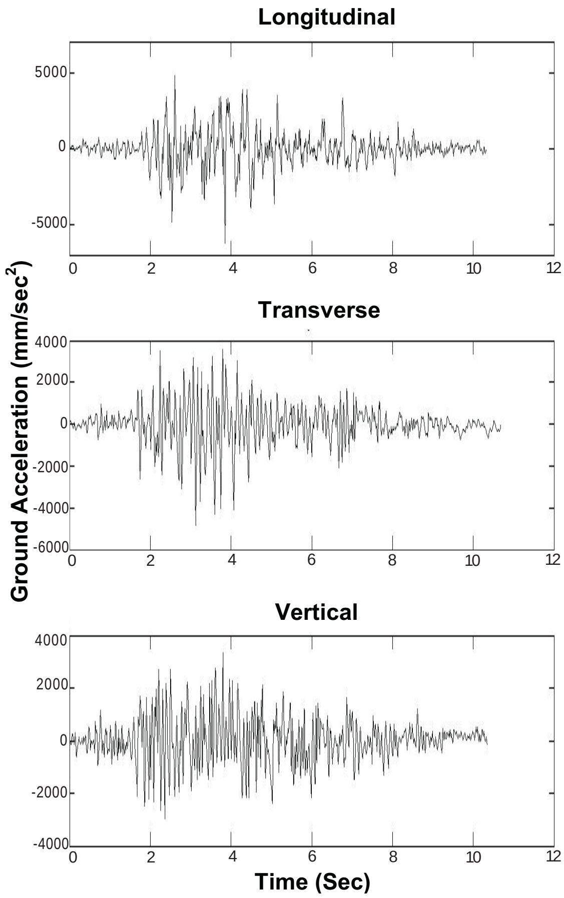


Fig. 3
Accelerograms of Koyna Earthquake (India) 11 December 1967

4.3 Epicentre, Hypocentre and Earthquake Waves

The point inside the earth mass where slipping or fracture begins is termed as 'focus' or 'hypocentre' and the point vertically above the focus on the earth's surface is termed as 'epicentre', as shown in *Fig. 1*. The position of the hypocentre is determined with the help of seismograph records obtained at many seismic stations around the world, which indicate the arrival times of different types of energy waves. 'Compressional' waves, which are also termed as 'longitudinal' or 'primary' (P) waves, travel the fastest; the 'shear' or 'transverse' or secondary' (S) waves, travel slower and the 'surface' or 'Rayleigh' (R)/'Love' waves the slowest. Thus on a seismograph station they arrive at different times. See *Fig. 2.* Using this time difference and the average velocities of different waves, the distance of the 'focus' from a point of observation is obtained. Such observations at several stations are used to locate the 'focus'; hence the *epicentre* too.

4.4 Strong Ground Motion

The basic data needed for design of engineering structures is a record of *ground acceleration versus time*. Accelerographs which have the characteristics of being rugged and capable of withstanding shock and also easily calibrated, are used for recording strong motion. A typical strong motion accelerograph would have three accelerometers, two horizontal to record motion in X and Y directions respectively and one vertical to record motion in the Z direction. A typical strong motion record is shown in *Fig. 3.*

The instrument, unlike the seismograph, does not operate continuously. A trigger is used which operates the recording in the accelerograph unit when the ground acceleration exceeds a particular threshold level (usually adjustable in modern instruments). The instrument would continue to record, for a fixed time beyond the last pulse exceeding threshold level. The speed of recording is much more in accelerograph as compared to seismograph in order to have a better resolution of the record.

4.5 Magnitude and Energy of an Earthquake

As designated by Richter, the "Magnitude" of an earthquake is standardised as "Logarithm (to the base 10) of the maximum amplitude of the ground motion as recorded in millimeters at a distance of 100 km from the epicentre on a Wood-Anderson Torsion Type seismograph with period of 0.8 second and magnification of 2800." Since the distance of the instrument from the epicentre will usually not be exactly 100 km, the following equation will determine the Magnitude 'M':

$M = \log A - \log A_o$ .... *(1)*

Table 1:
Approximate relationships between M, MM intensity and felt area

Earthquake magnitude Richter M	Expected annual number	Maximum expected intensity MSK	Radius of felt area km	Felt area $(km)^2$
4.0 - 4.9	6,200	IV - V	50	7,700
5.0 - 5.9	800	VI - VII	110	38,000
6.0 - 6.9	120	VII - VIII	200	125,000
7.0 - 7.9	18	IX - X	400	500,000
8.0 - 8.9	1	XI - XII	800	2,000,000

Source: "Earthquakes" by Don de Nevi, Celestial Arts, Calif., May 1977 - p.102.

where 'A' is the trace amplitude *(refer to Fig. 2)* at any station and $\log A_o$ the distance correction for near as well as for distant earthquakes. A correction, for the type of instrument or reliability of observations depending upon local conditions, is further applied to get the true magnitude 'M'. Values obtained at various stations are then compared and a mean value of Magnitude is assigned to the earthquake.

Earthquakes may have a Magnitude from less than 1 upwards to more than 9, but no shock smaller than 5.0 causes appreciable damage and earthquake larger than 9.5 is improbable. The extent of damage depends upon the depth of focus, shallower earthquake causing higher damage in smaller areas. Very shallow shocks even of small size could cause damage locally. Usually earthquakes have their focus not shallower than about 5 km and could go deeper than 300 km.

An earthquake of magnitude 5.0 may cause damage within a radius of about 8 km but that of magnitude 7.0 may cause damage in a radius of 80 km and that of 8.0 over a distance of 250 km. The felt area of these earthquakes will have their radius equal to about 110, 400 and more than 800 km respectively. Fortunately, damaging earthquakes (M>5.0) are not as frequent as the smaller ones, and the major ones (M>7.0) occur only rarely (see *Table 1).* But whenever such a large earthquake does occur, the devastation caused is indeed very large. It may be clarified that the damage-area of an earthquake is not circular but rather elliptical in one direction with the epicentre eccentrically placed in it. Also the damage area may be extended along river courses due to local soil effects.

The well known damaging earthquakes in India are listed in *Table 2.*

A relationship between strain energy 'E' released by an earthquake and its magnitude is given by Richter as follows:

$\log_{10} E = 11.4 + 1.5\, M$ .... *(2)*

Table 2:
Some better known damaging earthquakes in India

Year	Area	Date	(IST) Time hr:m:s	Latitude degrees North	Longitude degrees East	Magnitude M	Max. MM Int.	Deaths
1819	Gujarat (Kutch)	Jan.16	Mid Night	-	-	8.0	XI	Many Thousand
1833	Bihar	Aug.26	-	27.5	86.5	7.7	XI	Hundreds
1897	Assam (Shillong)	Jun.12	16:36:-	25.0	92.0	8.7	XII	1600
1900	Kerala (Palghat)	Feb 8	-	10.7	76.7	6.0	-	
1905	Himachal Pradesh (Kangra)	Apr 4	06:20:-	32.5	76.5	8.0	XI	20000
1930	Assam (Dhubri)	Jul 3	02:33:34	25.8	90.2	7.1	IX	Many*
1934	Bihar -Nepal	Jan 15	14:13:26	26.6	86.8	8.3	XI	14000
1941	Andamans	Jun 26	-	12.4	92.5	8.0	X	Many
1943	Assam (NE)	Oct 23	22:53:17	26.8	94.0	7.2	X	
1950	Assam (NE)	Aug 15	19:39:28	28.7	96.6	8.6	XII	1500
1956	Gujarat (Anjar)	Jul 21	21:02:36	23.3	70.0	7.0	VIII	Hundreds
1956	Uttar Pradesh (Bullandshahar)	Oct 10	-	28.1	77.7	6.7	VIII	Many
1958	Uttar Pradesh (Kapkote)	Dec 28	-	30.0	80.0	6.3	VIII	Many
1960	Delhi	Aug 27	21:28:59	28.3	77.4	6.0	VII	
1963	Kashmir (Badgam)	Sep 2	07:04:32	33.9	74.7	5.5	VII	Hundreds
1966	Western Nepal	Jun 27	-	29.5	81.0	6.3	VIII	
1966	Uttar Pradesh (Moradabad)	Aug 15	-	28.0	79.0	5.3	VII	
1967	Nicobar	Jul 2	-	9.0	93.4	6.2	-	
1967	Maharashtra (Koyna)	Dec 11	04:21:19	17.4	73.7	6.5	VIII	200
1970	Andhra Pradesh (Bhadrachalam)	Apr 13	-	17.6	80.6	6.5	VII	
1970	Gujarat (Broach)	Mar 23	07:23:03	21.7	72.9	5.7	VII	
1975	Himachal Pradesh	Jan 19	-	32.5	78.4	6.5	VIII	
1988	Bihar - Nepal	Aug 21	04:39:10	26.76	86.62	6.6	VIII	1003
1991	Uttar Pradesh (Uttarkashi)	Oct 20	02:53:-	30.75	78.86	6.6	VIII	715
1993	Maharashtra (Killari)	Sep 30	03:55:47	18.07	76.62	6.3	VIII	7928
1997	Jabalpur	May 22	04:22:31	23.1	80.1	6.0	VII+	38
1999	Uttaranchal (Chamoli)	Mar 29	00:35:13	20.45	79.42	6.8	VIII	hundreds
2001	Gujarat (Bhuj)	Jan 26	08:46:43	23.40	70.28	6.9** (7.7)	X	13800

* *Many will mean less than a hundred*   *** 6.9 on Richter Scale & 7.7 on Moment Magnitude*

Energy released in earthquake of different magnitudes would give an idea of their relative destructive power. In a damaging earthquake it will be of the order of 10^{20} to 10^{25} ergs. Due to the logarithmic scale, the destructive energy of an earthquake of M+1 is about 31.6 times that of magnitude M. Thus E of Magnitude 8.0 earthquake will be 1000 times E of M 6.0 earthquake.

5 EARTHQUAKE EFFECTS

5.1 Force Generation Mechanics

The above understanding of earthquakes indicates that an earthquake causes vibratory ground motion. It is not an external force applied to a building, structure or system like wind pressure, weights of materials or the traffic on bridges, etc.

Then, how is the so called earthquake force caused which can destroy every thing? Refer to *Fig. 4* showing a single storey single bay framework.

When the ground moves suddenly to the right from A to A” *(Fig. 4b)*, the columns which were straight before, will tend to bend since the top weight will tend to remain behind and an inertia force will act to the left. The reverse will occur when the ground will move to the left from A to A’, the inertia force will occur to the right. Since the ground vibrates randomly bothways, the top weight will also vibrate both ways from the mean position. The bending of the columns will

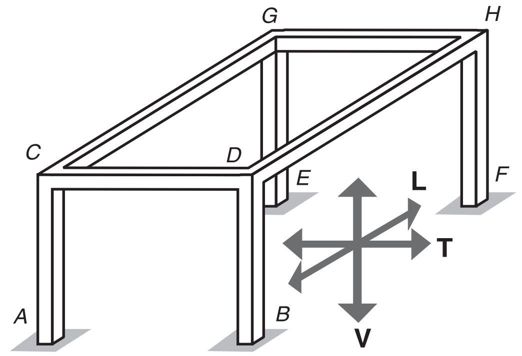


fig 4 a
Section of building

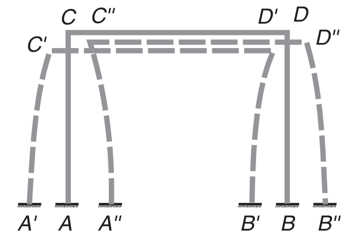


fig 4 b
Transverse vibration **(T)**

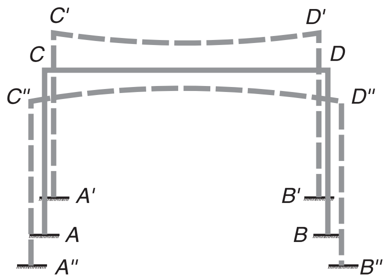


fig 4 c
Vertical vibration **(V)**

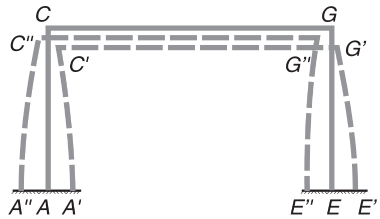


fig 4 d
Longitudnal vibration **(L)**

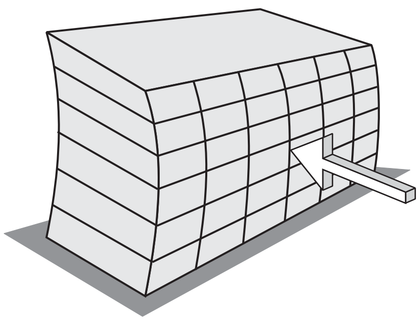


fig 4 e
Resultant inertia force of building

Fig. 4
Seismic vibrations of a building and resultant earthquake force

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also occur both ways. Similar effects take place when the earthquake vibrations shake the building vertically up and downward as shown at (c) or longitudinally as shown at (d) in *Fig. 4*. Since the earthquake motion can be resolved in 3 perpendicular directions, the building usually vibrates and develops forces along all three principal axes. Hence the ground motion creates forces in the building (*Fig. 4e*) due to:

i. the building has weight (or mass)
ii. the mass is connected with the ground through columns (or walls) which resist the forces created by the relative movement of the top with the base.

Now it is found that greater the mass and stiffer the resisting member, larger the force produced in the structure. *Thus the earthquake ground motion is converted by the structure itself into forces that act on its various elements.*

5.2 Nature of Seismic Stresses

The structural elements such as walls, beams and columns which were bearing only vertical loads before the earthquake, have now to carry horizontal bending and shearing effects as well. When the bending tension due to earthquake exceeds the vertical compression, net tensile stress will occur. If the building materials are weak in tension, such as brick or stone

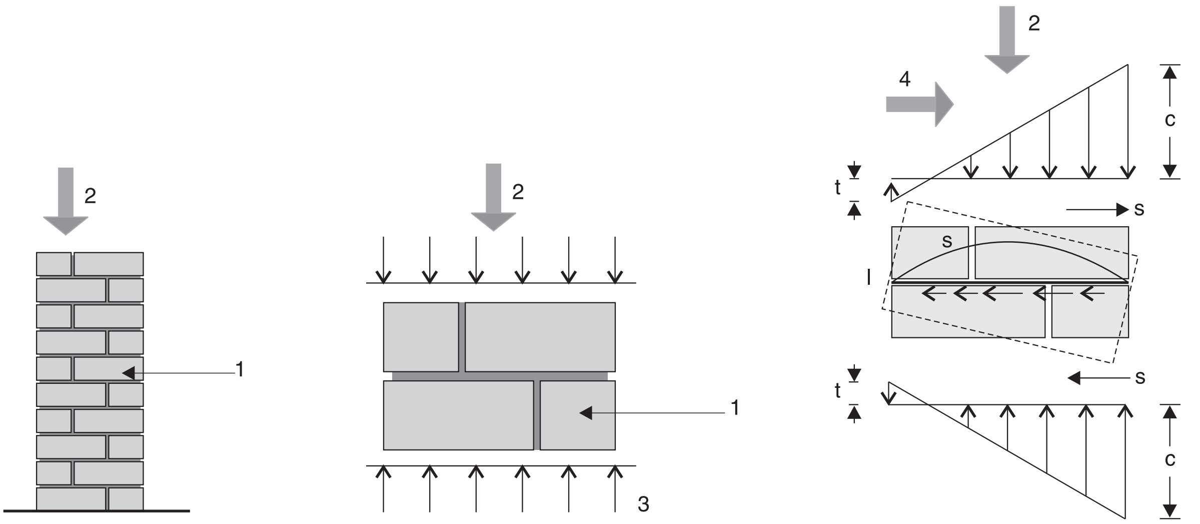


fig 5 a
Wall

fig 5 b
Stresses before earthquake

fig 5 c
Stresses during earthquake

Legend
1 wall element
2 vertical load from above the wall element
3 reaction from below
4 earthquake force

c compressive stress
t tensile stress
s shearing stress

Fig. 5
Stress condition in a wall element

masonry, cracking occurs which reduces the effective area for resisting bending moment, as shown in *Fig. 5.* It follows that the strength in tension and shear is important for earthquake resistance.

5.3 Factors Affecting Structural Damage

Whether a building or structure will remain undamaged, get somewhat damaged, or collapse completely, will depend on the following three factors:

i. The intensity of earth shaking (indicated by the ground accelerations caused at the base of the structure)
ii. The dynamic parameters of the structure (namely, the masses of various elements, their stiffnesses, and deformation-energy dissipating capacity or damping)
iii. The strength of the foundation soils; the load resisting capacity of the individual elements, their connections and the whole assembly for carrying the earthquake forces produced in conjunction with other concurrently applied dead and live loads.

Since the soils, the buildings and other structures, are varying greatly in their characteristics as well as strength of materials and the design details, it is not surprising that they show quite different behaviour during a given earthquake occurrence. Apparently those of weaker materials such as earthen walls; stone masonry and brickwork built using weak mortars, and having heavy roofs, suffer much more severe damage including collapse, than those built using good cement mortar and lighter roofs or those light weight wooden buildings which are provided with secure connections. Similarly, reinforced concrete or steel buildings of good design and detail usually escape without damage except in very high intensity earthquakes, but those of poor design quality and inadequate detailing may fail even under moderate intensity earth-

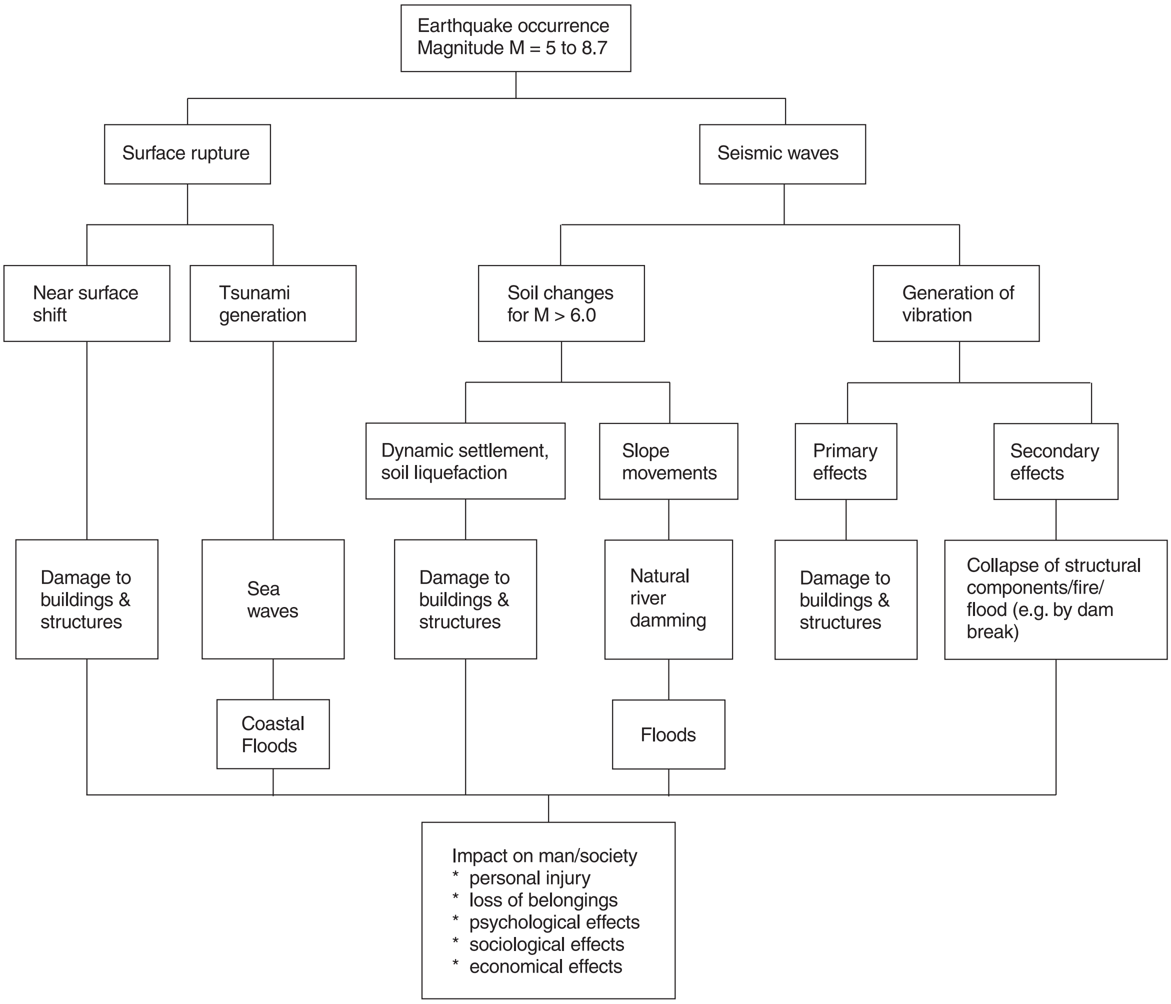


Fig. 6
Possible overall effects of earthquake hazards
Source: UNDRO

quakes. Therefore *for earthquake safety, not only the building materials should be strong and of good quality but also the design and detailing as well as quality of construction should be good* and according to the Indian Standard Specifications.

5.4 Overall Effects of Major Earthquakes

The possible overall effects of earthquakes on ground, buildings and structures are shown in *Fig. 6*. The extent of damage will naturally depend on the earthquake magnitude and the local conditions, higher the magnitude and weaker the soils, more extensive and catastrophic will be the earthquake effects on buildings and structures. Hence to minimise the disastrous effects of an earthquake, which would have the probability to occur in future, ample preventive measures need to be adopted in *every* construction scheme: new settlements, buildings and bridges; transportation and canal systems; airports and dams; communications and fire stations; water supply and sewerage systems; schools and hospitals; community and religious structures; etc. *Nothing infact remains unshaken whenever a major earthquake occurs, and the safety depends only on the measures adopted in the design and construction from seismic resistance viewpoint.* Fortunately, appreciable and effective knowhow exists now in the form of codes and standards and other published literature by the use of which *non-collapsable structures are feasible at not-too-great an additional cost*.

TABLE 3:
CATEGORIES OF SEISMIC DAMAGE

Damage Category		Extent of Damage in General	Suggested Post-Earthquake Action
O	No damage	No damage	No action required
G1	Slight Non-structural Damage	Thin cracks in plaster, Falling of plaster bits in limited parts.	Building need not be vacated. Only architectural repairs needed.
G2	Slight Structural Damage	Small cracks in walls, falling of plaster in large bits over large areas; damage to non-structural parts like chimneys, projecting cornices, etc. The load carrying capacity of the structure is not reduced appreciably.	Building need not be vacated. Cracks in walls need grouting. Architectural repairs required to achieve durability. Seismic strengthening is desirable
G3	Moderate Structural Damage	Large and deep cracks in walls; widespread I.c., cracking of walls, columns, piers and tilting or falling of chimneys. The load carrying capacity of structure is partially reduced.	Building needs to be vacated, to be reoccupied after restoration and strengthening. Structural restoration and seismic strengthening are necessary after which architectural treatment may be carried out.
G4	Severe Structural Damage	Gaps occur in walls; inner or outer walls collapse; failure of ties to separate parts of buildings. Approximately 50 percent of the main structural elements fail. The building takes a dangerous state.	Building has to be vacated. Either the building has to be demolished or extensive restoration and strengthening work has to be carried out before reoccupation.
G5	Collapse	A large part or whole of the building collapses.	Clearing the site and reconstruction.

Source: Guidelines for Earthquake Resistant Non-Engineered Construction published by IAEE, 1986.

6 INTENSITY AND ISOSEISMALS OF AN EARTHQUAKE AND SEISMIC ZONING

6.1 Intensity Scales

The intensity of an earthquake as felt or observed through damage is described as “Intensity” at a certain place on an arbitrary scale. A 10 point scale was first devised by Rossi-Forel (1885) and changed to 12 point later by Mercalli (1904). It was modified in 1931 by Neuman and came to be known as Modified Mercalli Scale. The Intensity Scale was further detailed in 1964 now called MSK as given in **IS:1893-Appendix D**. The MSK scale, is now in use generally, and is presented in *Annexure-1* where the damaging intensities V to IX only are listed for ready reference. From the description it will be seen that the Intensity Scale presents a qualitative description of the intensity of shaking experienced at a place. Naturally the Intensity will decrease with distance from the epicentre. The bigger the earthquake, higher will be

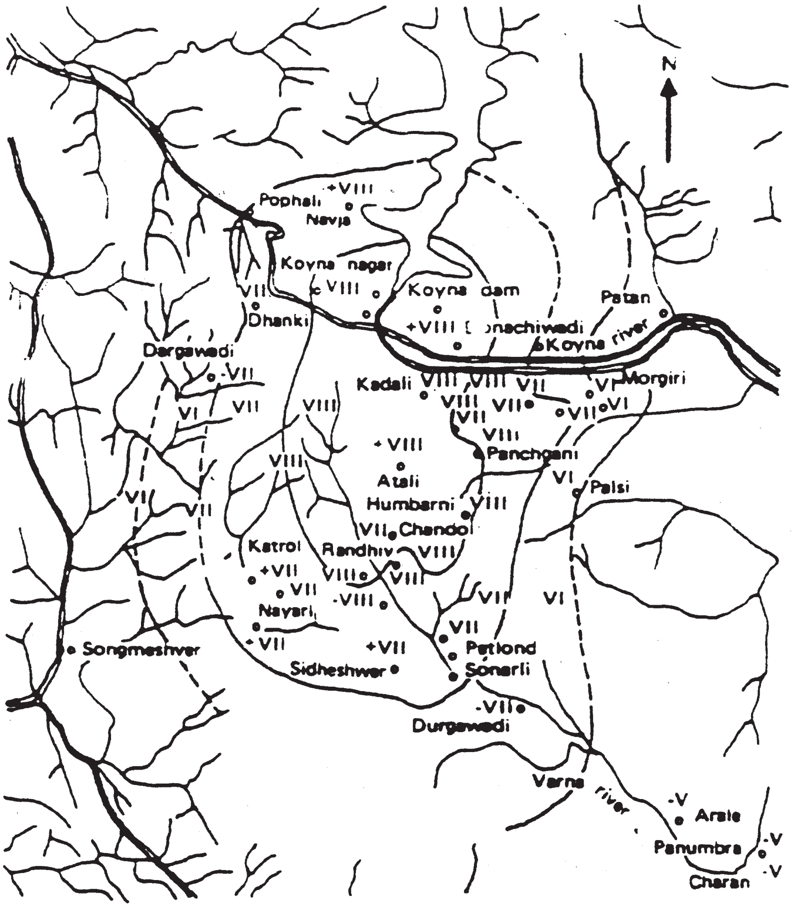


Fig. 7
Isoseismal map
(showing intensities during 1967 Koyna earthquake of M=6.5)

the maximum Intensity caused and larger will be the area covered by each Intensity.

The maximum intensity of shaking attained during an earthquake of given magnitude depends upon the depth of focus as well as soil condition. For shallow focus earthquakes, of depth about 30 km or less, an approximate relationship may be expressed between Magnitude and Maximum Intensity in the epicentral area. Representative values are given in *Table 1.*

6.2 Isoseismals of an Earthquake

Observation of Intensities are made soon after the occurrence of a damaging earthquake through an on-the-spot study of effects and damages according to the Intensity Scale. There are five grades of damage under which the building damage is classified as shown in *Table 3.* The Intensity to be assigned to an area will depend on the maximum damage sustained by a building type and percentage of such damaged buildings to the total of this building type in that area.

A map of the affected area is then prepared on which the Intensities assigned to various places are marked. Areas having the same Intensity are then enclosed by contour lines separating the areas of different Intensities. Such a map is called an 'Isoseismal Map'.

Fig. 7 shows the Isoseismal map of the Koyna Earthquake of Dec.11, 1967, where the maximum intensity reached was VIII.

6.3 Seismic Zoning

The earthquake activity in different parts of India is not of the same severity. Hence, the country had earlier been classified into five zones I to V so that the forces for which structures are to be designed at any site are varied according to the severity of probable earthquake Intensities (See **IS : 1893-1984** - Zoning Map). The maximum Intensities considered for the five zones were (see *Annexure 1* for descriptions of MSK Intensities):

MSK IX or more in Zone V,
MSK VIII in Zone IV,
MSK VII in Zone III,
MSK VI in Zone II, and
MSK V or less in Zone I.

In the fifth revision of IS:1893 in 2002, the Seismic Zoning Map has also been revised to eliminate Seismic Zone I by merging into Seismic Zone II. Therefore, now only four Zones exist as zones II, III, IV and V in which the area covered by Zones IV and V remains unchanged, Zone III has been enlarged in peninsular India and Zones II covers earlier Zones I & II with minor additional modifications.

This zoning has been worked out primarily depending on the known seismic history of the regions, the postulated seismic activity in future, and the indicative time intervals between two consecutive occurrences in the same area. In Peninsular India, for example, although the maximum intensity of VIII has occurred at certain points such as Koyna (1967), Killari (1993) and Jabalpur (1997), the time interval is so large, a few centuries, between two such intense occurrences in the same area, that, from economical considerations, the zoning of lower intensity may be adopted there.

It is also emphasised that the adopted zoning is directly applicable for the design of average structures. But for important projects such as large dams, major bridges, refinery projects. etc., it is necessary to carry out geological and seismological studies for the site in question to arrive at site-dependent design parameters and seismic forces to cater for future earthquakes.

7 EARTHQUAKE FORCE FOR DESIGN

The earthquake force for design of various types of buildings is specified in IS:**1893 (Part-I)-2002** in detail and reference must be made to that standard. The information considered relevant for understanding the process of design is given below for ready reference.

7.1 The Earthquake Force

The forces caused in a structure when an earthquake ground motion passes underneath depend on its own dynamic characteristics, particularly the following:

a. *Mass and stiffness distribution.* The natural periods, the mode shapes and mode participation factors are derived for the structure in the elastic range. The fundamental period T of natural vibrations is crucial in determining the earthquake force for design.

b. *Energy dissipation property.* When a structure is vibrated, it dissipates the input dynamic energy through its elements, the supports and the foundations, and the vibrations get damped. *Larger is the damping, less the force developed.*

c. *Inelastic energy dissipation.* Besides the energy dissipation during the elastic range, well designed ductile structures can dissipate large amount of energy beyond yield deformation. But brittle structures do not have this capacity except through friction which develops after their cracking. In this respect steel is ductile while plain concrete and all types of masonry remain brittle. Therefore, for earthquake safety against collapse, proper *reinforcing of concrete and masonry with steel bars is considered crucial.* Steel frame structures, though made from a ductile material, may also suffer due to *buckling instability*, or due to *fracture at joints*. Hence proper detailing of the elements and the connections will be equally important in steel buildings also.

7.2 Design Seismic Coefficient

The design seismic coefficient is specified in, IS:**1893 (Part-1)-2002** by using 'design response spectrum' method (see **IS:1893-6.4**). Two levels of earthquake force are considered:

(i) Maximum Considered Earthquake (MCE) is the most severe earthquake peak ground acceleration (PGA) specified for the seismic zone in IS:**1893** e.g. Z = 0.36 for zone V, 0.24 for IV, 0.16 for III and 0.1 for Zone II.

(ii) Design Basis Earthquake (DBE) is the earthquake which can reasonabily be expected to occur at least once in the design life of the building (say 50 to 100 years). In the absence of apporpriate value of the earthquake, DBE is taken *half* of MCE.

The design horizontal seismic coefficient A_h is given by:

$$A_h = \frac{Z \,.\, I \,.\, Sa}{2 \;R \;\; g} \quad \text{......(3)}$$

where:

Z= MCE zone factor. The factor 2 in the denominator is used to reduce MCE to DBE

I = a factor to increase the safety of important and hazardous structures and systems: 1.0 for ordinary buildings; 1.5 for important service and community building such as schools, hospitals, water tanks, emergency buildings like telephone exchanges and fire brigades, large assembly structures, etc.; may be taken even higher but less than or equal to 2.0 for special buildigns if so decided by the user.

R= Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0. A few value of R for buildings are given below:

S.No.	Building Frame Systems	R
i	*Load bearing masonry wall buildings*	
	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
	d) Confined masonry buildings	3.0
ii	*RC frame buildings*	
	a) Ordinary RC moment-resisting frame (OMRF)	3.0
	b) Special RC moment-resisting frame with full ductile details	5.0
iii	Ordinary reinforced concrete shear walls	3.0
iv	Ductile shear walls	4.0
v	*Buildings with Dual systems*	
	a) Ordinary shear wall with OMRF	3.0
	b) Ordinary shear wall with SMRF	4.0
	c) Ductile shear wall with OMRF	4.5
	d) Ductile shear wall with SMRF	5.0

* Sa/g = Average response acceleration coefficient for rock or soil sites as given by IS:**1893-Fig.2**.

7.3 Design Base Shear in Buildings

After the Seismic co-efficient A_h is determined based on the time period and damping values, the design base shear force in buildings is calculated by

$$V_B = A_h W \quad \text{......} \quad (4)$$

where:

W = total dead load (DL) + appropriate amount of live load (IL)

DL = sum of weights of all permanent elements and fixtures above the base of the building

IL = percent live load assumed to be the part *having mass and actually present* at the time of earthquake occurrence (25 to 50 per cent depending on loading class, see **1893 (Table-8)*.***

Note: It is assumed that the earthquake is not likely to occur simultaneously with wind or maximum flood or maximum sea waves.

7.4 Multistorey Buildings

7.4.1 For multistoried frame building, a number of modes are to be considered. For dynamic analysis, the value of A_h as defined for each mode should be determined using the natural period of vibration of that mode. The values of moments, shears and thrusts calculated in each beam and column in each mode are to be combined to get the total values under the earthquake force. Depending on the regularity conditions of the building frame, different rules have been given in IS:1893 for combination of modal values (see **1893:7.8.4.4. & 7.8.4.5**)

7.4.2 Regular and Irregular Configuration

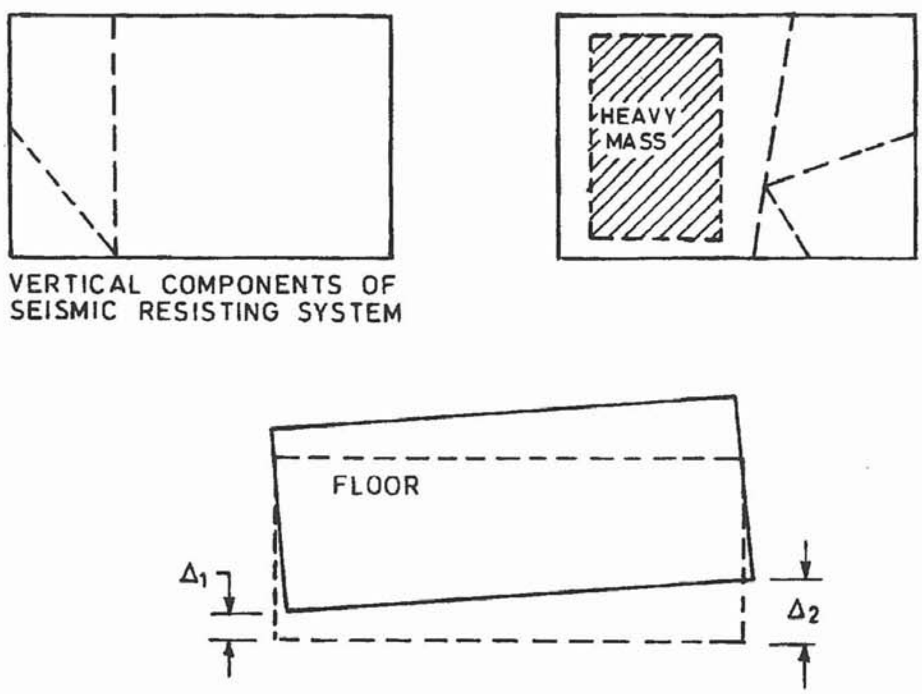
To perform well in an earthquake, a building should possess four main attributes, namely simple and regular configuration, and adequate lateral strength, stiffness and ductility. Buildings having simple regular geometry and uniformly distributed mass and stiffness in plan as well as in elevation, suffer much less damage than buildings with irregular configurations. The irregular buildings are defined in ***1893-7.1 as:***

a. *Plan Irregularities are* **(1893-7.4, Fig.3)***:*
   i) Torsion Irregularity
   ii) Re-Entrant corners
   iii) Diaphragm Discontinuity
   iv) Out- of- Plane Offsets
   v) Non-parallel Systems

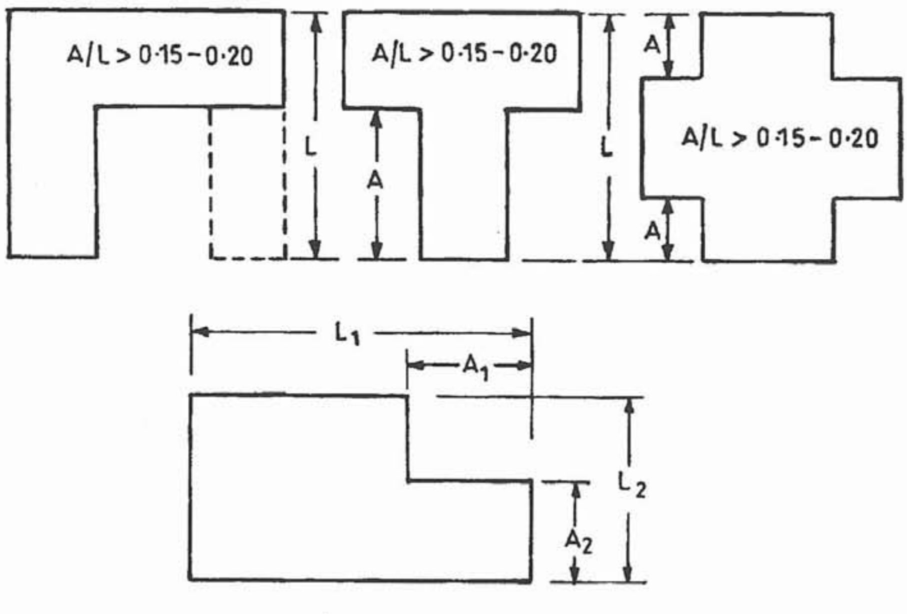
b. *Vertical Irregularities are defined :* **(1893-7.4, Fig.4)**
   i) Mass Irregularity
   ii) Vertical Geometric Irregularity
   iii) In-Plane Discontinuity in Vertical Elements Resisting Lateral Force
   iv) Stiffness Irregularity – Soft Storey
   v) Stiffness Irregularity – Extreme Soft Storey
   vi) Discontinuity in Capacity- Weak Storey

7.4.3 Empirical Fundamental Natural Period

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

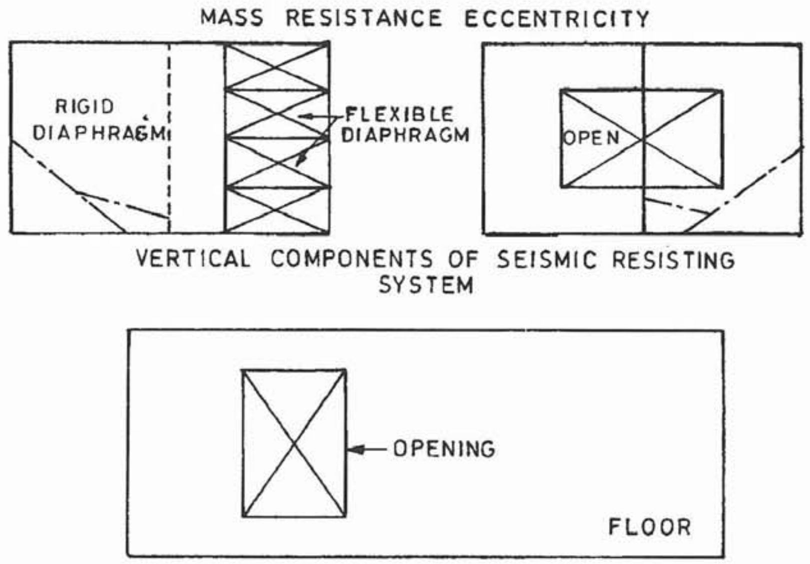


3 A Torsional Irregularity

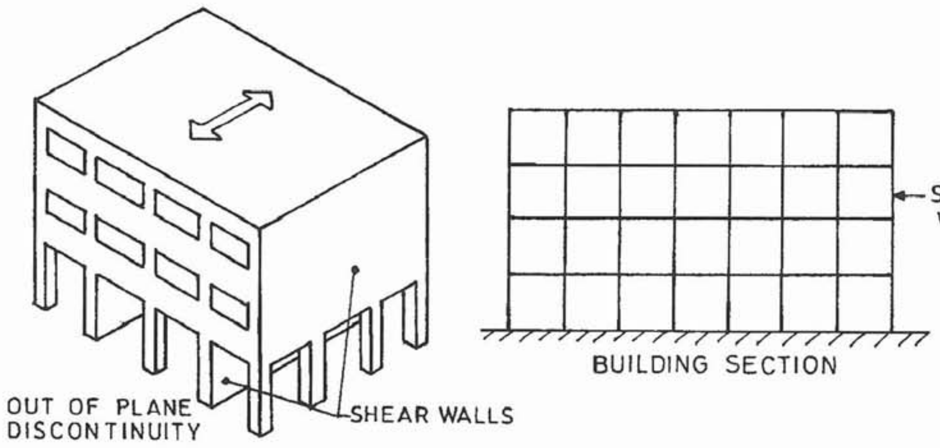


3 B Re-entrant Corner

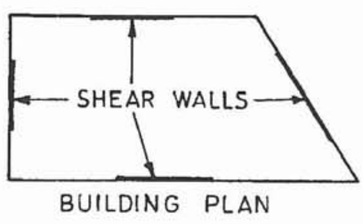
IS 1893 (Part 1) : 2002 FIG. 3 PLAN IRREGULARITIES — *Continued*



3 C Diaphragm Discontinuity

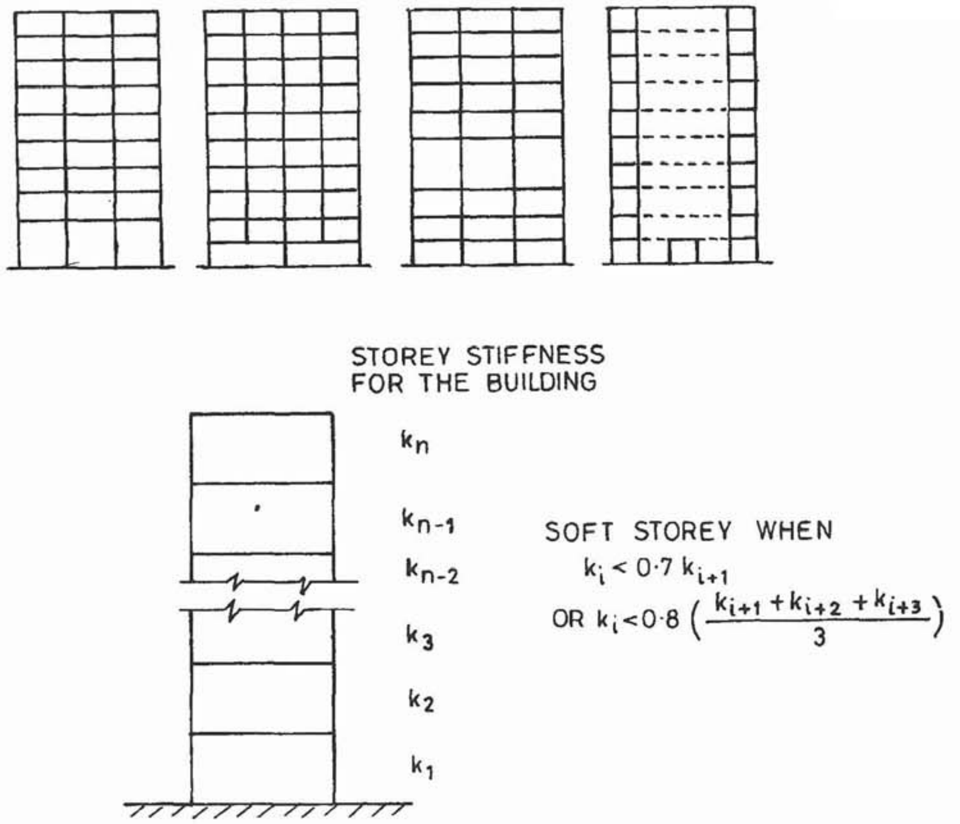


3 D Out-of-Plane Offsets

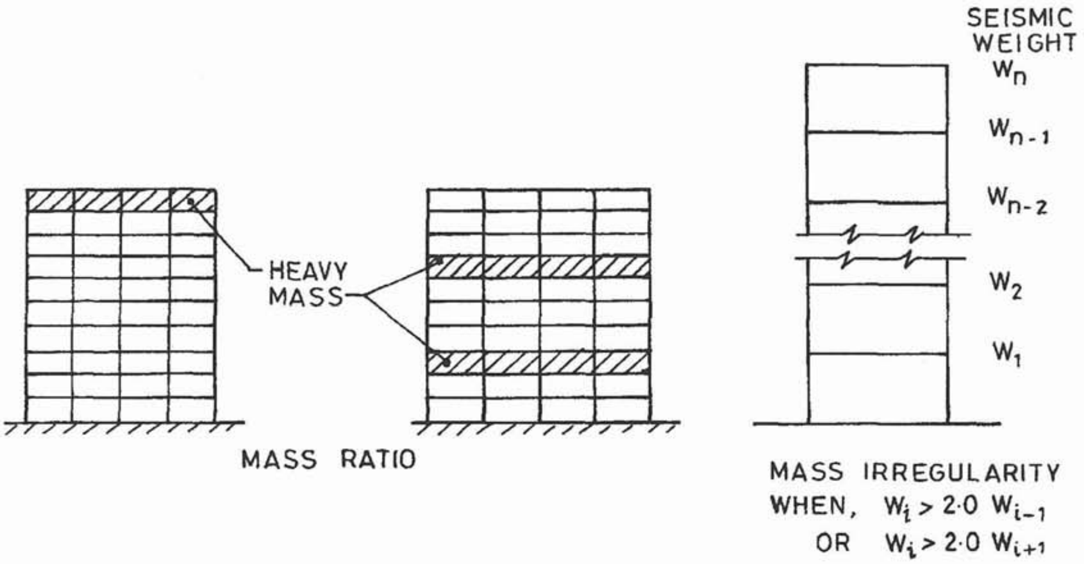


3 E Non-Parallel System

IS 1893 (Part 1) : 2002 FIG. 3 PLAN IRREGULARITIES

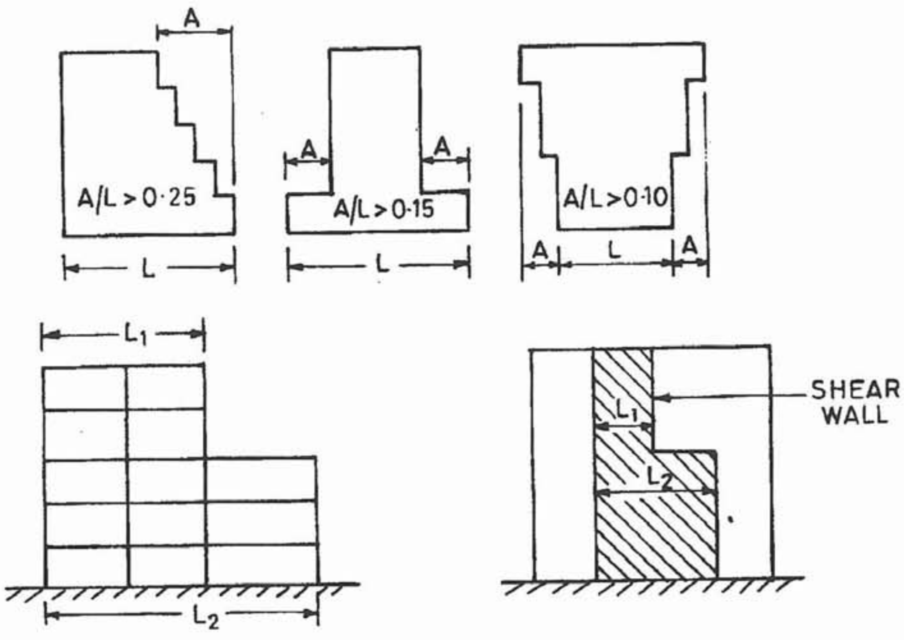


4 A Stiffness Irregularity

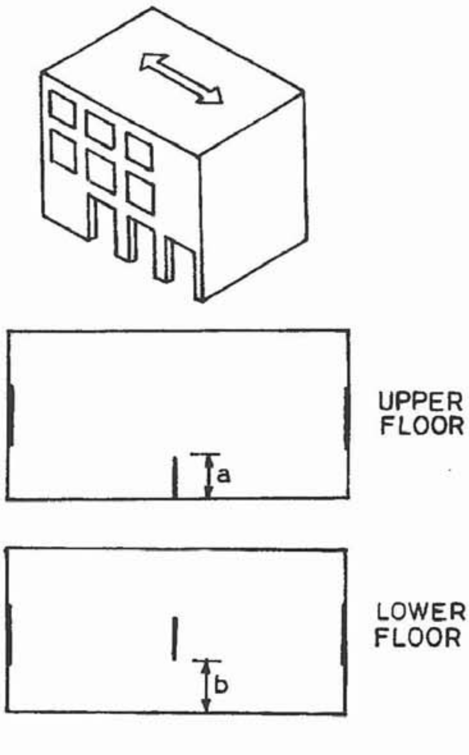


4 B Mass Irregularity

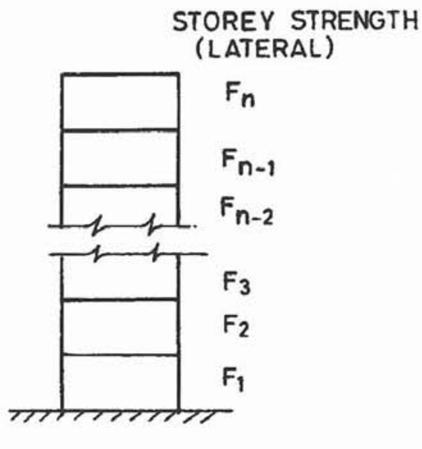
IS 1893 (Part 1) : 2002 FIG. 4 VERTICAL IRREGULARITIES — *Continued*



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 $F_i < 0.8\ F_i + 1$

IS 1893 (Part 1) : 2002 FIG. 4 VERTICAL IRREGULARITIES

$T_a = 0.075h^{0.75}$ for RC frame building

$= 0.085\ h^{0.75}$ for steel frame building

b) The approximate fundamental natural period of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09h}{\sqrt{d}} \quad \text{.......} \quad (5)$$

where

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

h= Height of building, in m.

Note: The height 'h' 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.

7.4.4 Distribution of Earthquake Force along Building Height

The design lateral force V_B shall be distributed to the various floor levels. The overall design seismic force thus obtained at each floor level, will then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

For multistoreyed buildings, the floor level earthquake load (where all masses of floors and half the walls just above and below the floor are assumed to be lumped) are worked out according to inverted-parabola-distribution as:

$$Q_i = V_B \frac{W_i\ h_i^2}{W_1\ h_1^2 + W_2 h_2^2 + ... + W_n h_n^2} \quad \text{.......} \quad (6)$$

where Q_i = lateral earthquake force at floor (i), i = 1,2.,n

W_i = load (DL + IL) lumped at floor (i)

h_i = height measured to level i above the base

n = number of storeys, or floors including roof.

By summing up the floor level forces above any storey, the shear force in that storey is obtained. The distribution of earthquake forces and shears along the height of a 10 storeyed building is shown in *Fig. 8.*

The building frame, which is by nature three dimensional (3D), has to be analysed for the lateral forces as determined above. It should be realised that besides the bending moments and shear forces caused in beams and columns, axial forces (tension and compression) are also caused in them which grow in importance as the building height increases. Hence these effects should also be carefully worked out.

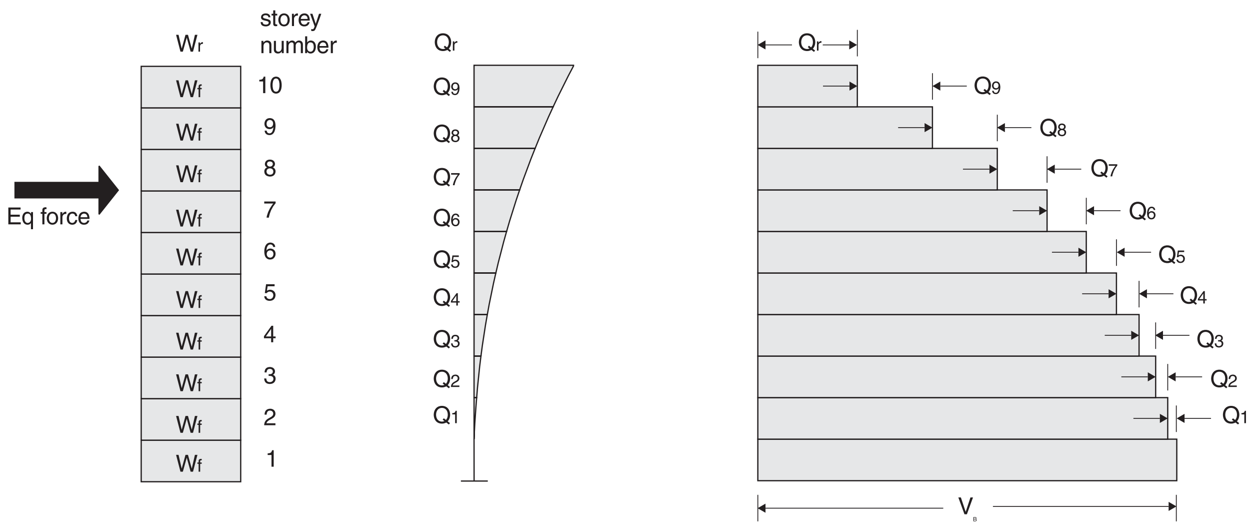


fig 8 a
frame

fig 8 b
distribution of forces

fig 8 c
distribution of shears

Fig. 8
Force and shear distribution for ten-storeyed building
(Source: IS:1893-2002)

7.4.5 Minimum Lateral Force for Design

Dynamic analysis may be performed either by the Time History Method or by the Response Spectrum Method, However, in either method, the so calculated base shear (V_B)shall be compared with a base shear ($\bar{V}_B$) calculated using the empirical fundamental period T_a, as per 7.4.3 above. Where V_B is less than $\bar{V}_B$, all the response quantities (for example member forces, displacements, storeys forces, storeys shears and base reactions) determined in dynamic analysis shall be multiplied by $\bar{V}_B/V_B$.

8 ANALYSIS AND DESIGN OF BUILDINGS

8.1 Lateral Load Analysis

Analysis of building frames for the lateral earthquake loads could be carried out by a number of methods, (a) some approximate methods based on statical equilibrium making the frame statically determinate by a number of assumptions, (b) some more accurate methods using plane frame approximation but considering stiffnesses of the beams and columns, and (c) computer analyses using 2D or 3D idealisations. While the methods (a) and (b) are suitable for quick approximate preliminary design, the methods (c) are the most accurate and should be used for final design and checking. In addition, it is important to realise the importance of the points given in the following Paras.

8.2 Torsion

Buildings as well as their structural frames are subjected to torsion, when the centre of gravity of masses above any storey and the centre of stiffnesses of the elements of the storey do not coincide and there is, thus, an eccentricity between the actuating and the resisting forces *(see Fig. 9)*. This happens even in symmetrical frames due to ‘accidental’ eccentricity, though to a lesser extent. Hence torsional shears and resulting moments in the elements must be analysed, particularly in view of the fact that post-earthquake damage studies have found torsion to be one of the predominant factors contributing to structural damage and collapse. See ***1893-7.9*** for design eccentricity to be used for calculating torsional shears.

8.3 Drift

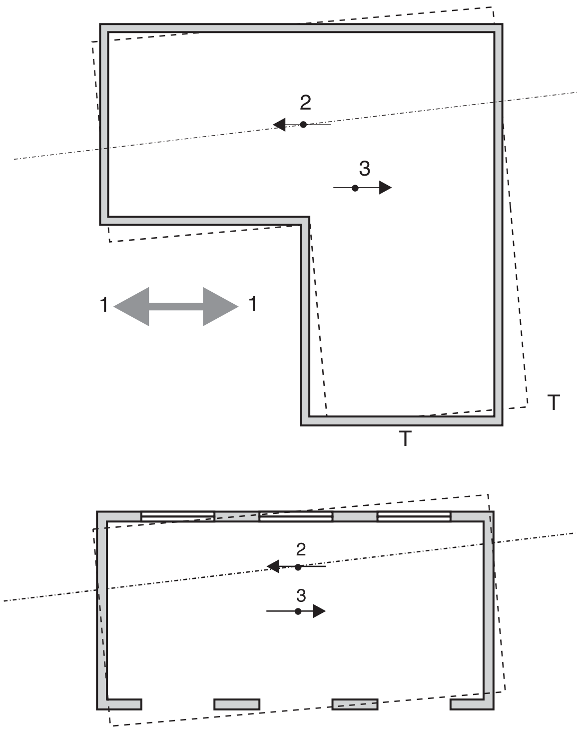
Drift is defined as the ratio of the relative displacement '∇' between two consecutive floors to the vertical height 'h' between them (see *Fig. 10*). Control on drift is required to prevent distortion and damage of windows and the additional moments caused by vertical loads 'P' on the columns becoming eccentric by an amount of ∇. This is termed as P-∇ effect and can lead to buckling of the flexible columns. The limit placed on drift by **1893-7.11.1** is 0.004. For separation to be provided between two adjacent blocks, refer ***1893-7.11.3.***

8.4 Vertical Seismic Acceleration

Usually the horizontal seismic forces as determined earlier are sufficient for designing buildings. In some cases, however, vertical seismic forces also become important and should be considered in design, either alone or in combination with the horizontal seismic forces. For this purpose, the vertical design acceleration acting upward/downward is given by **(1893-6.4.5)**

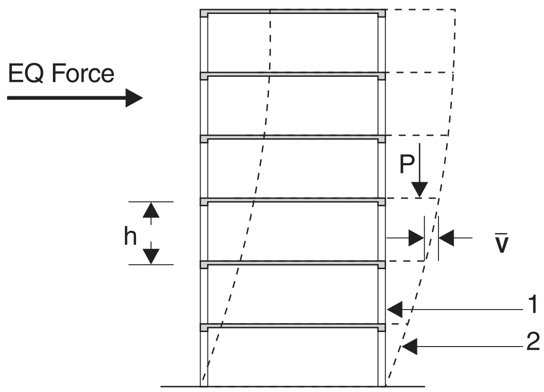
$A_v = 2/3\ A_h$ ....(7)

Consideration of A_v in design is particularly important where stability against overturning is concerned and in the design of horizontal cantilevers and their anchorages. For protection of lives and property from the fall of horizontal projections like balconies, chajjas, large cornices, etc., larger seismic co-efficient than for the main structure are specified **(1893-7.12.2.2),** that is, the vertical force for their design should be based on five times the value of A_v.



Legend
1 Earthquake force
2 Centre of stiffness or resisting force
3 Centre of gravity or the applied inertia forces
4 Twisted building

Fig. 9
Torsion of unsymmetrical plans



Legend
1 Standing Frame
2 Deflected Frame
3 Drift = **∇**/h

Fig. 10
P-∇ ***effect in flexible frames***

8.5 Appendages

Frequently buildings have parapets, water tanks, smoke chimneys and small *barsatis* projecting above the building roof. Due to 'whipping' effect of the earthquake motion, these are subjected to much larger seismic acceleration. Hence these and their supports and connections with the structural frame should be designed for 5 A_h where A_h is used for the building as a whole **(1893-6.4.2)**.

8.6 Buildings with Soft Storey

In case of 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, i.e.

i) 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.

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

The columns and beams of the soft storey are to be designed for 2.5 (but < R) times the storey shears and moments calculated under seismic loads

OR

Besides the columns designed and detailed for the calculated storey shears and moments, shear walls which are to be added symmetrically in both directions of the building as far away from the centre of the building as feasible, are to be designed exclusively for 1.5 (but <R-1) times the lateral storey shear force calculated as before.

Note: R times the calculated values being the maximum elastic values, need not be exceeded

8.7 Secondary Elements

Attention should be paid to the design and detailing of secondary elements of the building, such as the lift wells and hoisting mechanisms, the staircases, the infill wall panels, the internal permanent partitions and the expansion joints (see the provisions in IS:***1893-2002***, IS:***4326-1993***, IS:***456-2000***).

8.8 Partial Safety Factors

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

i. 1.5 (DL+IL)
ii. 1.2 (DL+IL±EL)
iii. 1.5 (DL±EL)
iv. 0.9 (DL±1.5 EL)

where DL = Dead Load, IL= Imposed Load and EL= Earthquake Load effects on the element to be designed.

8.9 Increase in permissible stresses in materials

8.9.1 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.

8.9.2 Increase in allowable pressure in soils

When earthquake forces are included, the allowable bearing pressure in soils shall be increased (see ***Table- 1*** of IS: ***1893-2002)***.

9 SITING OF SETTLEMENTS AND BUILDINGS

9.1 Effect of Site Conditions on Building Damage

Past earthquakes show that site conditions significantly affect the building damage. Earthquake studies have almost invariably shown that the intensity of a shock is directly related to the type of soil layers supporting the building. Structures built on solid rock and firm soil frequently fare better than buildings on soft ground. This was dramatically demonstrated in the 1985 Mexico City earthquake, where the damage on soft soils in Mexico City, at an epicentral distance of 400 km, was substantially higher than at closer locations. In the 1976 Tangshan, China earthquakes, 50% of the buildings on thick soil sites were razed to the ground, while only 12% of the buildings on the rocky land near the mountain areas totally collapsed. Rigid masonry buildings resting on rock may on the contrary show more severe damage than when built on soil during a near earthquake as in Koyna (India) earthquake of 1967 and North Yemen earthquake of 1980. Buildings constructed in old river course in Philippines were destroyed in Bagio earthquake due to liquefaction,

Lessons learnt from recent earthquakes show that the topography of a building site can also have an effect on damage. Buildings built on sites with open and even topography are usually less damaged in an earthquake than buildings on strip-shaped hill ridges, separate high hills, and steep slopes.

9.2 Siting of Settlements

i. Steep sites may have problems of landslides and rock falls and should either be avoided or effectively improved if adopted.
ii. Plain sites with loose fine sands with high water table are liable to liquefaction and subsidence under earthquake intensities VII and higher. These sites **(**see **1893-Table 1: Note 5)** should be avoided, unless improved, for building construction. Such areas should better be reserved for parks, play ground, etc.

9.3 Building Safety

Building safety starts by choosing a safe site. Such a choice is usually not available with many people who are constrained to build on whatever site they are able to get. Unsafe sites such as in **9.2** should be improved as follows for achieving safety of the building:

i. a steeply sloping site may be improved by terracing and constructing breast and retaining walls; and
ii. a site liable to liquefaction or subsidence may be improved by compaction, stabilisation, or sand piling, etc. (see ***1893-Table 1:Note 3***).

It may be mentioned that the improvement methods usually involve large expenses which should be carefully considered before hand.

9.4 Relocation of Site after Disaster

After a severe disaster which has destroyed most parts of a village or township, question sometimes arises whether to relocate the settlement. It is a very difficult decision since, firstly, uprooting the agricultural population may increase their distance from the farms and fields,and secondly, the capital investment needed will be much higher due to the costs of infrastructure facilities, community buildings, etc. Hence such a decision should be taken with utmost care and in consultation with the affected community.

10 FACTORS AFFECTING DAMAGE TO BUILDINGS

Besides the site related factors, the principal factors that influence damage to buildings and other man-made structures are listed below:

10.1 Building Configuration

An important feature is regularity and symmetry in the overall shape of a building. A building shaped like a box, such as rectangular both in plan and elevation, is inherently stronger than one that is L-shaped or U-shaped, such as a building with wings. An irregularly shaped building will twist as it shakes, increasing the damage. Some configuration irregularities in buildings are shown in *Fig. 11.* Such irregularities ***(1893-7.1)*** also increase the complexity in design and the cost of earthquake resistance. Whether they contribute architecturally is open to question.

10.2 Openings in Walls

In general, large window and door openings in walls of a building tend to weaken the walls. Therefore, fewer and smaller the openings, less the damage the building will suffer during an earthquake. If it is necessary to have large openings, special provisions should be made to ensure structural integrity.

10.3 Rigidity Distribution

The rigidity distribution in a building along the vertical direction should be regular, since the changes in the structural rigidity of a building from one floor to the next (see *Fig. 11)* will

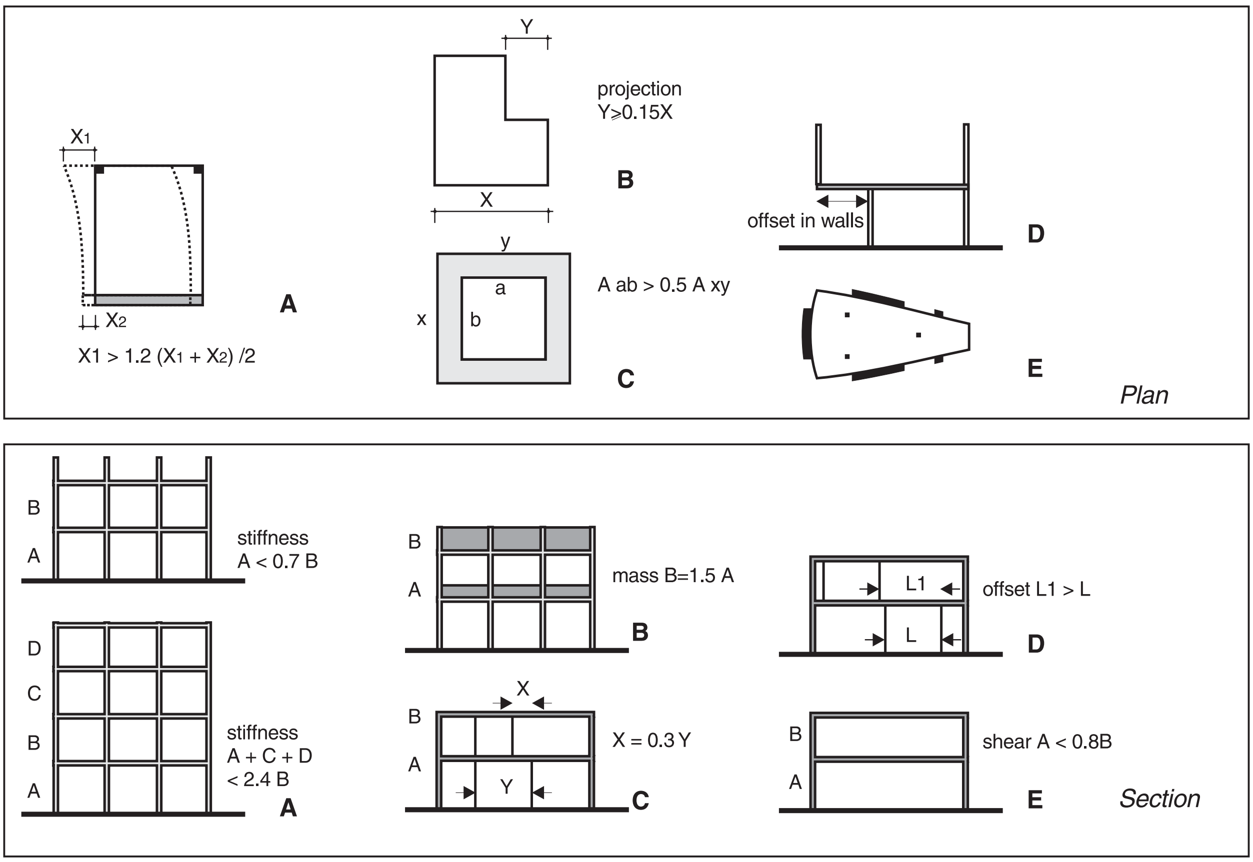


Fig. 11
Configuration irregularities
Source: Uniform Building Code of USA, 1988

increase the potential for damage, and should be avoided. Columns or shear walls should run preferably continuously from foundation to the roof, as they will provide a proper load path from the roof to the foundation. Building with floating columns are considered unsuitabile for seismic regions due to their poor performance as seen in Bhuj earthquake of 2001.

10.4 Ductility

By ductility is meant the ability of the building to bend, sway, and deform by large amounts without serious damage or collapse. The opposite condition in a building is called brittleness arising both from the use of materials that are inherently brittle and from the wrong design of structure using otherwise ductile materials. Brittle materials crack under load; some examples are walls made of adobe, brick and concrete blocks. It is not surprising that most of the damage during the past earthquakes was to unreinforced masonry structures constructed of brittle materials, poorly tied together. The addition of steel reinforcements can add ductility to brittle materials. Masonry and concrete, for example, can be made ductile by proper use of reinforcing steel. To achieve desired degree of ductile deformation, special ductile details in columns, beams and shear walls as well as in masonry buildings are specified in IS: ***13920-1993*** and IS: ***4326-1993.***

10.5 Foundation

Buildings which are structurally strong to withstand earthquakes sometimes fail due to inadequate foundation design. Tilting, cracking and failure of superstructure may result from soil liquefaction and differential settlement of footings.

Certain types of foundations are more susceptible to damage than others. For example, isolated footings of columns are likely to be subjected to differential settlement particularly where the supporting ground consists of different or soft types of soil. Mixed types of foundations within the same building may also lead to damage due to differential settlement.

Very shallow foundations deteriorate because of weathering, particularly when exposed to freezing and thawing in the regions of cold climate or wetting and drying of black cotton (expansive clay) soils.

10.6 Quality of Construction and Maintenance

In many instances, the failure of buildings during the earthquake has been attributed to poor quality of construction, use of substandard materials, poor workmanship, and careless maintenance. Example are: inadequate skill in bonding, absence of “through stones” or bonding units in field stone masonry, use of unsoaked dry bricks while laying in cement mortar, lack of curing of masonry and concrete, that is, improper and inadequate construction.

10.7 Additions to Existing Structures

Additions to existing buildings if not made according to seismic safety norms, may not only be damaged during an earthquake but also become cause of damage to the parent building. Therefore:

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

ii) An addition that is not structurally independent should be designed and constructed such that the *entire* building conforms to the seismic resistance requirements for new building unless the following conditions are complied with:

a. The addition complies with the requirements for new structures.

b. The addition does not increase the seismic force in any structural elements of the existing building by more than 5 percent unless the capacity of the element subject to the increased force is still in compliance with the standard, and

c. The addition does not decrease the seismic resistance of any structural element of the existing building below that required by the design codes.

10.8 Change in Occupancy

When a change of occupancy results in a building structure being re-classified to a *higher importance* factor (I), the building will need rechecking for seismic safety as per the coded stipulations.

11 CATEGORIES OF EARTHQUAKE DAMAGE

An outline of damage categories was described in *Table 3* on the basis of past earthquake experience. Therein the appropriate post-earthquake action for each category of damage was also suggested. This information is found most useful in post-earthquake surveys and estimating the cost of rehabilitation of buildings.

12 ARCHITECTURAL DESIGN FEATURES

There are certain features which if taken into consideration at the stage of architectural planning and design of building, their performance during earthquakes will be appreciably improved. Some of these are in what follows:

12.1 Lightness

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

12.2 Projecting and Suspended parts

i. Projecting parts like large cornices, facia stones, parapets, etc., should be avoided as far as possible, otherwise they should be properly reinforced and firmly tied to the main structure, and their design shall be in accordance with **1893-7.12.2.1, 7.12.2.2.**

ii. Ceiling plaster should preferably be avoided, otherwise it should be as thin as possible and applied with care to ensure good adhesion.

iii. Suspended ceiling should preferably be avoided. Where necessary, it should be light, adequately framed and securely connected.

12.3 Building Configuration

12.3.1 Plan

The building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity, or centres of mass and rigidity of the building should be made to coincide with each other, as shown in *Fig. 12*. If symmetry of the structure is not possible in plan, elevation or mass distribution, provision must be made for torsional and other effects due to earthquake forces in the structural design. Also, structurally, a cellular plan with floor space divided into separate rooms will be more resistant to seismic forces than one large room with mobile partitions. Otherwise the long walls should be supported by RC columns or buttresses (*see Fig. 13)*.

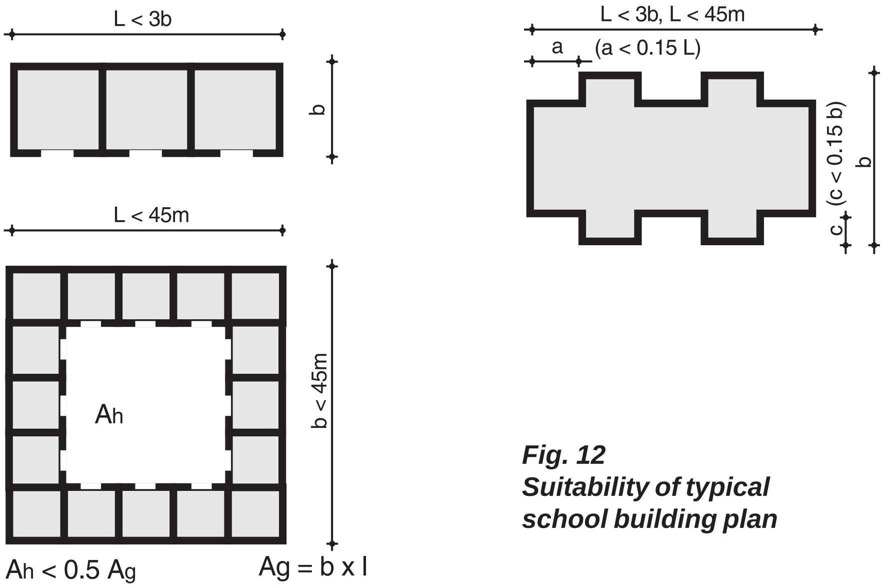


Fig. 12
Suitability of typical school building plan

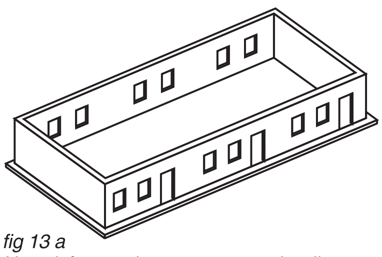


fig 13 a
Unsatisfactory; long unsupported walls

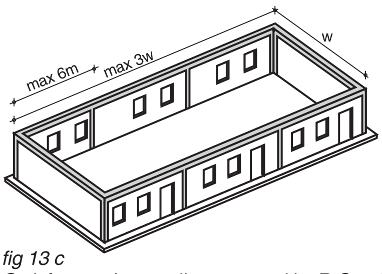


fig 13 c
Satisfactory; long walls supported by R.C. columns

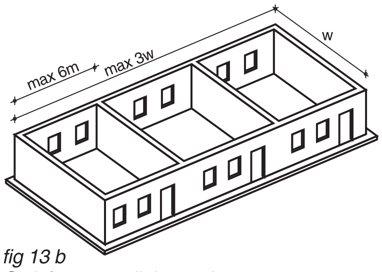


fig 13 b
Satisfactory; cellular enclosures

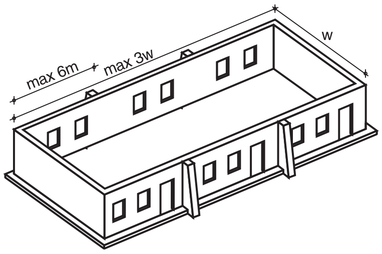


fig 13 d
Satisfactory; long walls supported by buttresses

Fig. 13
Lateral supports to long walls

12.3.2 Optimal Seismic Designs

The structure should be designed to have adequate strength against earthquake effects along *both* the horizontal axes. This can be achieved either by having columns of square sections or by orienting the major axes of the columns or shear walls along the two axes of the building. A few optimal seismic designs are shown in *Fig. 14*. Some special features and their beneficial effects can be mentioned as follows:

Features .... Beneficial Effect

- Low height-to base ratio .... Minimize tendency to overturn
- Equal floor heights .... Equalises column/wall stiffness
- Symmetrical plan shape .... Reduces torsion
- Identical resistance on both axes .... Balanced resistance in all directions
- Uniform section and elevations .... Eliminates stress concentrations

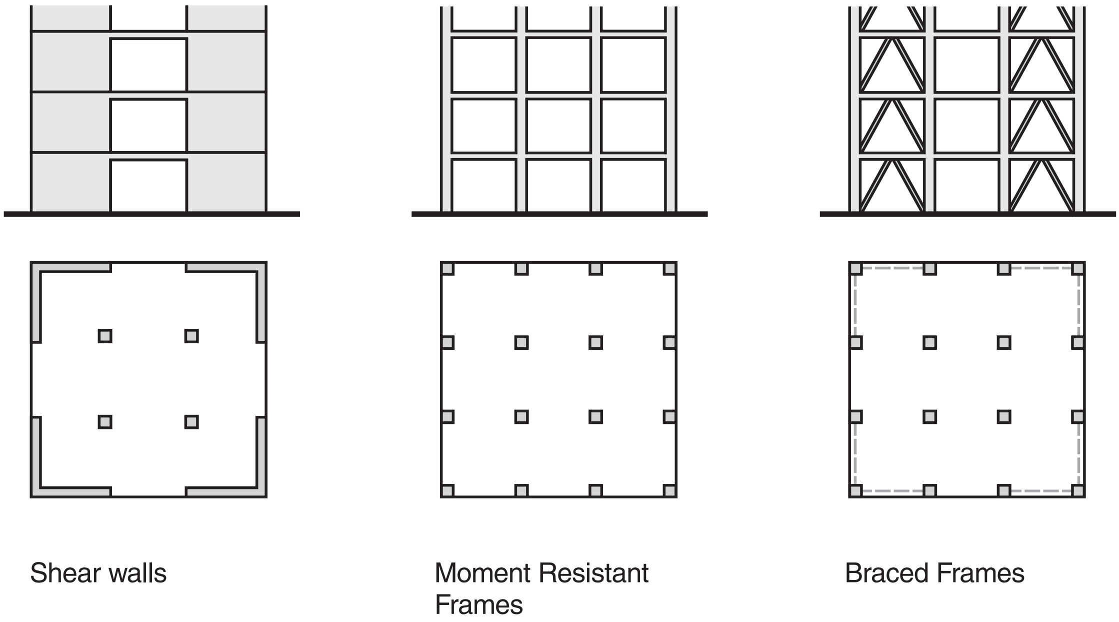


Fig. 14
The optimal seismic design

- Seismic resisting elements at perimeter .... Maximum torsional resistance
- Short spans of beams .... Low unit stress in members
- Redundancy (Rigid connections) .... Toleration of failure of some members
- Ductile Detailing .... Reserve energy to prevent collapse

12.3.3 Separation of Wings

Large buildings having plans with shapes like, L, C, T, E and Y should preferably be separated into rectangular blocks by providing separation sections at appropriate places. *See Fig. 15*. Separation section is 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 hammering due to earthquake. Crumple section is a separation section filled or covered with appropriate material which can crumple or fracture during the earthquake.

12.3.4 In case of framed construction, members should be duplicated on either side of the separation or crumple section. As an expensive and not so good alternative, in certain cases, such duplication may not be provided, but the portions on either side are designed to act as cantilevers.

12.3.5 Where separation is necessary, a complete separation of the parts should 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 should take care of movement owing to temperature changes also. For details, refer **4326-5.1, 5.2.** (for special separation, see **4326-Table 1).**

Note: Single houses normally have small projections causing unsymmetry. These should, preferably be limited as shown in Fig. 15.

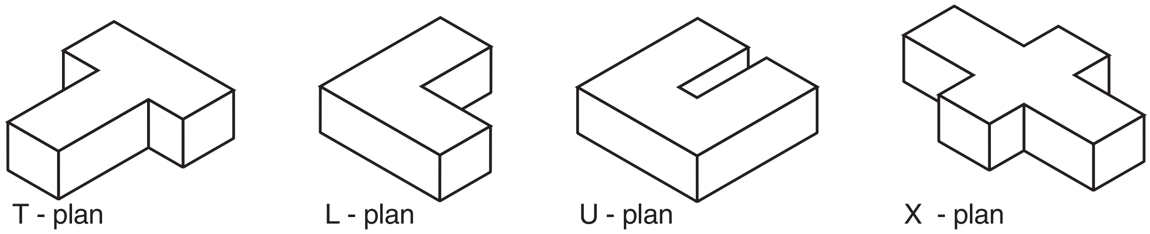


Fig.15a: unsymmetric plans

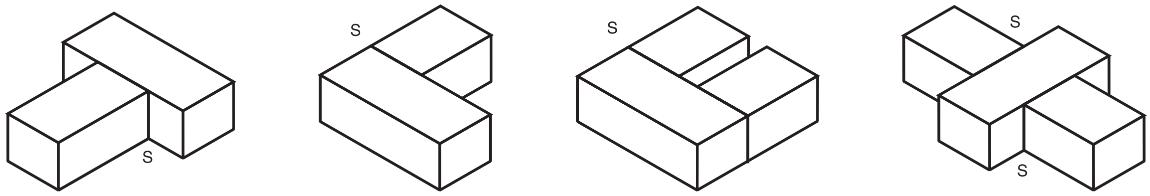


Fig.15b: use of separation section (S)

Fig. 15
Architectural planning of large buildings

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13 DETAILING FOR DUCTILITY

13.1 Need for Ductility in Structural Frames

The currently adopted performance criteria in the earthquake codes is the following:

i. *The structure should resist moderate intensity of earthquake shaking without structural damage.* Such an intensity which could occur a number of times in the life span of the structure, is catered for by the code-based design seismic coefficients, A_h (see **1893-6.4.2**).

ii. *The structure should be able to resist exceptionally large intensity of earthquake shaking without collapse.* Such an intensity could occur not more than once in the life of the structure. It is not catered for by the codal design seismic coefficients but by incorporating details for ductile deformations *(IS: **13920-1993**).*

Providing earthquake resisting capability costs money, which increases geometrically as the design intensity increases if *no-damage design* is adopted as the criterion. Such an approach though feasible is exorbitantly expensive for residential or community buildings. The code has therefore adopted (through a response reduction factor R) lower than maximum probable acceleration coefficients A_h for the seismic zones to take care of the frequent low intensity earthquakes, and insisted on appropriate reinforcing details for achieving adequate ductile deformations beyond yielding (or first crack occurrence in structural members) to take care of large intensity of once-in-life earthquake intensity occurrence. Thus the criterion adopted is *no-collapse design.*

13.2 Nature of Recommendation for Ductility

a. For reinforced concrete moment-resistant frames, ***IS:13920-1993*** explicitly lays down the requirements and detailing of beams, columns, shear walls and their joints, which must be used for earthquake safety.

b. For structural steel frames, Indian Standards Handbook for Structural Engineers SP: 6(6)-1972 gives details for plastics design of steel frames which are also suitable for achieving ductility in large intensity earthquakes.

c. For masonry buildings and structures, the requirements of steel reinforcing at critical points are suggested in various earthquake related IS: Codes (4326, 13827, 13828 of 1993), with implicit consideration for ductility. Therefore, *even where calculations based on code-based seismic coefficients may not indicate tension steel requirements, the steel reinforcement suggested in the form of seismic bands and vertical steel bars at corners and junctions of walls as well as jambs of openings must be provided* since these are based on larger seismic coefficients (about $4A_h$)for probable maximum earthquake, and ductility consideration.

14 DETAILING OF R.C. MOMENT RESISTANT FRAMES

In view of the facts that by far the largest number of taller-than-three-storey buildings are being built in the country and all over the world, moment-resistant R.C. frames or Dual system of frames with shear walls are being used and that many such buildings are being built in non-engineered way by masons, it is considered useful to highlight the crucial detailing aspects here. For fully engineered buildings, reference must of course be made to **IS:456-2000** and **IS:13920-1993**. The ductility detailing is specifically *required* in (a) all structures in Seismic Zones IV and V (b) important structures in Zone III, (c) industrial structures in Zone III and (d) structures of more than 5 storeys in Zone III. However, it will be a good practice to use such detailing in all R.C. frame buildings in all zones and particularly cyclone prone and tsunami prone coastal areas where corrosion resistant precautions should also be taken.

The critical zones in reinforced concrete frames where ductility of sections and confinement of concrete by closely spaced stirrups or spiral is required are shown in *Fig. 16* and explained below:

i) Ends of beams upto a length of about twice the depth of the beam where large negative moments and shears develop are likely locations for plastics hinges. Here shear and moment reversal is also possible under large seismic forces.

ii) Ends of columns where maximum moments develop due to lateral forces. Values of maximum column moments closely approaching plastic moment capacity can be expected and these moments are likely to undergo full reversal. High lateral shears of reversible nature can be developed based on moments of opposite sign at the column ends. The length of such zones is about one-sixth of the clear height of the column between floors.

iii) Joint regions between beams and columns undergo very high reversible local shears, diagonal cracking and local deformation may occur causing significant local rotation at joint.

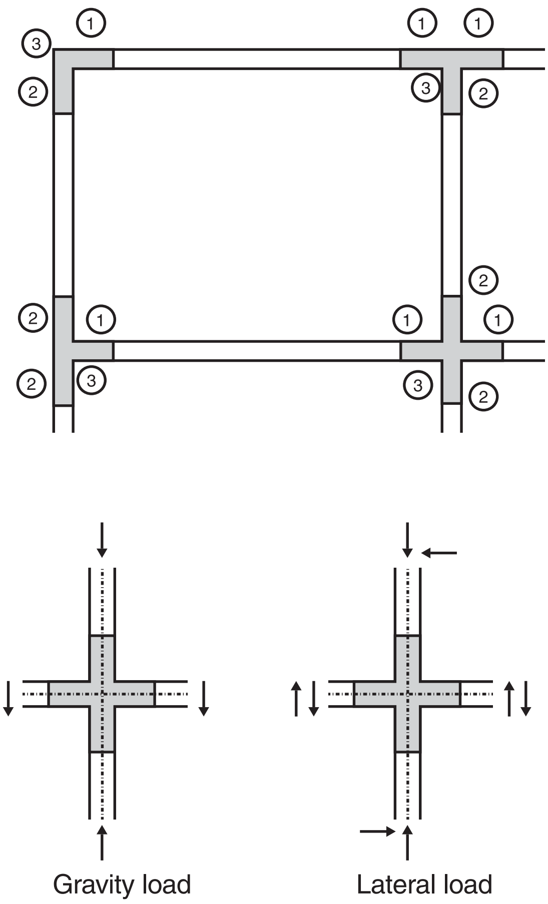


Fig. 16
Critical section in RC frame

14.1 Concrete and Steel Grades

For buildings having more than 3 storeys, the concrete of grade M20 (1:1.5:3) or richer mix and steel reinforcement of grade Fe 415 or TMT (Fe 500) should be used.

14.2 Detailing of Beams

i. Web width b_w should be 200 mm or more, and overall depth not more than 0.25 of clear span.

ii. The tension steel area ratio should not be less than p_{min} nor more than 0.025, where

$$p_{min} = 0.24 \frac{\sqrt{f_{ck}}}{f_y} \qquad (7)$$

For concrete of M20 and steel F_e 415, the steel ratio p_{min} will be .00259.

iii. The beams should have at least two bars satisfying the minimum reinforcement criterion on top as well as bottom face throughout the length of the beam with full bond length anchorage in the end columns, and continuity in the adjacement spans. Other bars coming into the joint should be anchored or made continuous in a similar way. *See Fig. 17.*

iv. Full bond length will mean the length for developing tensile yield L_d *plus* 10 times the bar diameter *minus* two times the bar diameter for each 90^o bend ($L_d + 10\Phi - 2\Phi$ for each 90^o bars).

v. The longitudinal bars should be spliced near the quarter-span points of the beam, only half the bars at one section. The lap length shall be L_d and the splice should be contained in stirrups spaced @ 150 mm.

vi. The transverse stirrups should be designed to ensure the shear capacity of the beam to exceed the flexural load capacity (see **13920-6.3**)

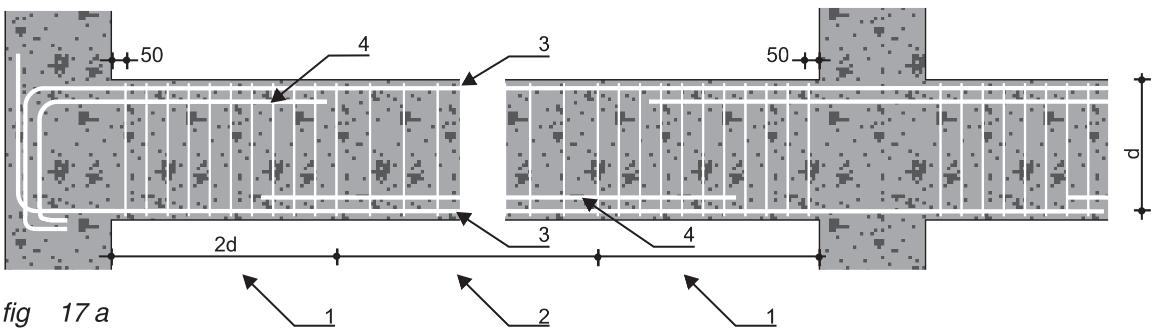


fig 17 a

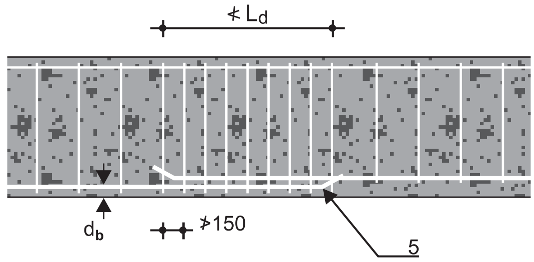
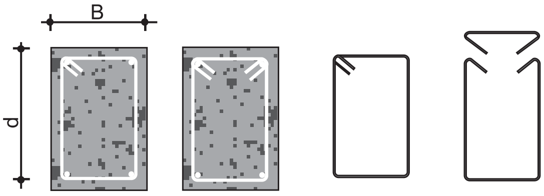


fig 17 b



c *d* *e* *f*

Legend

1 Hoop spacing ≤ d/4
2 Hoop spacing ≤ d/2
3 Minimum 2 bars, $A_s \geq p_{min}$ Bd
4 Maximum $A_s \leq p_{max}$ Bd
5 Splice

Fig. 17
Beam reinforcement

vii. The spacing of stirrups *(Fig. 17)* shall not exceed d/4 or $8d_b$ but not less than 75 mm, in the end 2d length of the beam, elsewhere the spacing not to exceed d/2. d_b is the diameter of main bars in the beam and d the effective depth of the beam.

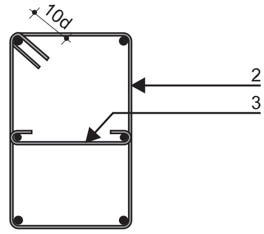


fig 18 a
Closed stirrup

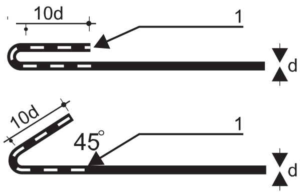


fig 18 b
Special hooks

Legend
1 Special hook
2 Closed stirrup
3 Cross-tie

fig 18
Closed hoop or stirrup

14.3 Detailing of Columns

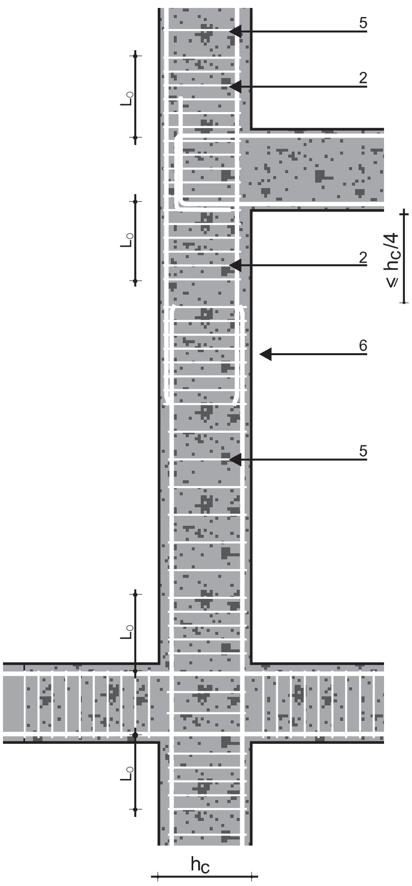
i. Columns shall have a minimum side of 200 mm for beam spans ≤5 m and number of storeys ≤5, otherwise 300 mm and designed as per IS:456-2000 using Design Aids SP:16-1980 for direct and bending stresses combined.

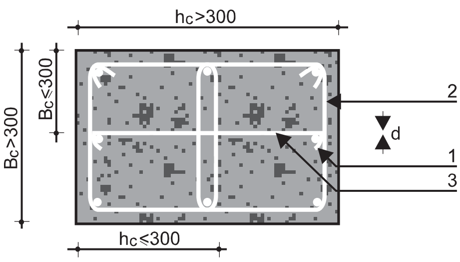
ii. Transverse ties shall be in the form of closed hoops (*see Fig. 18).*

iii. Special confining hoop reinforcement shall be computed **(13920-7.4.7)** and provided near ends in a length equal to 450 mm, or one-sixth of clear height of column, or the longer side of the rectangular column, (or the diameter of circular column), whichever is greater *(Fig. 19)*. The spacing of these hoops shall not exceed 0.25 of the minimum width of the column but not less than 75 mm and not more than 100 mm.

iv. The transverse steel requirement shall also be worked out for the shear caused by lateral loads and that which could possibly be caused in the column by the moment capacities as its ends **(13920-7.3.4)** whichever larger.

v. The longitudinal steel bars should be spliced within the middle 2/3 height of the column, the

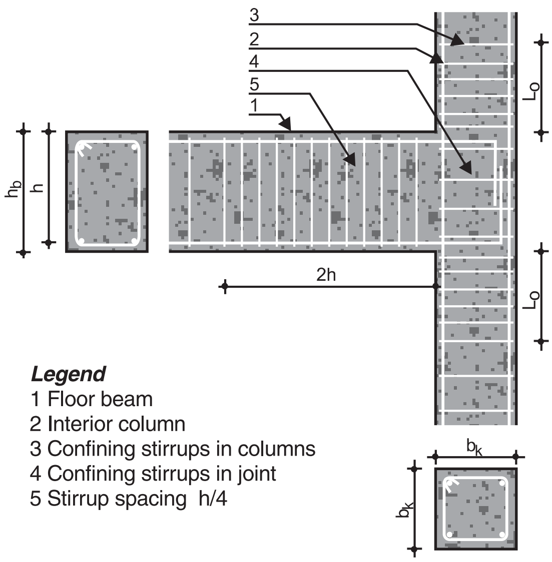




Legend
1 Special hook
2 Closed stirrup
3 Cross-tie
4 Special confining reinforcement in joints
5 Usual stirrups
6 Splice in column bars

Fig. 19
Reinforcement detailing in RC column

and splice enclosed within closed hoops of 6 mm Φ @ 150 mm apart *(Fig. 19).*



Legend
1 Floor beam
2 Interior column
3 Confining stirrups in columns
4 Confining stirrups in joint
5 Stirrup spacing h/4

Fig. 20
Detailing of column beam joint

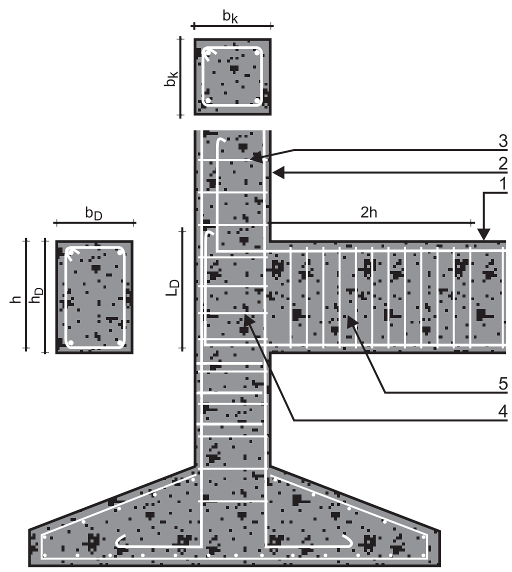
14.4 Detailing of Beam-Column Connections

The concrete forming the common area of beams and columns in the joint gets subjected to complex stresses due to bending, compression, tension and shearing. In order to avoid jumbling of bars from all sides, it will be appropriate to do the following *(See Fig. 20).*

i. Pass the column reinforcement 'through' without splicing within the joints.
ii. Pass the beam reinforcement 'through' without anchoring in the joint except in the end columns; (columns wider than the beams will facilitate this detail).
iii. Provide confining hoops, like that in the lower column, in the joint also which will take care of the diagonal tensions as well.

14.5 Detailing of Foundation, Plinth Beam, Column Joint

i. Individual column footings resting on soft to medium soils or piles, and pile groups, are to be connected together at ground or plinth level by struts/ties along both axes **(4326-4.3.4).** The strut/tie shall be a minimum of 200 x 200 mm in section with 4 -10 mm dia H.S.D. bars longitudinally and 6Φ stirrups @ 150 mm apart.
ii. Footings can be relieved much from bottom-end moments of the columns, if these struts are designed to resist the column moments. The reinforcement detailing as shown in *Fig. 21* may be followed for the case with the strut-beams.



Legend
1 Floor beam
2 Interior column
3 Confining stirrups in columns
4 Confining stirrups in joint
5 Stirrup spacing h/4

Fig. 21
Detailing of joint of column with plinth beam and footing

15 DUCTILE DETAILING OF STEEL FRAMES

The critical zones requiring special consideration are the same as shown in *Fig. 16*. In the case of steel frames, the details needing particular care are local and lateral buckling. If buckling takes place elastically, there will be little ductility. Hence such details of sizes etc. are adopted that buckling should occur only after enough plastic strains have occurred.

15.1 Local Buckling

The full plastic moment capacity and redistribution of moment in frames can be attained only if the members can undergo large strains without the local buckling of its plate elements even after yielding. The maximum width to

Table 4:
Maximum Width/Thickness Ratio of Plate Elements

Type of element	Maximum b/t ratio		Measurement of width b of plate element
	Any fy	fy = 250 MPa	
a. Unstiffened outstand, e.g. projecting out plates, or angles, stiffeners, flanges of T,I,Z sections	$\frac{136}{\sqrt{fy}}$	8.5	Plates : Distance from free edge to first line of rivets. Stems of L,T,Z : Nominal width of Flanges : Half the nominal width of I or T flange
b. Unsupported width	$\frac{512}{\sqrt{fy}}$	32	Distance between parallel lines of rivets
c. Web under bending & compression			
i. P/Py > 0.27	$\frac{688}{\sqrt{fy}}$	43.5	Clear depth of web between flanges
ii. P/Py - 0.27	$\frac{1120}{\sqrt{fy}} - \frac{1600}{\sqrt{fy}} \cdot \frac{P}{P_y}$	$70 - 100 \frac{P}{P_y}$	----- do -----
d. Web in shear	$\frac{1120}{\sqrt{fy}}$	70	Clear depth of web between flanges

Notes :

1. *The restricted ratios given here above must be ensured in plastic-hinge zones. In other lengths remaining elastic, the ratios as in IS:800 may be adopted.*
2. *For webs in shear, that is, case (d,) if b/t exceeds 688/ √fy (i.e. 43.5 for fy = 250), P/Py on the section will be restricted to 0.7-0.01 b/t of web.*

thickness ratio may be adopted from this point of view as shown in Table 4.

Generally, all I-Sections satisfy the above width-thickness ratio requirements. But if a section shows inadequacy, either it should be replaced by another heavier section, or the weak plate element may be stiffened. For the flanges, the stiffening may consist of either cover plates or plates welded at their edges transversely so as to make a sort of lip. Alternatively, plates as deep as the I-Section may be welded to the flanges on both sides at the critical sections so as to make a box section. This has the further advantage of increasing the lateral buckling strength of the beam or the column. The web plate may be stiffened by means of longitudinal stiffeners at about 1/2 the depth of section from the compression flange.

15.2 Lateral Buckling of Beams

Due to yielding of beam section, the lateral stability of the beam is markedly reduced. Hence at the point where plastic hinges are expected, lateral bracing must be provided excepting that it may be omitted at that point where the plastic hinge develops in the very last. Between the two braced hinge locations, the rules for the elastic stability may be used (see **IS:800-2006**).

The bracing members themselves should be strong and stiff so that lateral buckling at the plastic hinge is restrained without much sideways movement. The magnitude of the force in such members is found to be rather small, of the order of about 1% of yield load of the main member. From the point of view of stiffness, however, the cross sectional area of the bracing member may be designed to carry at its safe working stresses, a tensile or compressive force equal to 2% of fy.A where A is the area of the member braced.

15.3 Design of Columns for Ductility

The redistribution of moments in the plastics theory assumes that the mechanism failure of a frame will not be preceded by elastic or inelastic buckling of columns. **SP 6(6)-1972** recommends the following rules for design of columns in frameworks:

1). The slenderness ratio L/r_x shall not exceed 120, where r_x is the radius of gyration in the plane of the frame.

2). The maximum axial load P on any column shall not exceed 0.6 P_y nor a value given by

$$P = \frac{8700\ P_y}{(l/r_y)^2} \qquad \text{(8)}$$

where l is the unbraced length of the column and r_y its radius of gyration perpendicular to the plane of the frame.

3). Columns in continuous frames where sidesway is not prevented should be so proportioned that

$$\frac{2P}{P_y} + \frac{L}{70\ r_x} \leq 1.0 \qquad \text{(9)}$$

or

$$\frac{P}{P_y} \leq 0.5 - \frac{L}{140 r_x} \qquad \text{(10)}$$

where L is the distance centre to centre of adjacent members connected with the column.

Sidesway may be assumed to be prevented if the frame under consideration is attached to an adjacent structure of sufficient rigidity against sway or if the floor slabs of the frames are securely held to walls, termed commonly as shear walls, which are parallel to the frame. Alternatively, diagonal bracing system may be used in the plane of the frame. Otherwise, the frame is to be considered to undergo sidesway and the above restriction on P_y has to be taken into account.

4). The moment M'p acting together with P as limited above shall not exceed Mp nor values given in Tables 5 and 6.

The above specifications cover practically all columns with various end conditions. A column of any end conditions if designed as case III column will always be on the conservative side. Hence in case of doubt, any column may be designed as a case III column.

Table 5:
Design of Columns

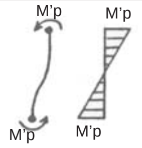
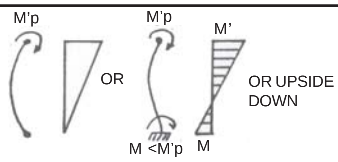
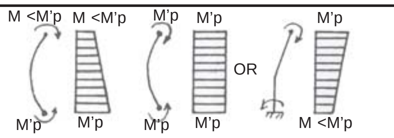
Case	End Condition of Columns	Max. Value of (M'p/Mp)	Remarks
I		1.18 -1.18 . P/P_y	If $l/r_x < 60$ and $P/P_y <$ 0.15 use M'p = Mp in case I & II
II		B-GP/P_y *(where B&G given in Table 6)*	
III		$1.0 - K \cdot P/P_y - J \cdot (P/P_y)^2$ *(where K & J given in Table 6)*	

Table 6:
Coefficients for Column Design

L/r_x	Case II		Case III		L/r_x	Case II		Case III	
	B	G	K	J		B	G	K	J
20	1.142	1.182	0.703	0.461	70	1.182	1.406	1.560	-0.429
25	1.145	1.194	0.774	0.389	75	1.188	1.460	1.675	-0.562
30	1.148	1.205	0.847	0.316	80	1.194	1.524	1.799	-0.707
35	1.151	1.217	0.922	0.242	85	1.200	1.600	1.930	-0.866
40	1.155	1.231	1.000	0.165	90	1.206	1.688	2.071	-1.041
45	1.159	1.247	1.081	0.0832	95	1.213	1.788	2.222	-1.231
50	1.163	1.267	1.166	0.0036	100	1.220	1.903	2.584	-1.440
55	1.167	1.292	1.256	-0.0970	105	1.227	2.033	2.556	-1.668
60	1.172	1.323	1.351	-0.198	110	1.234	2.179	2.741	-1.916
65	1.177	1.360	1.452	-0.309	115	1.242	2.343	2.937	-2.185
70	1.182	1.406	1.560	-0.429	120	1.250	2.525	3.147	-3.478

15.4 Design of Connections for Ductility

Connections are the vulnerable sections in structures because usually these are the points of maximum moments where plastic hinges can occur. Requirements of a good connection are:

1). It should be able to carry the full plastic moment of the member connected. At a corner where two members meet, it may be designed for Mp of the weaker member.

2). It should have sufficient yield capacity so that complete redistribution of moments could take place in the structure to transform it into a mechanism. The connecting welds should not be required to yield for this purpose, the yielding should rather be arranged in plates.

3). It should be economical, that is, connection material required should be minimum and fabrication should be simple.

Connections of four types - corner, side, top and interior - as shown in *Fig. 22* will be considered here.

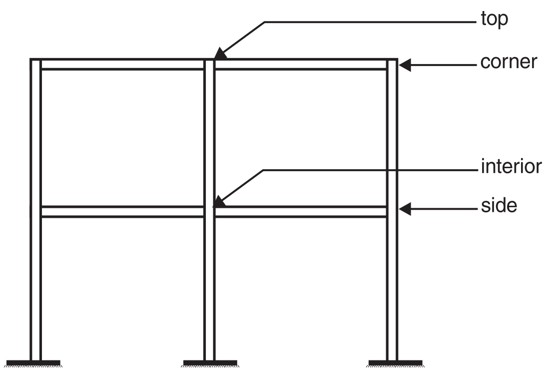


Fig. 22
Typical connections

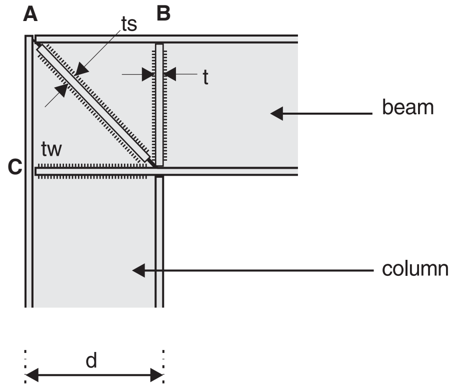


Fig. 23
Corner connections

a. *Corner Connection*

A detail of the corner connection is shown in *Fig. 23*. The stiffener plates AB and AC (flange of beam itself in *Fig. 23*) are essential parts of the connection. If Z is the elastic modulus of the member whose plastic moment governs the design and d its depth (say column in *Fig. 23*), the minimum thickness of the web plate t in the connections which would be safe against shear produced by flange force is given by

$$t = \sqrt{3}\ Z/d^2 \quad \text{......} \quad (11)$$

If the web is thinner, that is $t_w < t$, either web doublers, that is, additional plate to make up the required thickness, are to be used, or diagonal stiffeners shown in *Fig. 23* must be used on one or both sides of the web. The thickness of this stiffener may be found from

$$t_s = \frac{\sqrt{2}}{b_s} \cdot \left(\frac{z}{d} - \frac{t_w d}{2}\right) \quad \text{......} \quad (12)$$

where b_s is the total width of the diagonal stiffeners.

b. *Interior Connection*

At a connection between column and beam, either the column goes through the joint and the beam is discontinuous or vice-versa. Usually the column is kept continuous and the beams transmit their flange forces to the flanges of the column as concentrated forces under which local out-of-plane bending of the column flanges and crippling of the column web may take place. To check against such damage and overstress, either the web *doubler*s are used as shown at *Fig. 24(c)* or web stiffeners in line with beam flanges are provided, which is more common, as shown in *Fig. 24(b)*. According to IS:800-2006, the minimum web thickness of the column required without the stiffeners against a compression flange is given by

$$t = \frac{A_t}{T_b + 5h_2} \quad \text{......} \quad (13)$$

where A_t = area and T_b = thickness of beam flange and h_2 = distance from outer face of column flange to toe of root fillet on the web of the column. Against a tension flange, minimum thickness of column flange is required as

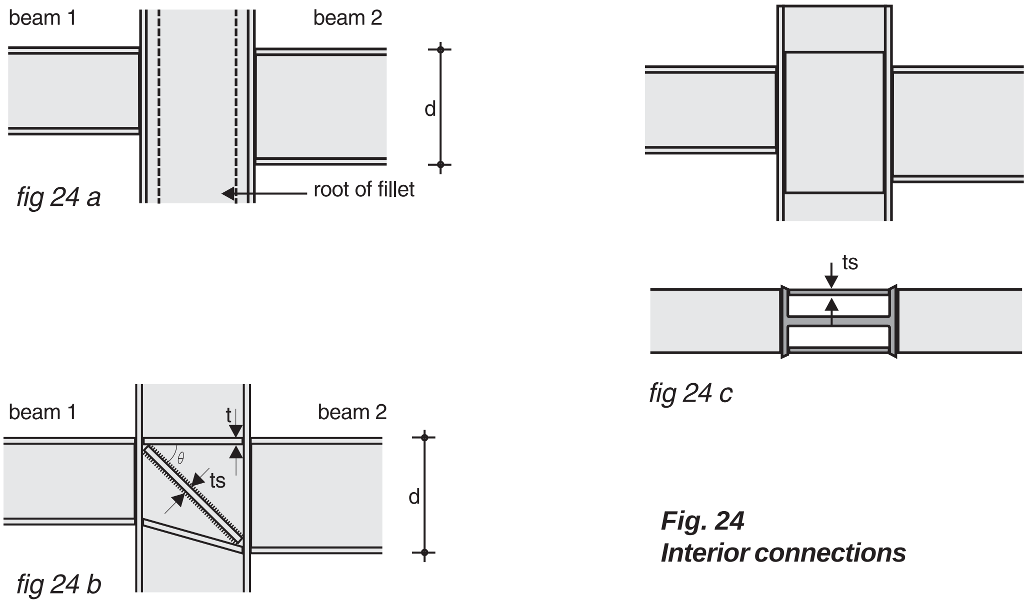


Fig. 24
Interior connections

$$t_t = 0.4\sqrt{A_t} \quad \textit{(14)}$$

If either of the two requirements fails, the horizontal stiffener plate will be needed. Its area shall be calculated by

$$A_{st} = A_t - t(T_b + 5h_2) \quad \textit{(15)}$$

The thickness of the stiffener should be chosen from local buckling considerations, that is, width/5 or more. The ends of the stiffener plates may be milled for full bearing or welded by butt or double fillet welds of equal strength.

Diagonal stiffeners may sometimes be necessary when the unbalanced moment at the connection produces excessive shear stress due to unbalanced flange forces as in the case of corner connection. In an interior connection, minimum safe thickness t of web plate in mm without diagonal stiffeners may be found from

$$t = \frac{0.0071\ M}{A_p} \quad \textit{(16)}$$

where M is the unbalance moment in Nmm from beams 1 and 2, A_p is the area of the connection in mm^2 given by the product of the depth of larger beam with the depth of the column section. If the thickness of the panel t_w is less than t, diagonal stiffeners will be necessary. Their thickness t_s may be found from the following expression

$$t_s = \frac{M(1-t_w/t)}{fy\ b_s\ d\ \cos\theta} \quad \textit{(17)}$$

where M and t are as defined in Eq. 17 and fy is the yield stress = 250 MPa.

c. *Side Connection*

In a side connection, the lighter beam vanishes. Therefore the full moment of the connected beam becomes unbalanced. Thus all expressions of the interior connections are applicable to the side connections with this modification.

d. *Top Connection*

In a top connection, the top column vanishes. This does not affect the expressions for interior connections. Therefore, they will be applicable as such.

Riveted connections may be designed to resist moments and shears for the ultimate loads. For ultimate load analysis the calculated load capacity of welds, bolts and rivets shall be taken as 1.7 times the values calculated using the permissible working stresses.

16 CATEGORIES OF NON-ENGINEERED BUILDINGS

For categorising the buildings with the purpose of achieving seismic resistance at economical cost, three parameters turn out to be significant:

i. Seismic intensity zone where the building is located;
ii. Functional importance of the building;
iii. The stiffness of the foundation soil.

A combination of these parameters will determine the extent of appropriate seismic strengthening of the building.

16.1 Seismic Zones

As stated in **6.3** earlier, the macro level seismic zones in India, are defined on the basis of Seismic intensity scales. From this view point, the risk of damage in the four risk zones may be taken as follows:

- Zone V — Risk of Widespread Collapse and Destruction (MSK IX or greater);
- Zone IV — Risk of Partial Collapse and Heavy Damage (MSK VIII likely)
- Zone III — Risk of Moderate Damage (MSK VII likely)
- Zone II — Risk of Minor Damage (MSK VI or smaller intensity likely).

The extent of special earthquake strengthening should be greatest in Zone V and, for reasons of economy, can be decreased in Zones IV and III with relatively little special strengthening in Zone II. However, since the principles stated in **9** and **10** are good principles for buildings in general (not just for earthquake), they should always be followed even in Zone II and the cyclone prone coastal areas.

16.2 Importance of Building

The important buildings and structures as defined in **7.2** should have higher strengthening provisions, sustain less damage than ordinary ones during the earthquake.

16.3 Strength and Stiffness of Soil

Three soils types are considered here (See **1893 - Table 1**)

i. *Type I Rock of Hard Soils* - Well graded gravels and sand gravel mixtures with or without

Table 7:
Building Categories for Earthquake Resisting Features
In Masonry and Earthen Buildigs

Importance Factor	Seismic Zone			
	II	III	IV	V
1.0	B	C	D	E
1.5	C	D	E	E^+
Note:- Category A is now defunct as zone I does not exist any more.				

clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC) having standard penetration N value above 30.

ii. *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) with N > 15.

iii. *Type III Soft Soils* - All soils other than SP with N< 10.

In the absence of soil investigation, for general purposes of construction of buildings upto 3 storeys, the following definitions may alternatively be used.

Hard - those having safe bearing pressuring P_a of $30t/m^2$ or more.
Medium - those having P_a between 10 and 30 t/m^2
Soft - those having P_a less than 10 t/m^2.

16.4 Combination of Parameters

For defining the categories of buildings for seismic strengthening purposes, four categories are defined in Table 7 in which the Seismic Zones and Importance factor are used as the basis of classification since it combines the two critical factors. The masonry buildings upto 4 storeys will be quite stiff with the fundamental time period less than 0.4 sec, hence (as per ***1893-fig.2 -Response spectra***), they will all have the same amplified value of Sa /g. Therefore the soil type is not considered in Table 7 for defining building categories.

Now category E will require maximum strengthening and category B the least inputs. The general planning and designing principals are, however, equally applicable to all of them.

17 SAFETY OF NON-ENGINEERED BUILDINGS

As per the 2001 Census of India, the country has preponderance of non-engineered housing stock in all seismic zones. Out of the total 249.1 million dwelling units, the buildings with various wall types account for the numbers as given in Table 8.

It is seen that those worst affected in earthquakes (earthen and stone houses) account for 39.9% and the other brittle ones (burned brick) are 44.9%, that is, a total of 84.8% are vulnerable, if shaken by an earthquake of moderate to severe intensity. The collapse of masonry and adobe dwellings gets initiated in Intensity VII and large scale destruction take place in intensity VIII and IX as demonstrated in Uttarkashi Earthquake of 1991, Killari earthquake of 1993, Jabalpur earthquake of 1997, Chamoli earthquake of 1999, Bhuj earthquake of 2001 and J & K earthquake in 2005 resulting in huge loss of lives. For safety of human lives and property, it is therefore essential that all such new buildings are built with earthquake resisting features in the first instance and those existing now in Zone V, IV and III without such features, should be retrofitted so as to achieve at least the minimum

Table 8:
Various Building Types by Wall Materials in India
(Census of Housing 2001)

Building Type	Number of Units	Percent of Total Housing Units
Earthen Walls (mud, unburnt brick/block)	73.8 million	29.6%
Stone walls	25.5 million	10.3%
Burned Brick walls	111.9 million	44.9%
Concrete walls	6.5 million	2.6%
Wood & Ekra walls	3.2 million	1.2%
GI and other metal sheets, Bamboo, thatch, leaves, etc.	28.2 million	11.4%
Total Housing Units	**249.1 million**	**100%**

safety in future probable earthquake occurrences.

For a quick reference, damaging earthquake effects and the corresponding resisting features in respect of buildings are summarised in tabular form in three parts: Table 9 – Soils and Foundations; Table 10 – Building walls; and Table 11 – Building roofs and floors.

Table 9:
Soils and Foundations, Earthquake Effects and Resisting Features

S.No.	Type of soil	Damaging Effect of Earthquake	Earthquake Resisting Feature
1.	Type I Hard	None	Use any foundation type.
2.	Type II Medium	• Not much in Zones II and III • Relative lateral movement possible- in Zones IV, V	• Use any foundation type. • Use tie beams[2] in case of individual column foundations.
3.	Type III soft a. Low water table	• Not much in Zone II • Relative lateral movement possible in Zone III to V	Use any foundation type. Use Plinth Band[3]. • Use tie beams[2] to connect individual column foundations or combined column footings or provide rafts or piles as needed for the loads.
	b. Liquefiable with high water table	• Some relative movement in Zone II • Relative lateral and vertical movements possible in Zone III • Liquefaction resulting in tilting/ overturning of buildings and structures likely in Zones IV & V	• Use plinth beams/tie beams to connect individual column foundations[2]. • Use piles going to stable soil layer or minimum 10m length. Driven piles preferable. OR Improve the soil to a depth of 7 to 8 m or upto stable layer if met earlier, by dynamic compaction or by compaction piles.
	c. Black cotton soil	Soil not seen to be affected in Intensity VIII shaking in Latur or Jabalpur earthquakes, but effect of ground motion amplified on the buildings	Use tie beams to connect individual column footings[2]. Use plinth band in case of strip foundations[3]. Use of under-ream piles preferable.

1. *Refer to IS:1893-2002 Table 1 for soil types, Zone factor Z and Table 6 for importance factor.*
2. *See details in para 14.5*
3. *See details in para 18.5*

Table 10:
Building Walls, Earthquake Effects and Resisting Features

S.No.	Type of Wall	Damaging Effect of Earthquake	Earthquake Resisting Feature
1.	All masonry walls (with flat roofs)	a. Shattering of masonry in heap of materials b. Cracking and separation of walls at corners and junctions of walls c. Diagonal cracking in piers between windows and doors d. Vertical bending cracks near top of walls e. Horizontal cracks near base of storeys	• Use of good quality building units, cement sand or cement-lime-sand mortar, good quality of construction. • Use of lintel band, in all internal and external walls with continuity in the reinforcement. • Control on size and location of openings and use of reinforcing bars at jambs. • Use of ceiling level band (like lintel band) below the roof or floor. • Use of vertical reinforcing bars at corners and junctions of walls.
2.	All masonry walls (with pitched roofs)	Besides a) to e) above, f. Cracking and falling of gable walls.	• Use bands at eave level in all walls and on top of all gable walls integrated with the eave bands.
3.	Random rubble masonry walls	Besides a) to f) as above, g. Delamination of inner and outer wythes of the wall, bulging and falling of wythes	• Use of walls not thicker than 450 mm with provision of 'through' stones or bonding elements in the walls and at the corners & T-joints. (use of good cement mortar reduces chances of delamination).
4.	Wood stud wall	h. Deforms and collapses	• Use 'sill' held down to footings by bolts, diagonal braces in vertical plane of walls and in the horizontal plane at top of wall connecting to perpendicular wall.
5.	Wood frame with brick nogging	i. Brick nogging falls out of wall j. Deformation of wood frame	• Use hold-fasts to hold the nogging. • Use diagonal braces in vertical and horizontal planes as for wood-stud wall construction.
6.	Clay/Adobe/ Unburnt brick walls	k. Shattering of walls in heap of material. l. Cracking and separation at corners. m. Wide cracking and instability of piers between openings n. Overturning of walls	• Use better quality clay to make adobe and clay mortar, also control on wall length and height • Use made of bamboo or timber lintel band in all walls with continuity between perpendicular walls • Control on size and location of openings. • Use of pilasters in long walls and at corners and T-junctions of walls

Table 11:
Building Roofs and Floors, Earthquake Effects and Resisting Features

S.No.	Type of Roof/Floor	Damaging Effect of Earthquake	Earthquake Resisting Feature
1.	Raftered Roof	a. Rafters are displaced and fall down, damage walls by pulling and pushing	• Use full trusses or A-frame arrangement by connecting rafters in pairs through ties.
2.	Trussed Roof	b. Anchors broken, gables pushed out and damaged, trusses shift and fall down. c. Failures of truss joints in wooden trusses d. Long walls are toppled since without lateral support at top.	• Provide X-bracings in planes of rafters in about every 4th bay, and in horizontal plane of main ties in similar bays. • Provide adequate iron straps on wood truss joints. • Provide eave level trussed bracing in long rooms in addition to eave level band to provide diaphragm action.
3.	Sloping roof using RC Prefab elements	e. The prefab elements are disturbed, separated and may fall down.	• Connect the elements together and hold them to peripheral R.C. bands.
4.	Beams/Joists supporting stone pattis, or prefab PC or RC elements	f. The prefab elements and the beams/joists are disturbed, the elements may fall down, beams fall down if bearing length is small.	• Keep the bearing length of beams at least 200 mm, integrate the roofing elements by a RC screed, the whole roof/floor bounded by RC band.
5.	Jack arches resting on Steel girders	g. The arches are cracked longitudinally, tend to shift the girders horizontally and may fall down. h. Girders may fall down by slipping longitudinally	• Weld lateral ties with all steel girders and embed into RC band provided all round. • Weld diagonal braces to steel girders to convert the roof/floor into a horizontal grillage. • Encase ends of steel girders into all-round RC band.
6.	Tiled roofs	Tiles are disturbed, broken and fall down	• Use tiles with lug having holes and tie them to purlins by binding wire. • Stabilish the roof by providing R.C. bands one at the ridge and another at edge of the slope.

Naturally the damaging effect of earthquake will be expected to be more severe in higher intensity zones and less in lower intensity zones. Correspondingly, the earthquake resisting features will be needed to a larger extend in higher intensity than lower intensity zones. In some lower intensity zones, some of the resisting feature listed in the tables can be excluded entirely and some others may be reduced in dimensions so as to affect economy without sacrificing safety. The details to be adopted for various building categories given in Table 7 are given in subsequent paras.

18 BUILDINGS OF GOOD MASONRY STRENGTH

18.1 Building Types

The details of earthquake resistant design and construction given below are applicable to those buildings using fired bricks and other rectangular masonry units such as solid blocks of mortar, concrete or stabilised soil or hollow blocks of mortar or concrete, having adequate compressive strength. The compressive strength of the masonry units on the gross area should be a minimum of 3.5 N/mm^2 for ordinary one to 1½ storeyed buildings and 5.0 N/mm^2 or more for important and/or taller buildings. The required strength will depend on number of storeys and wall thickness adopted (Ref. **IS:1905-1987**).

18.2 Control on Length and Height of Walls

In load bearing wall construction, as a general guide for categories B to E, the wall thickness 't' should not be kept less than 190mm, wall height not more than 20t and wall length between cross-walls not more than 40t. If longer rooms are required, either the wall thickness is to be increased or buttresses of full height should be provided at 20t or less apart. The minimum dimensions of the buttress should be : its thickness and top width equal to t and bottom width equal to one sixth of the wall height.

The overall height of load bearing masonry buildings should be limited to 4 storeys for category B to D and 3 storeys for category E, and the storey height restricted to 3.5m, unless rational design calculations are carried out using **IS:1893** and **IS:1905**.

18.3 Mortar Mix

The following mortar mixes or richer are recommended to be used for various building categories; **IS:4326-Table 3**:

Building Category	*Proportions of Cement:Lime:Sand*	*Min. Comp. Strength at 28 days, MPa*
B	1:0:7 or 1:3:12	1.5
C	1:0:6 or 1:2:9	3.0 or 2.0
D	1:0:5 or 1:1:6	5.0 or 3.0
E	1:¼:4 or 1:½:4½	7.5 or 6.0

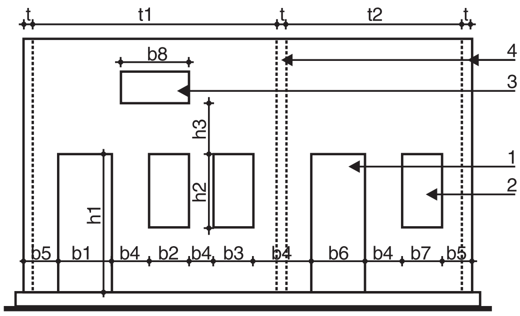
Note: Mixing of lime is not essential for strength but desirable for improving workability.

18.4 Control on Openings in walls

The door and window openings in walls should satisfy the requirements illustrated in *Fig. 25* and *Table 12*. However, when the openings do not satisfy these constrains, they should be boxed in reinforced concrete as shown in *Fig. 26*. In case the pier between two nearby openings is thinner than specified, the pier should be made stronger, say in plain or reinforced concrete.

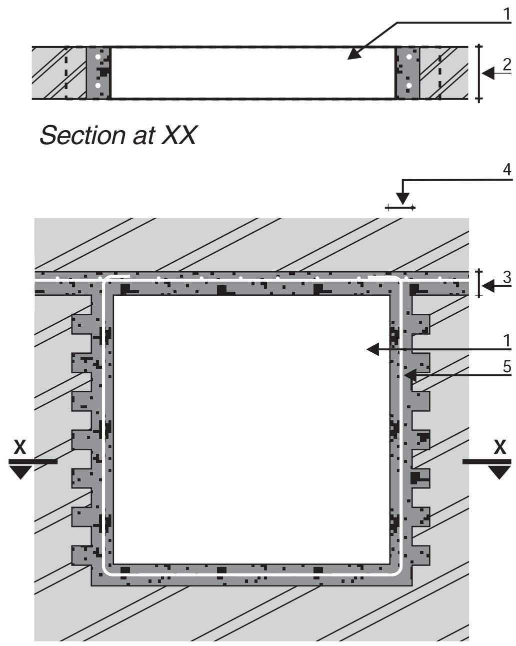
Table 12:
Size and Position of Openings in Bearing Walls (Fig. 25)

S. No.	Position in of Opening	Details of Opening for Building Category		
		A and B	C	D and E
1.	Distance b_5 from the inside corner of outside wall, Min	zero mm	230 mm	450mm
2.	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
3.	Pier width between consecutive openings b_4, Min	340mm	450mm	560mm
4.	Vertical distance between two openings one above the other h_3, Min	600mm	600mm	600mm
5.	Width of ventilator b_8, max	900mm	900mm	900mm



Legend
1 Door
2 Ventilator
3 Window
4 Cross wall

Fig. 25
Dimensions of openings and piers



Legend
1 Window
2 Wall thickness
3 Lintel thickness
4 Thickness of Jamb concrete
5 Vertical bar

Fig. 26
Strengthening masonry around window opening

18.5 Reinforced Concrete Bands

For integrating the walls of an enclosure to perform together like a rigid box and for imparting horizontal bending strength in the walls at critical levels, reinforced concrete bands are provided which *run continuously on all external and internal walls including fixed partition walls.* One or more of the following bands may be necessary in a building. Overall arrangement of reinforcing the masonry buildings is shown in *Fig. 27* for flat roof type and in *Fig. 28* for sloping roof type.

i. Plinth Band

This should be provided in those cases where the soil is soft or uneven in its properties as it usually happens in hill tracts. It will also serve as damp proof course. This band is not too critical.

ii. Lintel Band

This is the most important band and will incorporate in itself all door and window lintels, the

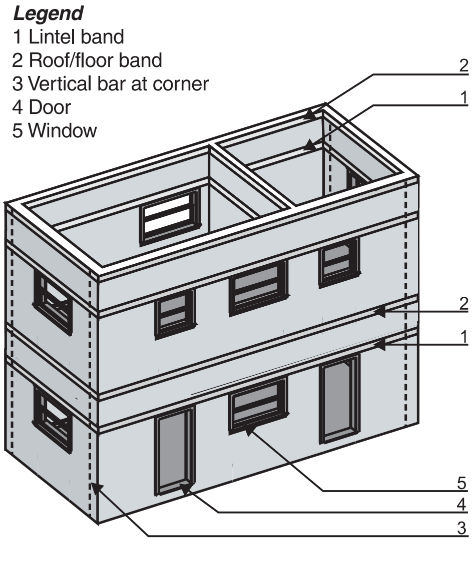


Fig. 27
Overall arrangement of reinforcing in masonry double storey building

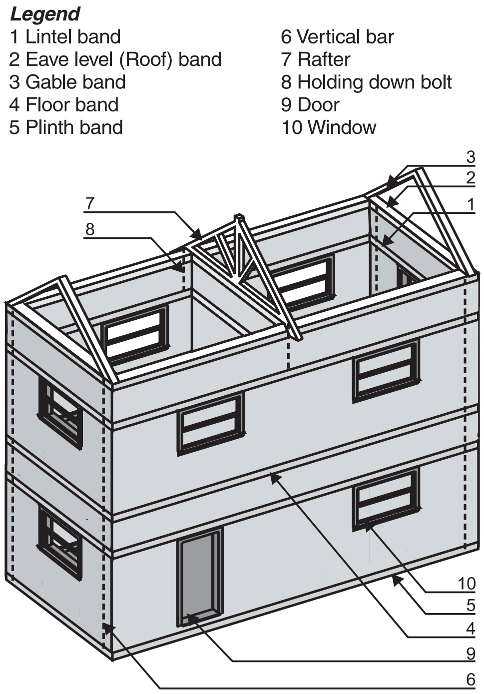


Fig. 28
Overall arrangement of reinforcing in masonry double storey building having pitched roof

reinforcement of which should be extra to the lintel band steel. It must be provided in all storeys.

iii. Ceiling/Roof Band

This band will be required below or in level with such floors or roof which consist of joists and flooring/roofing elements, so as to properly integrate them at ends and provide fixity into the walls.

iv. Eave Band

This band will be required at eave level of raftered or trussed roofs, and should go through the gable walls.

v. Gable Band

Masonry gable ends must have the triangular portion of masonry enclosed in a band, the horizontal part will be made continuous with the eave level band on longitudinal walls.

18.6 Details of the Bands

The various bands shall be made of reinforced concrete of grade not leaner than M15(1:2:4 nominal mix) laid over full width of the wall. The thickness of concrete should not be less than 75mm and the longitudinal reinforcement as indicated in Table 13. The section of the band and reinforcing details are shown in *Fig. 29*. As an alternative to full reinforced concrete section, composite section could be adopted as shown at (e) and (f) in the same figure provided that in case (f), the diameter of longitudinal bar will be taken larger by 2mm. The same sections may be used for the bands at various levels of the building.

Table 13:
Recommended Longitudinal Steel in Reinforced Concrete Bands

Span of Band BetweenCross Walls	Building Category **B**		Building Category **C**		Building Category **D**		Building Category **E**	
m	No. of Bars	Dia. mm	No. of Bars	Dia. mm	No. of Bars	Dia. mm	No. of Bars	Dia. mm
5 or less	2	8	2	8	2	8	2	10
6	2	8	2	8	2	10	2	12
7	2	8	2	10	2	12	4	10
8	2	10	2	12	4	10	4	12

NOTES:

1. *The number and diameter of bars given above pertain to high strength deformed bars (HSD or TOR).*
2. *Width of RC band (b) is assumed same as the thickness of the wall.*
3. *The vertical thickness of RC band be kept 75 mm minimum, where two longitudinal bars are specified, one each face; and 150 mm, where four bars are specified.*
4. *The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm dia (mildsted, ms)spaced at 150 mm apart or 8 mm dia HSD at 200 mm apart.*

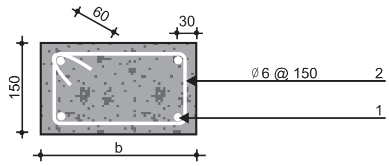


fig 29 a
Section with four bars

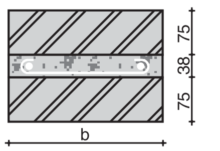


fig 29 e
R.B. band

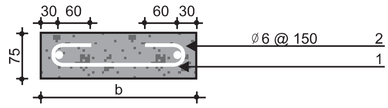


fig 29 b
Section with two bars

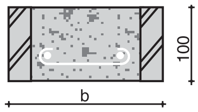


fig 29 f
Brick tiles used for forming

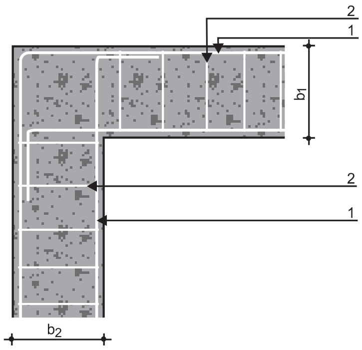


fig 29 c
Corner joint plan

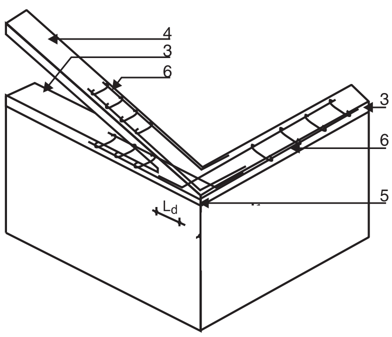
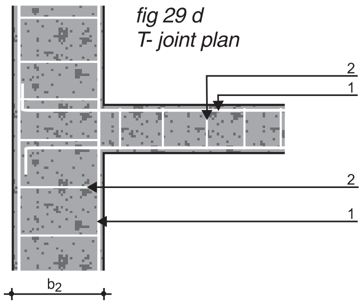


fig 29 g
Continuity of reinforcement in eave and gable bands

fig 29 d
T- joint plan



Legend
1 main bars
2 lateral ties / stirrups
3 eave band
4 gable band
5 joint of eave and gable bands
6 main reinforcing bars
L_d overlap length

Fig. 29
Reinforcement and bending detail in R.C.band

Table 14:
Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units

No. of Storeys	Storey	Diameter of HSD Single Bar in mm at Each Critical Section*			
		Category B	**Category C**	**Category D**	**Category E**
One		Nil	Nil	10	12
Two	Top Bottom	Nil Nil	Nil Nil	10 12	12 16
Three	Top Middle Bottom	Nil Nil Nil	10 10 12	10 12 12	12 16 16
Four	Top Middle Second Bottom	10 10 10 12	10 10 12 12	10 12 16 20	Four Storeyed Building not Permitted

* *The vertical bars will be covered with concrete M20 (or 1:1/1/2:3 nominal mix) or mortar 1:3 grade in suitably created pockets around the bars (see Fig.30). This will ensure their safety from corrosion and provide good bond with masonry.*

18.7 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 14. For walls thicker than 340 mm, the area of the bars shall be proportionately increased. *Fig. 30* shows details of brick work for the vertical bars by creating 115 x 115 pockets and filling them with M20(1:1½:3) cement concrete for good bond with brick work and corrosion resistance of bars.

The vertical reinforcement shall be properly embedded, with a minimum length equal to 55 diameters, 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 or suitably lapped to develop full tensile strength, preferably above the lintel band.

18.8 Framing of Thin Load Bearing Walls

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 must 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 lintel levels of the openings. The details are shown in *Fig. 31*. The sequence of construction between walls and columns is critical to good seismic performance of this system. The procedure will be,

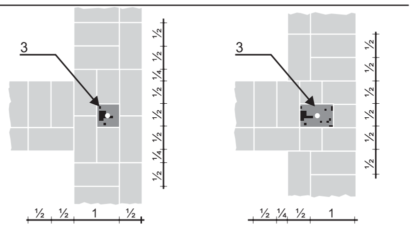


fig 30 e&f
Alternate courses at T-junction in one and a half brick wall

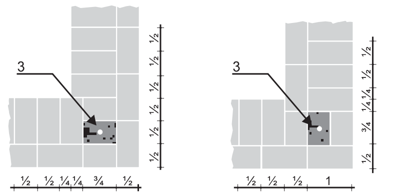


fig 30 c&d
Alternate courses in one and a half brick wall

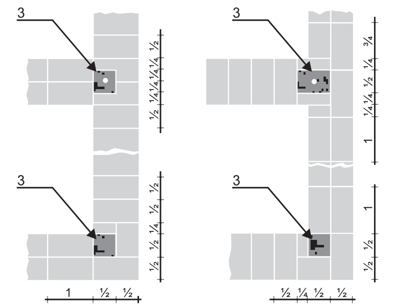


fig 30 a&b
Alternate courses in one brick wall

Legend
1 one brick length
1/2 half brick length
3 vertical steel bars with mortar / Conc filling in pocket

Fig. 30
Typical details of providing vertical steel bars in brick masonry

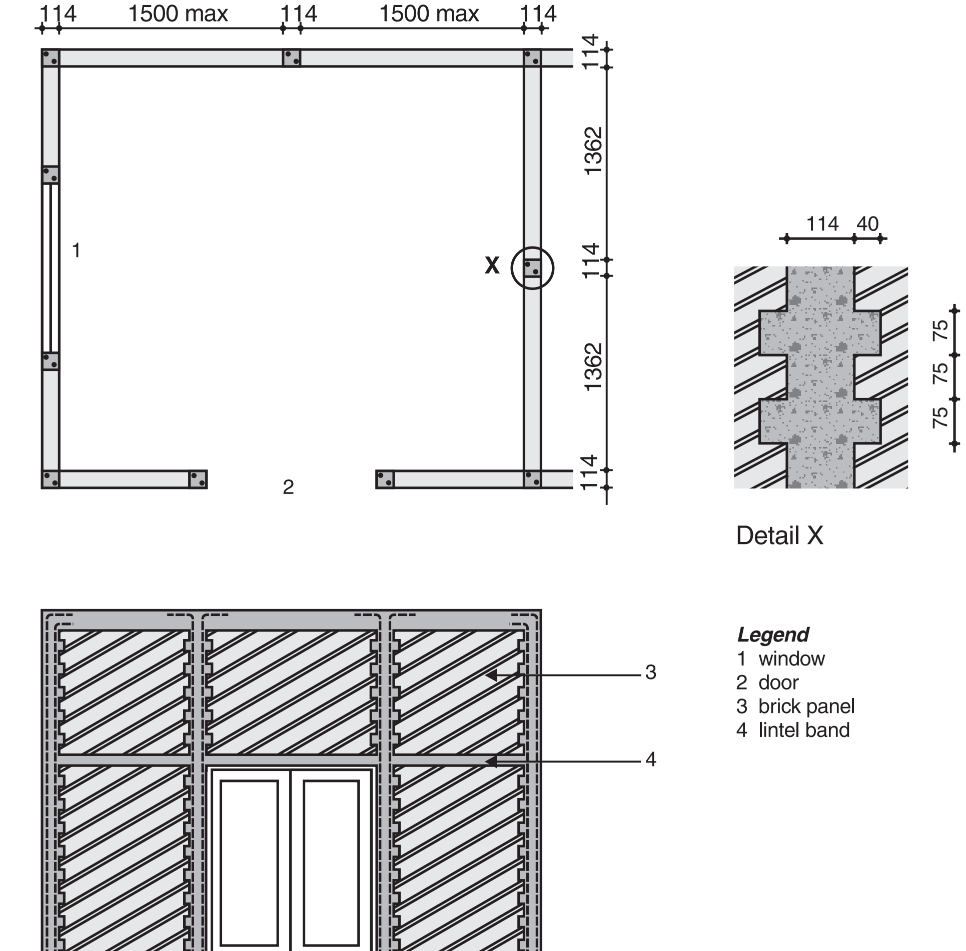


Fig. 31
Framing of thin load-bearing brick walls

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 M20(1:1½:3) concrete to fill the columns against the walls using wood forms only on two sides. The column 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. When the concrete is cast, *the wall should be wet by prewatering*.

Such construction may be limited to only two storeys maximum in view of its limited 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.

Note: *This construction scheme is similar to confined masonry which is used for taller buildings say upto 5 storeys, as a substitute for RC frame constructions. Its details are included in Annexure-2.*

18.9 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.

a. *Horizontal Band*

U-shaped blocks may be used for construction of horizontal bands at various levels of the storeys as shown in *Fig. 32(a)*, where the amount of longitudinal reinforcement shall be taken 25 percent more than that given in Table 13 and provided by using four bars and 6 mm dia stirrups. Other continuity details shall be followed, as shown before in *Fig. 29*.

b. *Vertical Reinforcement*

Bars, as specified in Table 14 shall be located inside the cavities of the hollow blocks, one bar in each cavity (*see Fig. 32b)*. 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 (by minimum 55 dia) 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 the figure which will avoid lifting and threading of bars into the hollows.

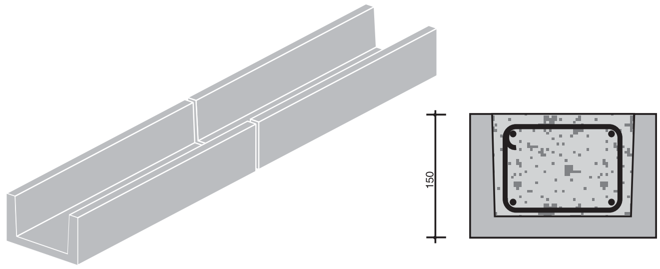


fig 32 a
U blocks for horizontal bands

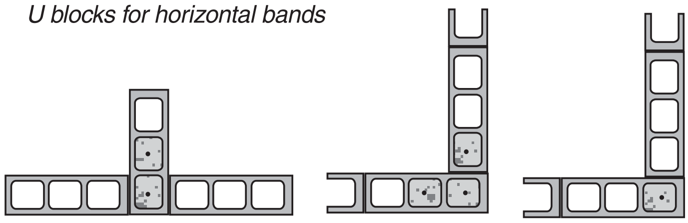


fig 32 b
Vertical reinforcement in cavities

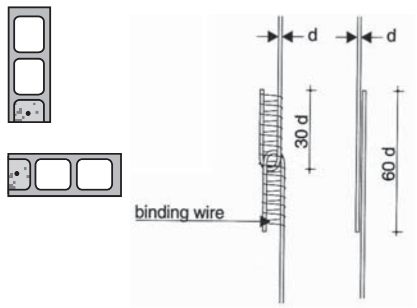


Fig. 32
Reinforcing hollow block masonry

19 BURNED BRICK BUILDINGS USING CLAY MUD MORTAR

These buildings are inherently weak due to low mortar strength and should not be permitted for important buildings with **I** = 1.5 and of Category D or E. The Important or Category D and E buildings, if already existing with mud mortar, should be retrofitted at the earliest to improve their earthquake resistance. The burned bricks should have a minimum compressive strength of 3.5 MPa for one, 5.0 MPa for 3 and more than 3 storey buildings.

19.1 Control on Wall Lengths and Buildings Height

a. The minimum wall thickness shall be one brick in one storey construction and in the top storey and 1.5 brick in bottom storeys of up to *three* storey construction. The length of wall between two consecutive perpendicular walls should not be more than 16 times the thickness.

b. The height of the building shall be restricted to the following, where each storey height shall not exceed 3.0 m:

For Categories B and C - three storeys with flat roof; and two storeys plus attic for pitched roof.

For Category D - two storeys with flat roof; and one storey plus attic for pitched roof.

Table 15:
Size and Position of Openings in Bearing Walls (see Fig.25)

Description		Building Category	
		A, B & C	D
i.	Distance b_5 from the inside corner of outside walls, Min.	230 mm	600 mm
ii.	Total length of openings. ratio; Max: $(b_1 + b_2 + b_3)/1_1$ or $(b_6 + b_7)/1_2$		
•	one storeyed building	0.46	0.42
•	2 & 3 storeyed building	0.37	0.33
iii.	Pier width between consecutive openings b_4	450 mm	560 mm
iv.	Vertical distance between two openings one above the other, h_3, *Min*	600 mm	600 mm

19.2 Control on Openings in Bearing Walls

a. The size and position of window and door openings shall be as given in Table 15 and *Fig. 25*. It will improve the strength of walls if the jambs of openings and the piers in between them are built using 1:6 cement mortar.

b. 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.

c. Where openings do not comply with the guidelines of Table 15, they should be strengthened by providing reinforced concrete lining as shown in *Fig. 26*.

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

19.3 Seismic Strengthening Features

The various seismic bands of reinforced concrete and the vertical steel bars at the critical sections shall be the same as given in **18.5** to **18.7.**

19.4 Water proofing

a. It will be useful to provide damp-proof course at plinth level to stop the rise of pore water into the superstructure.

b. Precautions should be taken to keep the rain water away from soaking into the walls so that the mortar is not softened due to wetness. An effective way is to take out roof projections beyond the walls by about 500 mm.

c. Use of a water-proof plaster on outside face of walls will enhance the life of the building and maintain its strength at the time of earthquake as well.

20 BUILDINGS WITH STONE MASONRY WALLS

20.1 Scope

Stone buildings using fully dressed rectangularized stone units, or cast solid blocks consisting of large stone pieces in cement concrete mix 1:3:6, may be built according to the details given in **18** or **19**. The random-rubble and half-dressed stone buildings are dealt with in this Para.

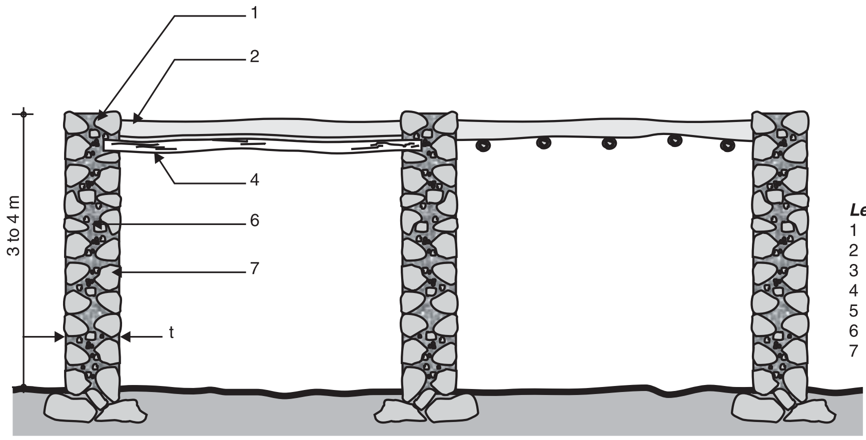


Fig. 33
Schematic cross-section through a traditional stone house

20.2 Typical Damage and Failure of Stone Buildings

Random rubble and half-dressed stone buildings (see *Fig. 33*) have suffered extensive damage and complete collapse during past earthquakes having Intensities of MSK VII and more. Besides other damage types, a typical and serious damage occurs by *delamination* and *bulging* of walls, that is, vertical separation of internal wythe from external wythe through the middle of wall thickness (see *Fig. 34*). This occurs mainly due to the absence of "through" or bond stones, weak mortar and chips filling between the wythes. In half-dressed stone masonry, the surface stones are pyramidal in shape having more or less an edge contact one over the other, thus the stones have an unstable equilibrium and easily disturbed under shaking condition. Crumbling and collapsing of bulged wythes occurs after delamination under heavy weight of roofs/floors, leading to collapse of roof along with walls or causing large gaps in walls.

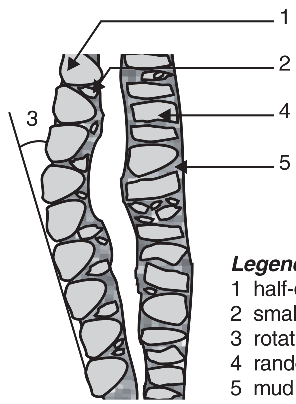


Fig. 34
Wall delaminated with buckled wythes

20.3 Construction Control

a. The mortar should preferably be cement-sand (1:6), otherwise, lime-sand (1:3) or clay mud of good quality.

b. The wall thickness 't' should not be larger than 450 mm even with mud mortar. It should be about 350 mm when using cement mortar, and the stones on the inner and outer wythes should be interlocked with each other.

c. The masonry should preferably be brought to courses at not more than 600 mm lift.

d. 'Through' stones of full length equal to wall thickness should be used in every 600 mm lift at not more than 1.2 m apart horizontally. If full length stones are not available, stones in pairs each of about 3/4 of the wall thickness, may be used in place of one full length stone so as to provide an overlap between them (see *Fig. 35*).

e. In place of 'through' stones, 'bonding elements' of steel bars 8 to 10 mm dia bent to S-shape or as hooked links may be used with a cover of 25 mm from each face of the wall (see *Fig. 35*). Alternatively, wood bars of 38 mm x 38 mm cross section or concrete

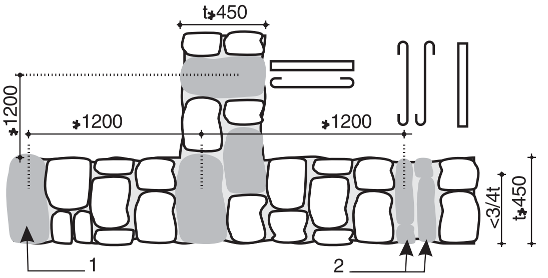


fig 35 a
Sectional plan of wall

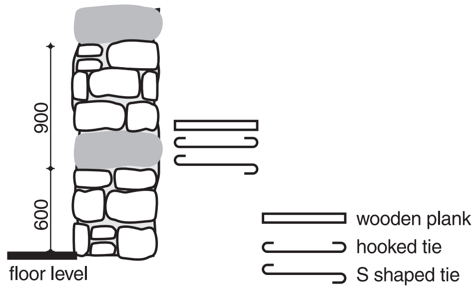


fig 35b
Cross-section of wall

Legend
1 through stone
2 pair of overlapping stone

Fig. 35
Through stone and bond elements

bars of 50 mm x 50 mm section with an 8 mm dia rod placed centrally may be used in place of 'through' stones. The wood should be well treated with preservative so that it is durable against weathering and insect action.

20.4 Control on Wall Length and Building Height

a. Height of the stone masonry walls (random rubble or half-dressed) should be restricted as follows, with storey height to be kept 3.0 m maximum, and span of walls between cross walls to be limited to 5.0 m:

   i. *For Category* B- Two storeys with flat roof or one storey plus attic, if walls are built in lime-sand or mud mortar; and one storey higher if walls are built in cement-sand 1:6 mortar.

   ii. *For categories C and D* - Two storeys with flat roof or two storeys plus attic for pitched roof, if walls are built in 1:6 cement mortar; and one storey with flat roof or one storey plus attic, if walls are built in lime-sand or mud mortar.

b. If walls longer than 5 m are needed, buttresses may be used at intermediate points not farther apart than 4.0 m. The size of the buttress be kept of uniform thickness with top width equal to the thickness of main wall, t, and the base width equal to one sixth of wall height.

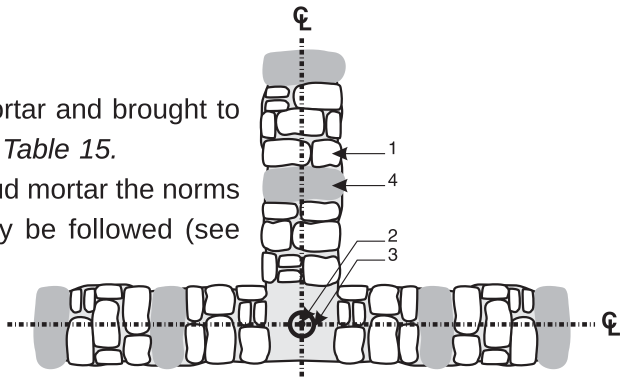
20.5 Control of Openings in Bearing Walls

a. For stone masonry built in cement mortar and brought to courses, the openings may be as per *Table 15.*

b. For random rubble masonry built in mud mortar the norms as laid down for earthen houses may be followed (see **21.4**).

20.6 Seismic Strengthening Features

The various seismic bands and vertical steel bars at the critical sections shall be the same as given in **18.5** to **18.7**. For installation of vertical bars in field stone masonry, use of a casing pipe of 100 mm dia is recommended around which masonry be built to height of 450 to 600 mm (*See Fig. 36*), and the pipe kept loose by



Legend
1 stone wall
2 vertical steel bar
3 casing pipe
4 through stone or bonding element

Fig. 36
Typical construction detail for installing vertical steel bar in random rubble stone masonry

rotating it. The pipe is then raised, the cavity filled with M15 (1:2:4) grade concrete and fully compacted.

20.7 Water Proofing

For protection against damage by water, use methods as given in **19.4**.

21 EARTHEN HOUSES

Earthen houses are weak against water action through rains and flooding as well as against earthquakes. They may be cracked at MSK VI, wide cracks and even partial collapse can occur at MSK VII and collapses are quite widespread under MSK VIII. Damage is much more severe in two storeyed than one storeyed houses. By good quality construction and earthquake resisting features, their collapse can be prevented under MSK VIII. Earthen buildings should not be permitted for important or Category E buildings and in flood prone eares.

21.1 Scope

Earthen houses of the following wall types are considered here

A. hand - formed by layers
B. Unburnt (sun-dried) bricks and blocks (Adobe) laid in mud
C Rammed earth in moulds
D. Earthen walls reinforced with bamboo or cane structure.

21.2 Control on Construction

a. The soil has necessarily to be clayey to have good cohesion and dry strength, but should not show fissures on drying. It has to be chosen also to suit the process of building wall stated in **21.1**. Simple tests are indicated in **IS:13827-6.1** to **6.4** for finding the suitability of the soil.
b. Hand formed walls should preferably be made tapering from 300 mm minimum at top and increasing with a batter of 1 in 12 at the base.
c. The bearing length of door and window lintels and roof joists should not be less than 300 mm.

21.3 Control on Length of Walls and Building Height

a. The height of earthen house should be restricted to one storey or one storey plus attic in seismic zones IV and V and to two storeys in other zones.
b. The height of wall ‘h’ should not be greater than 8 times its thickness ‘t’ nor 2.8 m.
c. The length of a room should not exceed 10t nor 64 t^2/h.
d. When a longer wall is needed, the walls should be thickened or strengthened by intermediate pilasters/buttresses.

21.4 Control on Openings in Bearing Walls

a. The width of a window or door opening should not exceed 1.2 m.
b. The distance between an outside corner and edge of opening or between two openings should not be less than 1.2 m.
c. The top level of windows and doors should be kept at the same level, say at 1.8 m above the plinth/floor level so as to have a continuous band.

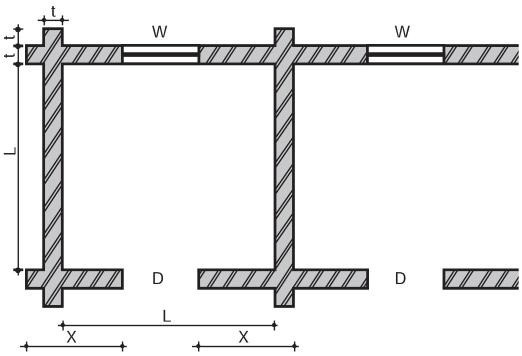


fig 37 a

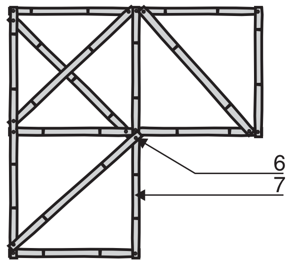


fig 37 c1
Bamboo dovels on corner without pilasters

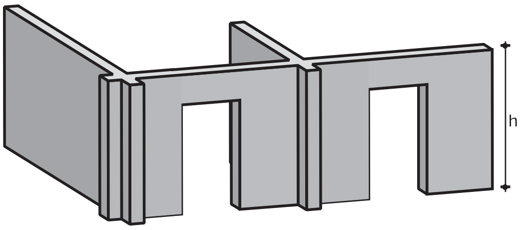


fig 37 b

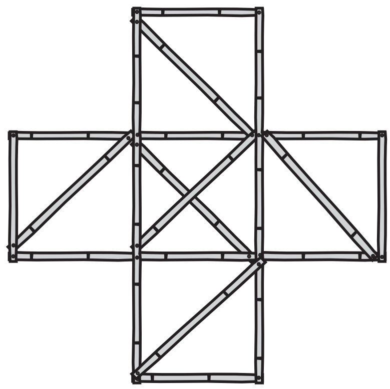


fig 37 c2
Bamboo dovels on corner with pilasters

Legend
1 D-Door, ≤ 1200
2 W-window, ≤ 1200
3 L-length, ≤ 10 t, 4 m
4 x-pier, ≥ 1200
5 h-height, = 2400-3000
6 Half split bamboo 50 mm dia
7 Wire nail 4 mm dia and 75 or 100 mm long clenched at other side

Fig. 37
Wall dimensions, pilasters at corners

21.5 Seismic Strengthening Features

a. *Pilasters*

Adopt a plan with dimensional control and pilasters of dimension txt at corners and junctions of walls, as shown in *Fig. 37*, in all seismic zones II to V and also in cyclone prone areas.

b. *Band or Collar Beams*

In seismic zones III to V, install two horizontal bands or collar beams, one at the lintel level of doors and windows and the other at ceiling or eave level to cover all walls continuously including the pilasters. In addition install dowels at corners and junctions of walls at window sill level. These bands and dowels could be of wood or bamboo as shown in *Fig. 38* and *39*, which should be seasoned and chemically preserved before laying.

A similar band should be laid on top of gable walls and tied with the eave bands (*Fig. 40*).

The lengthening joints in the band elements should be made strong using iron-strips and nails/bolts.

Note:

1. If the height of wall from floor to ceiling is 2.4 m or less, the lintel band can be omitted but the door and window lintels integrated with the ceiling/eave band (*Fig. 39*).
2. In Seismic Zone II, only one band at the ceiling/eave level may be used.

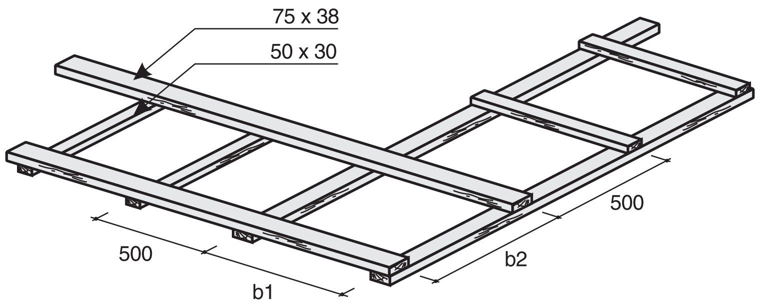
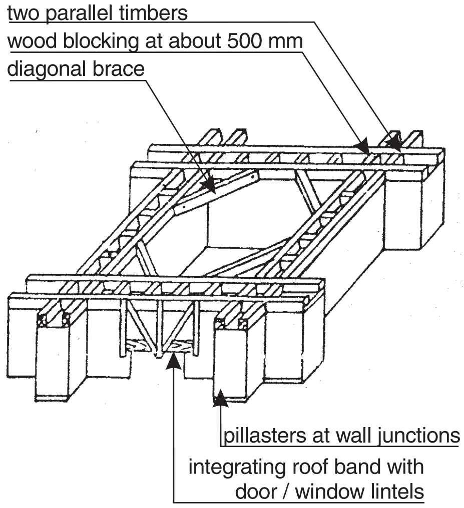


Fig. 38
Wooden band for low strength masonry earthern buildings



Where pillasters or buttresses are used at corner or T junctions, the collar beam should cover the buttresses as well. Use of diagonal struts at corners willl further stiffen the collar beam

Fig. 39
Roof band on pillastered walls

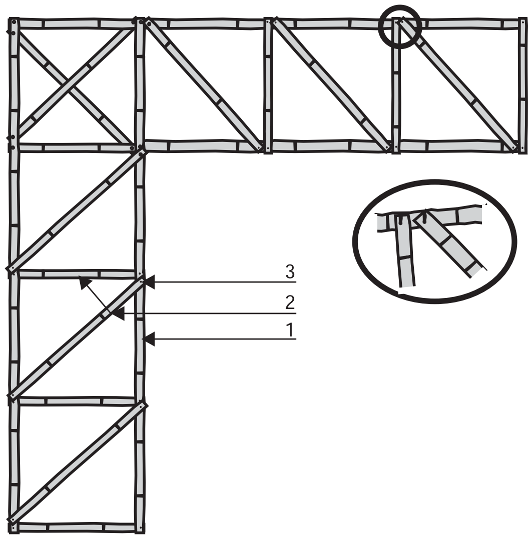
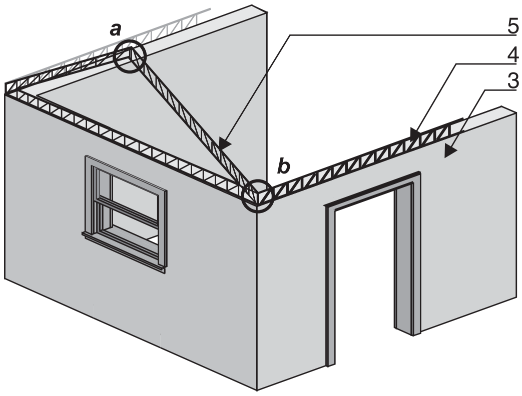
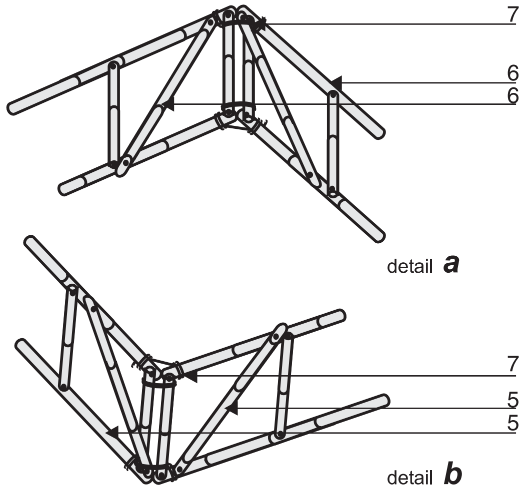


fig 40 a
Seismic band made of bamboo





Legend
1 half split 75 dia bamboo
2 half split 50 dia bamboo
3 through nails clenched at other ends
4 wall
5 eave level band
6 gable band
7 binding wire

Fig. 40
Connecting bamboo bands together

fig 40 b
Connecting bands

c. *Vertical Reinforcing*

This will be required in Seismic Zones IV and V and may be in the following forms:-

i. Vertical bamboos 100 mm in dia one at each corner or junction of walls from foundation to the eave/ceiling band to which the bands are securely nailed. (*See Fig. 41*).

ii. In zone V use additional vertical bamboos at the jambs of doors and windows.

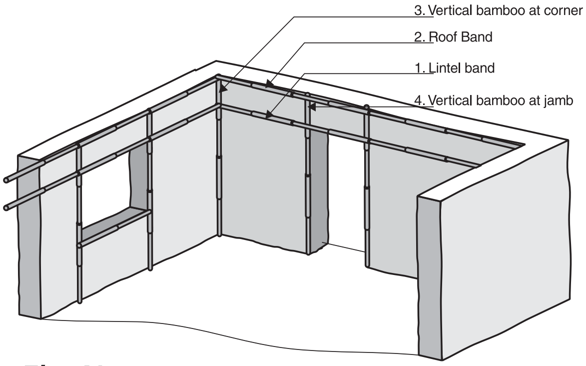


Fig. 41
Use of vertical bamboos at corners and jambs in earthern walls

iii. Alternatively, whole clay walls may be reinforced with structures of canes/bamboos of small dia, say 20-25 mm as shown in *Fig. 42*. Such structures will be tied/nailed to vertical bamboos at the corners or points of discontinuities.

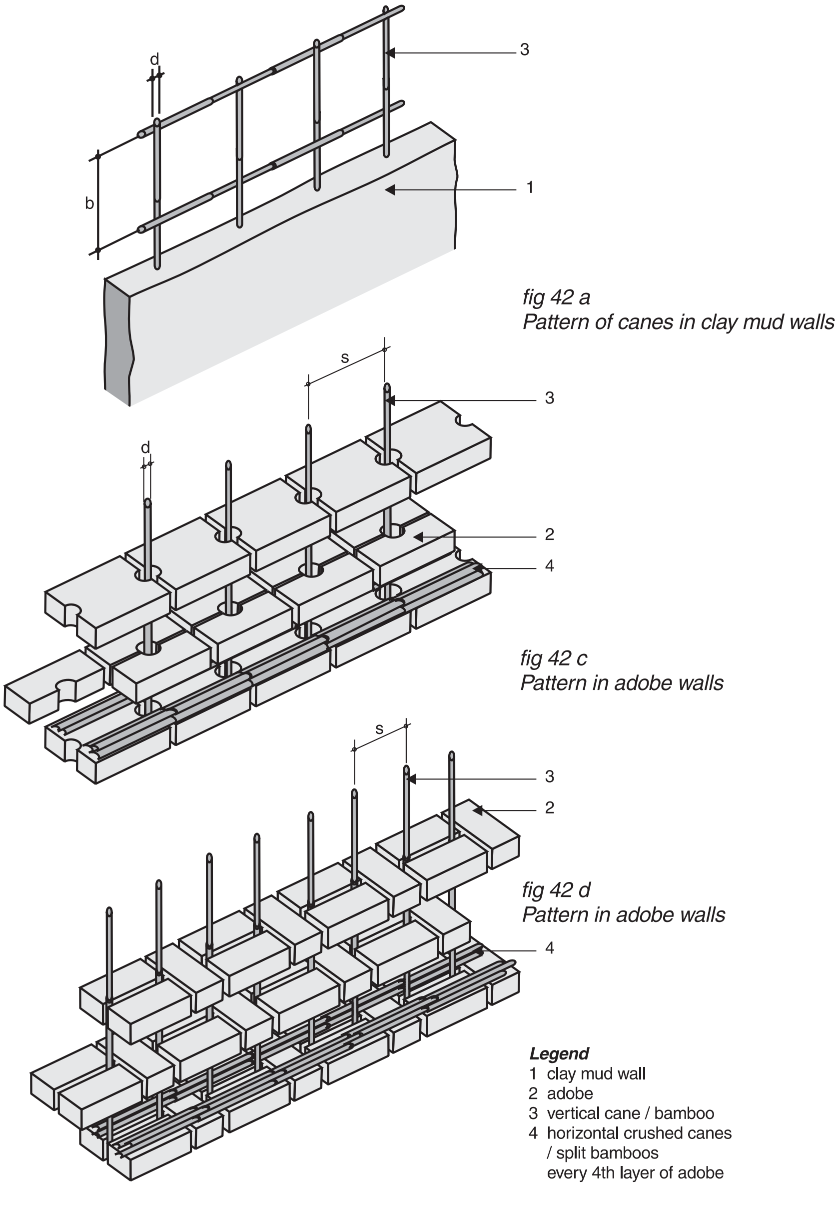


Fig. 42
Reinforcement in earthern walls

21.6 Foundation and Plinth

a. Footing

The footing should preferably be built by using stone or fired brick laid in cement or lime mortar. Alternatively, it may be made in lean cement concrete with plums (cement :sand : gravel : stones as 1:4:6:10) or without plums as 1:5:10. Lime could be used in place of cement in the ratio lime:sand:gravel as 1:4:8.

b. Plinth Masonry

The wall above foundation up to plinth level should preferably be constructed using stone or burnt bricks laid in cement or lime mortar. Clay mud mortar may be used only as a last resort.

The height of plinth should be above the flood water line or a minimum of 300 mm above ground level. It will be preferable to use a water proofing layer or heavy black polythene sheet at the plinth level before starting the construction of superstructure wall. If adobe itself is used for plinth construction, the outside face of plinth should be protected against damage by water by suitable brick facia or plaster. A water drain should be made slightly away from the wall to save it from seepage.

21.7 Water Proofing

For protection against damage by water, see **19.4**.

22 WOOD HOUSES

Timber has high strength per unit weight, hence very suitable for building earthquake resistant houses. But unless seasoned and preserved chemically, it is liable to rot and attack by insects losing its strength and stiffness with the passage of time. For this reason, the old wood buildings showed unsatisfactory behaviour during the Kobe earthquake killing thousands of persons as they collapsed. Then they caught fire and burned all the contents also. Attention therefore must be paid to the following:

i. Seasoning and chemical treatment of timber before use.
ii. Fire proofing details and maintenance.
iii. Tight joints using galvanized iron straps with nails/screws/bolts.
iv. Starting the timber construction above masonry plinth/pedestals above the high flood level.
v. Bracing the wood-frame in vertical as well as horizontal planes to prevent its distortion under the lateral seismic forces.

22.1 Foundation

The foundation may be strip type or isolated pedestals and the superstructure fixed to them as shown in *Fig. 43*.

22.2 Control of Bearing walls, Openings and Building Height

The plan of the building should be surrounded and divided by bearing wall lines, with maximum spacing of the bearing wall lines as 8 m. The width of openings in the bearing wall lines should not exceed 4 m and the opening kept at least 50 cm away from the corner. Adjacent openings should be at least 50 cm apart. (*See Fig. 44*).

All bearing wall lines of the upper storey should rest on the bearing wall lines of the lower storey. The bearing walls may have stud wall type or brick-nogged type construction. The height of the building will be limited to two storeys or two storeys plus attic.

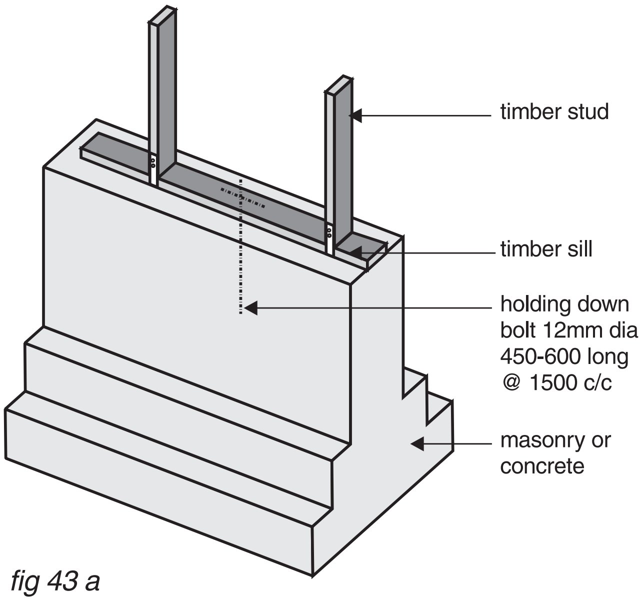


fig 43 a
Suitable for strip foundations

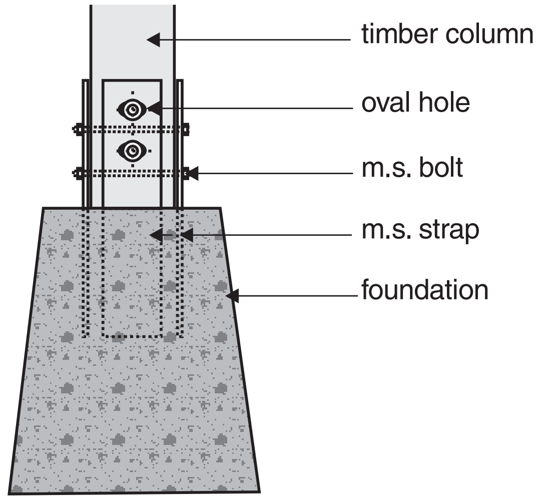


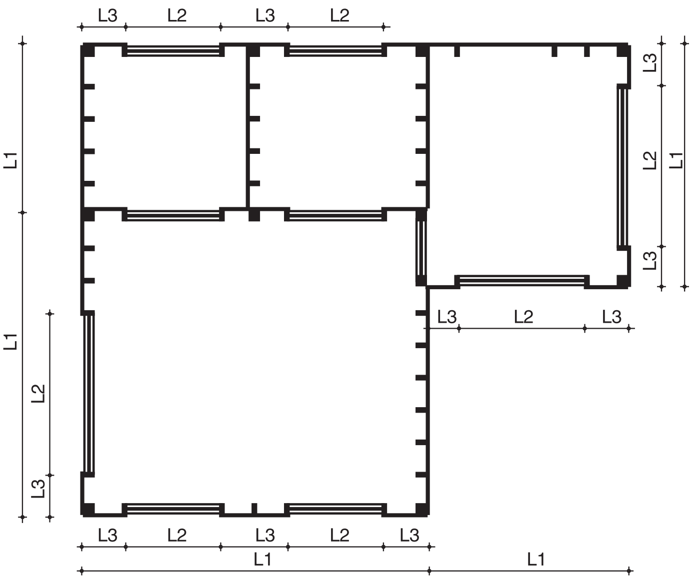
fig 43 b
Suitable for isolated column footings

Fig. 43
Details of connection of column with foundation

22.3 Stud Wall Construction

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 due to earthquake and wind. The wall covering may consist of matting made from bamboo, *ekra,* reeds, and timber boarding or the like. Typical details of stud walls are shown in *Fig. 45* (See **4326-10.7** for more details).

If the sheathing boards are properly nailed to the timber frame, the diagonal bracing may be omitted. The diagonal bracing may be framed into the verticals or nailed to the surface.



Legend
L1<= 8m Max spacing bearing wall lines
L2<= 4m Max width of openings
L3>= 50cm Min width of bearing wall adjacent to the openings

Fig. 44
Plan divided by bearing wall lines

22.4 Brick Nogged Timber Frame Construction

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. 46*. (See **4326-10.8** for details).

It will be useful to use holdfasts screwed to the wooden members for providing positive

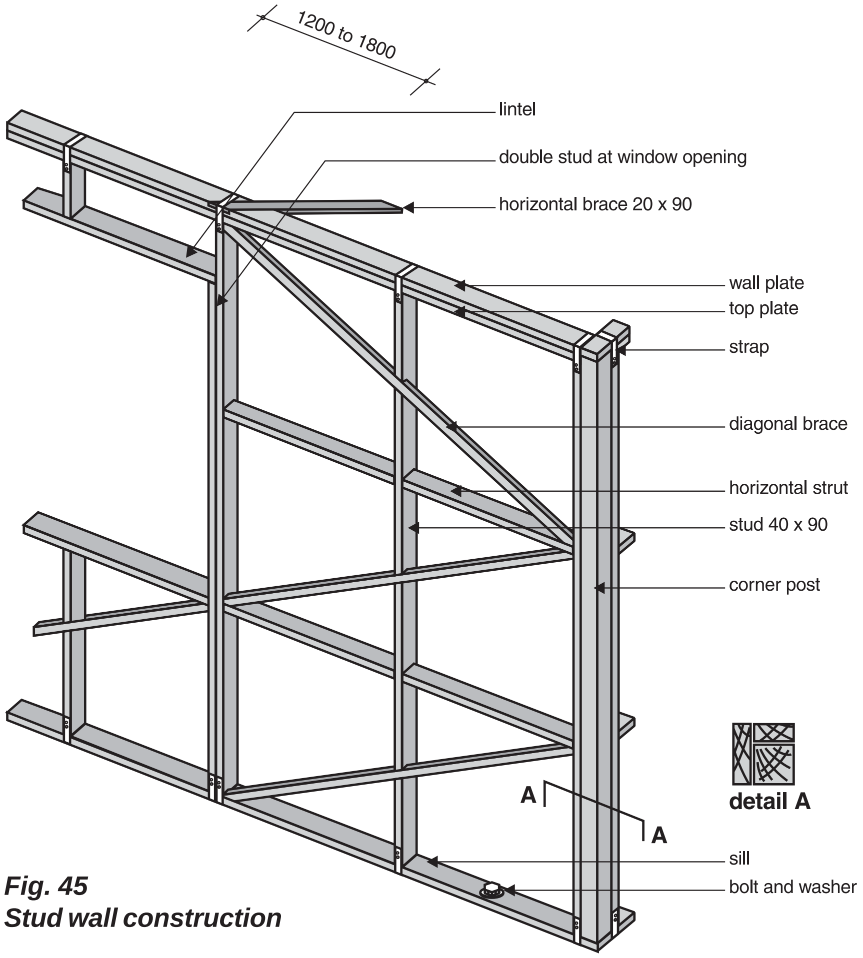


Fig. 45
Stud wall construction

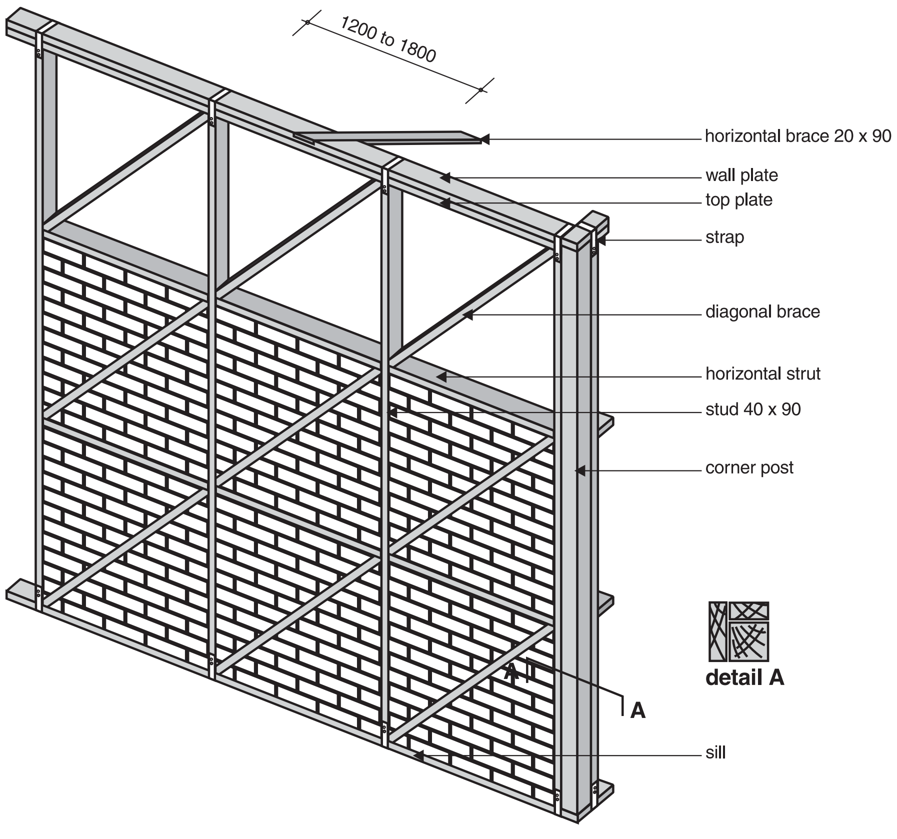


Fig. 46
Brick nogged timber frame construction

connection between the frame and the brick nogging for preventing out of plane falling of the brick work during earthquake shaking.

23 REPAIR AND RESTORATION OF DAMAGED BUILDINGS

Buildings affected by an earthquake may suffer both non-structural and structural damage, and should be clearly differentiated. Also the severity of the structural damage must be carefully studied before a decision is taken about repair and restoration or demolition of the structure. Often it has been seen that recommendations are made in great haste in favour of dismantling and reconstruction, which is not only costlier but also involves displacement of the residents or shifting of the functions being performed in the building. The proper categorisation of the damage as per Table 3 is therefore, of basic importance for post-earthquake rehabilitation program.

23.1 Non-structural/Architectural Repairs

The main purpose of these repairs is to bring back the architectural shape of the building so that all services start working and the functioning of building is resumed quickly. The actions will include 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 wiring.
d. Checking and repairing gas pipes, water pipes and plumbing services.
e. Re-building non-structural walls, smoke chimneys, boundary walls, etc.
f. Replastering of walls as required.
g. Rearranging disturbed roofing tiles.
h. Relaying cracked flooring at ground level.
i. Redecoration - white washing, painting etc.

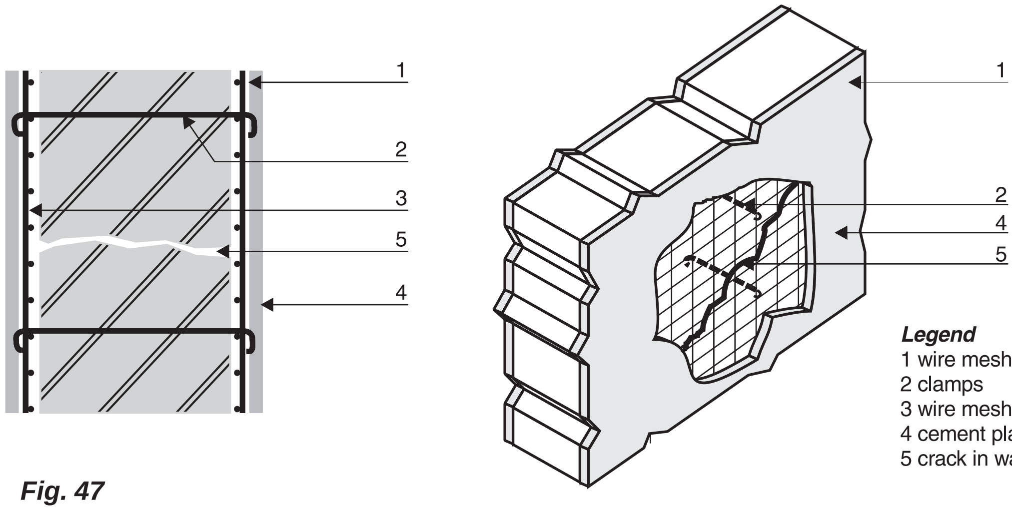
The architectural repairs as stated above do not restore the original structural strength of cracked walls or columns and may sometimes be very illusive, since the redecorated building will hide all the weaknesses and the building will suffer even more severe damage if shaken again by an equal shock since the original energy absorbing capacity will not be available.

23.2 Structural Repairs or Restoration

The main purpose of restoration is to carry out structural repairs to load bearing elements. It may involve cutting portions of the elements and rebuilding them or simply adding more structural material so that the original strength is more or less restored. The process may involve inserting temporary supports, underpinning etc. Some of the approaches are stated below:

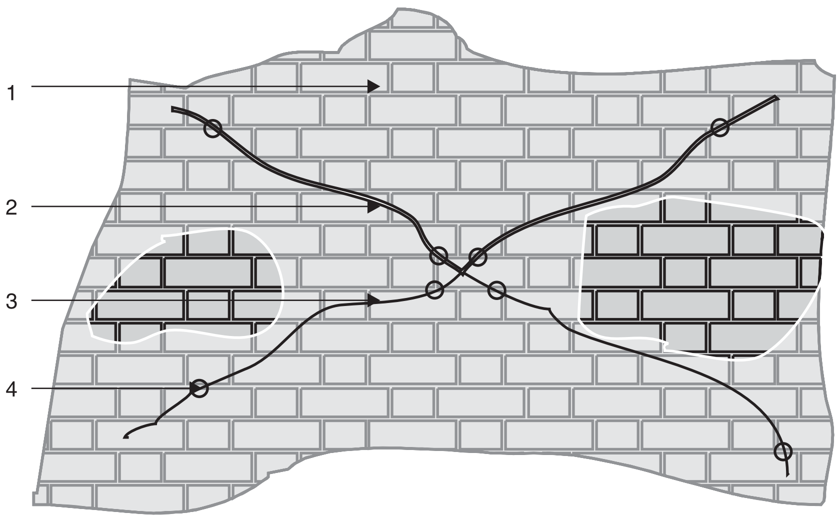
a. Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of non-shrinking mortar will be preferable.
b. Addition of reinforcing mesh on both faces of the cracked wall, holding it to the wall through spikes or bolts and then covering it suitably *(Fig 47)*. The strength of the reinforced cover plates should be designed to more than balance the lost strength of the cracked elements.
c. Injecting epoxy, cement grout or like materials, or cement mortar which is strong in tension, into the cracks in columns, beams, walls etc., depending on the width of crack and lost strength to be restored. See *Fig.48* and *Fig. 49*.

Where structural repairs are considered necessary, these should be carried out prior to or simultaneously with the architectural repairs so that total planning of work could be done in a coordinated manner and wastage is avoided.



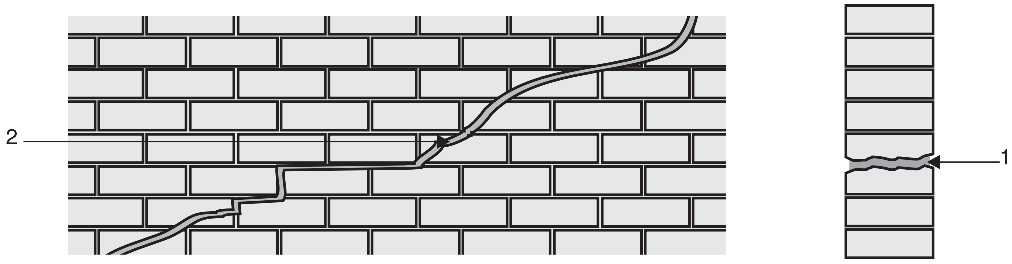
Legend
1 wire mesh on front face
2 clamps
3 wire mesh on back face
4 cement plaster
5 crack in wall

Fig. 47
Strengthening of existing masonry



Legend
1 plaster removed
2 crack
3 crack sealed after cleaning
4 grout ports

Fig. 48
Cement grout or epoxy injection in cracks



Legend
1 v-grooved joints
2 cement mortar

Fig. 49
Cement - coarse sand mortar in wide cracks

24 STRENGTHENING OF EXISTING BUILDINGS

The old existing buildings are damaged in earthquakes due to their original structural inadequacies, absence of earthquake resisting features, material degradation due to time, and alterations carried out during use over the years such as making new openings, addition of new parts inducing dissymmetry in plan and elevation etc.

The possibility of substituting a building with new earthquake resistant building is generally not feasible due to historical, artistic, social and cultural reasons. The complete replacement of the buildings in a given area will also lead to destroying a number of social and human links. In the Indian context where more than 80 percent residential buildings are highly vulnerable to damage and collapse in heavily populated area of the country, seismic strengthening of existing damaged or undamaged buildings is a definite requirement.

Strengthening by retrofitting gives the desired improvement in the original strength and *restoration* alone will not be adequate in future quakes.

24.1 Approach to Seismic Retrofitting

The structural deficiencies of the building to be retrofitted should first be evaluated by comparison to similar earthquake resistant buildings, or by calculation where found necessary and feasible. Guidance in this regard can be taken from *Tables 9 to 11.* Then the level of safety desired and the methodology for achieving that should be worked out along with costing of each measure. In most cases the aim of retrofitting will be to achieve safety from total or partial collapse in an earthquake intensity considered in IS:1893-2002 for the Zone in the which the building is situated. Fortunately this safety level is achievable with economically feasible inputs.

It should be understood that some features can not be achieved practically howsoever desirable they may be. For example the mud mortar used in masonry can not be replaced by cement mortar without dismantling and rebuilding the masonry. But the critical locations of the walls can be strengthened by installing ferrocement plates on its faces.

24.2 Cost of Repairs and Retrofitting

From practical and economy view point, the structural repairs and strengthening operations should be taken simultaneously and the architectural repair and finishing work should follow.

As a rough guide repair, restoration and retrofitting are resorted to if their cost remains less than 50 percent of the cost of reconstruction taking into account the salvaged material. Experience of repair and strengthening of damaged stone houses in Latur area indicated the cost to be less than 20 percent, and in Jabalpur city, the cost of repair and retrofitting of Housing Board's two to three storeyed buildings came to about 6 percent only.

24.3 Objectives of Seismic Retrofitting Procedures

a. Eliminating features that are sources of weakness or that produce concentrations of stresses in some members, such as asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor to the other, concentration of large masses, long rooms without cross walls or buttresses, large openings in walls without a proper peripheral reinforcement, etc.

b. Giving unity to the structure by providing a proper connection between its resisting elements, for example ensuring diaphragm action of roof and floors, improving connections between roofs or floors and walls, between intersecting walls and between walls and foundations.

c. Increasing the lateral strength in one or both directions by reinforcement or by increasing wall areas or the number of walls and columns.

d. Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members.

24.4 Seismic Retrofitting Features

Besides the structural repairs, the seismic retrofitting features will consist of one or more of the following features:

a. *Modification of Sloping Roofs*

- Replacement of brittle roofing elements like slates and clay tiles with corrugated galvanised iron or some other sheeting
- Conversion of raftered roofs to frames or trusses by installing ties and other members *(Fig. 50a)*
- Bracing of trusses in the planes of rafters and main ties *(Fig. 50b)*
- Strengthening the joints between various elements of sloping roofs by welding, strapping or tying by binding wire.
- Improving the anchoring of roof trusses into walls/columns.

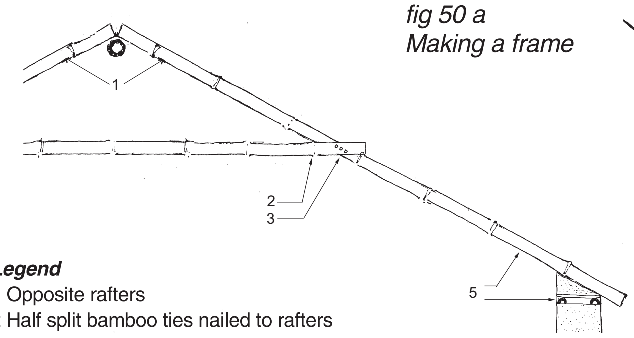
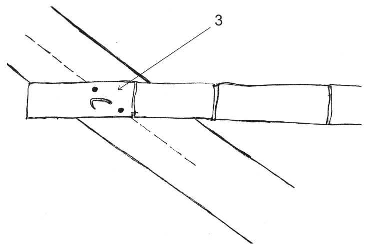


fig 50 a
Making a frame



Legend
1 Opposite rafters
2 Half split bamboo ties nailed to rafters
3 3 nails of 5mm dia at joints
4 50mm dia bamboo to be nailed to all the rafters
5 Seismic band and sloping rafter to be tied together by wire
6 Cross-bracings using bamboos to connect rafters at ends of each room

Note:
To avoid splitting the ends of bamboos, drill a 4mm hole in the bamboos before nailing

Fig. 50
Bamboo truss system

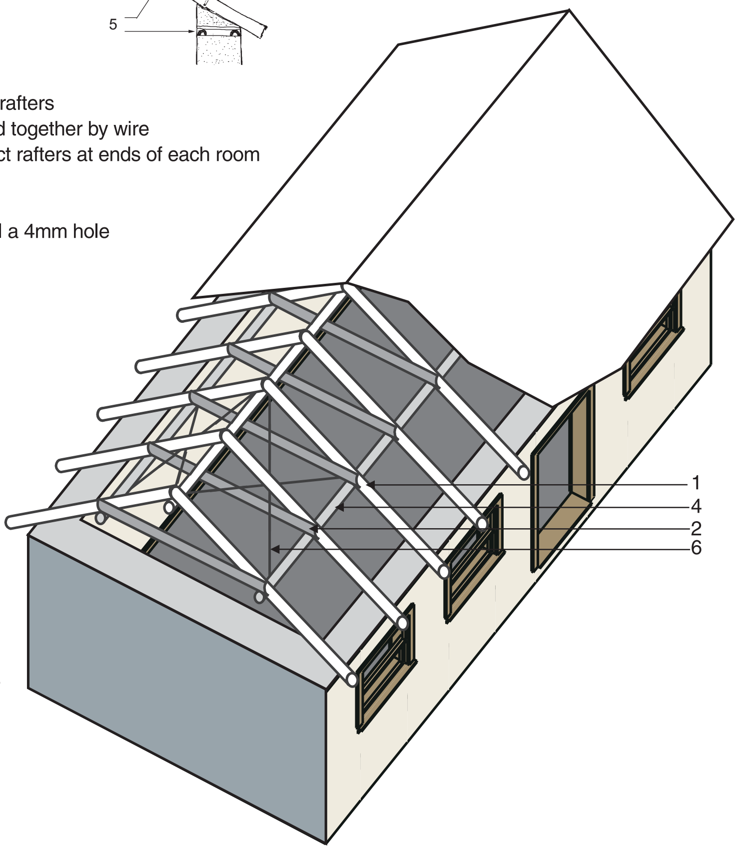
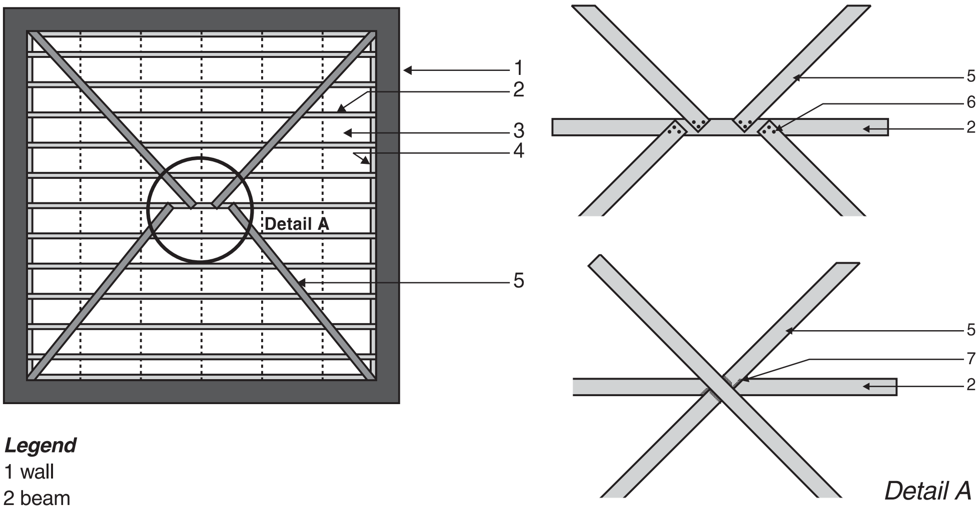


fig 50 b
Installing braces



Legend
1 wall
2 beam
3 flooring elements
4 planks / flats
5 diagonal brace
6 nailed joint, where wooden joist is used
7 weld joint where T or I joist is used

Fig. 51
Stiffening of joisted floor

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b. Modification of Flat Roofs and Floors

- Where the roof or floor consists of prefabricated units like RC rectangular, T or channel units or wooden ballis and joists carrying brick tiles, stone slabs or prefab RC elements, 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 *(Fig. 51)*. RC elements may either have 35-40 mm cast-in-situ-concrete topping with 6 mm bars 150mm c/c both ways and a horizontal cast-in-situ RC ring beam all round into which the ends of RC elements are embedded *(Fig. 52)*.
- Roofs or floors consisting of steel joists and flat or segmental arches must have horizontal ties holding the joists horizontally in each arch span so as to prevent the spreading of joists *(Fig. 53)*. If such ties do not exist, these should be installed by welding or clamping. In severe seismic Zone V, diagonal bracing may also be provided in addition.

c. Modification and Strengthening of Masonry Walls

- Addition of new cross walls or buttress in case of long rooms to provide stability to the long walls; fixing of the additions into existing walls is important by shear keys or dowels *(Fig. 54)*.
- Strengthening piers between non-compliant openings by ferrocement plating or closing/ reducing some of the windows *(Fig. 55)*.
- Addition of new walls to provide greater shear resistance or to minimise dis-symmetry.
- Installation of external and internal horizontal seismic belts in Category C, D, E buildings by covering the spandrel masonry in a depth of about 450 mm between the lintel and ceiling levels in each storey *(Fig. 56)*. In Category B houses the internal belt may be omitted. The following details may be followed:
  a. Height of belt=400 to 450 mm; longitudinal wires in weld-mesh=16 of about 2 mm dia at 25 mm c/c; transverse wires same dia at about 150 mm c/c; micro-concrete 1:1½:3 or cement-coarse sand mortar 1:3 of 30 to 35 mm thickness.
  b. The mesh should be continuous with 150 mm overlap at the corners or elsewhere.
  c. Remove plaster; rake out mortar joints to 12-15 mm depth; clean surface; wet surface

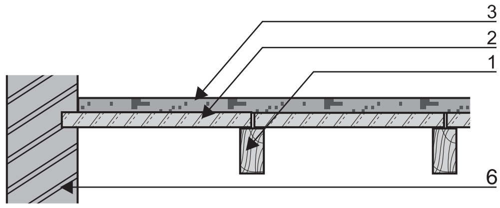


fig 52 a
Laterally flexible floor systems

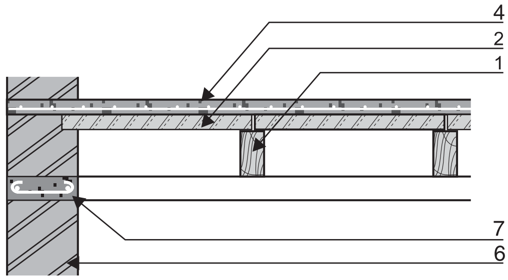


fig 52 b
Ceiling level band and R.C. floor

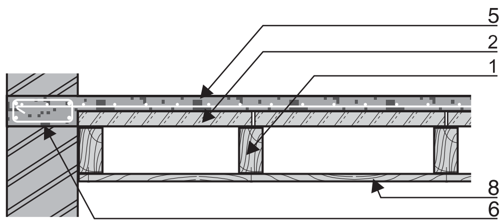
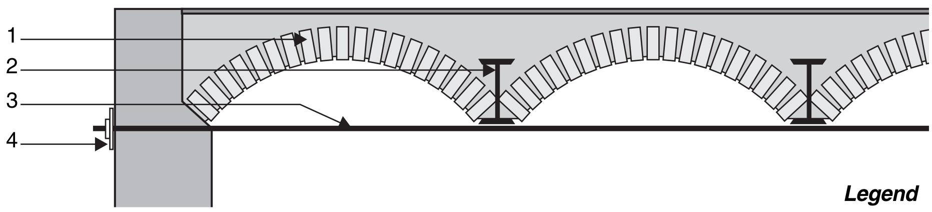


fig 52 c
Stiffened flexible floor with seismic band

Legend
1 wood joist
2 stone slab or prefab element
3 plain concrete or some other flooring
4 R.C. floor
5 R.C. screed
6 masonry wall
7 R.C. band
8 diagonal wood plank

Fig. 52
Flooring systems



Legend
1 arch
2 steel beam
3 flat iron or rod
4 bearing plate

Fig. 53
Strengthening an arched opening in masonry walls

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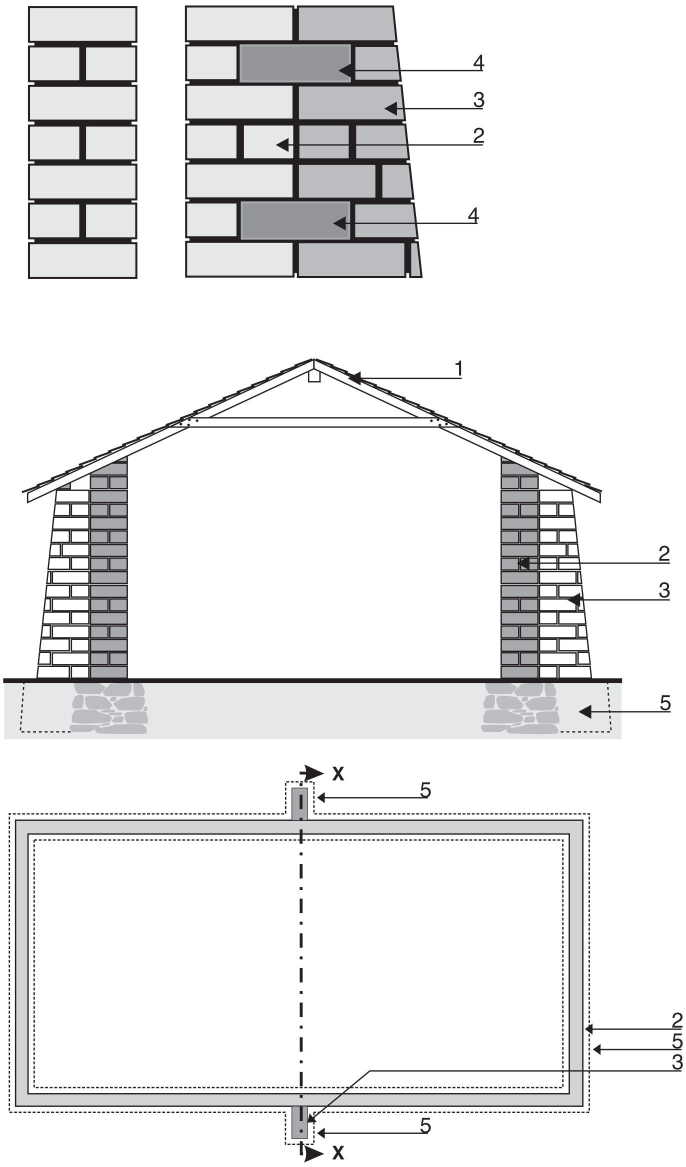
with water, apply neat cement slurry and pilaster first coat of 12 mm thickness and roughen its surface; fix the mesh with 150 mm long nails at about 450 mm apart while paster is still green; apply second coat of plaster of 16 mm thickness.

- Installation of vertical splints (with details as for the belts) at corners and junctions of walls in Category D and E buildings in addition to the seismic belts *(Fig. 57)*.
- Installation of RC bonding elements in every 1m^2 of area of random rubble and half-dressed stone walls

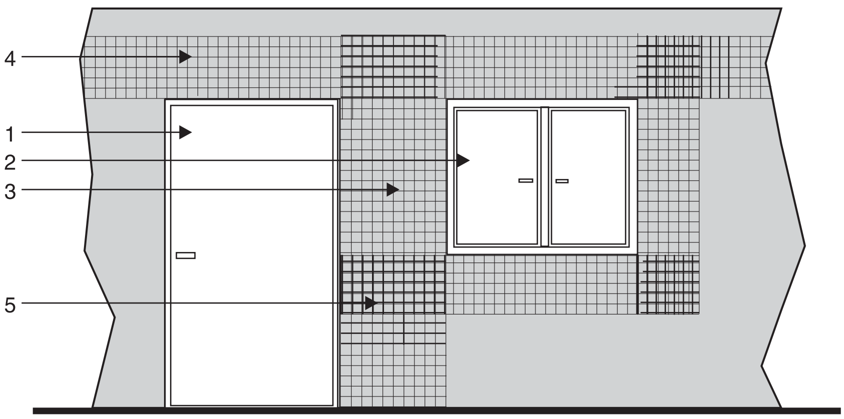
d. Strengthening R.C. Members

The strengthening of reinforced concrete members is a task that should be carried out by a structural engineer according to calculations. Here only a few suggestions are included to



Legend
1 roof with rafters
2 original wall
3 buttress, new brick work
4 'key' bricks for connecting buttresses
5 buttress footing

Fig. 54
Strengthening of long or high walls by buttresses

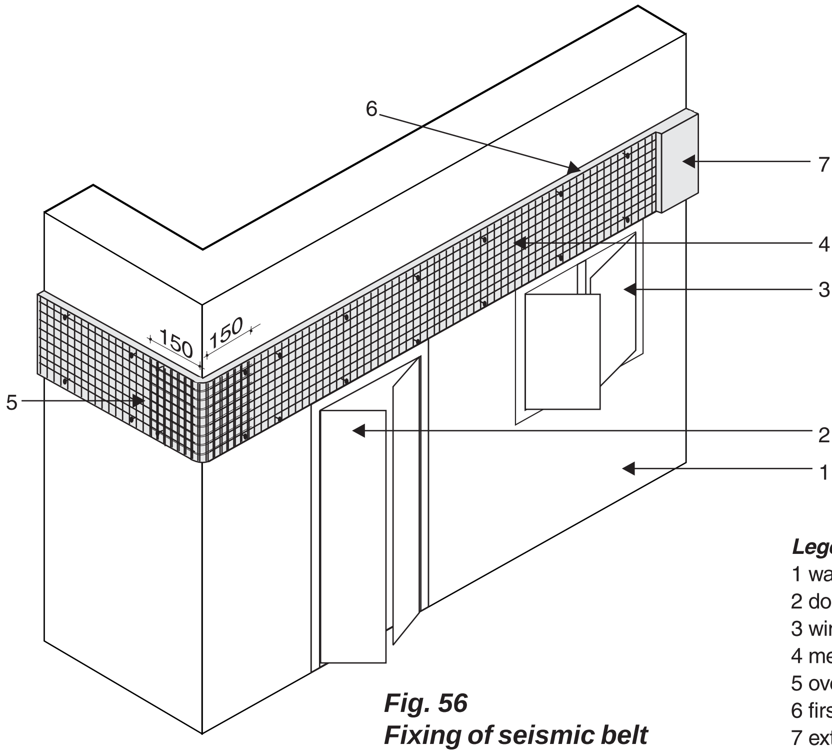


Legend
1 door
2 window
3 mesh of ferro-cement
4 seismic belt
5 overlap of mesh

Fig. 55
Reinforcing around opening

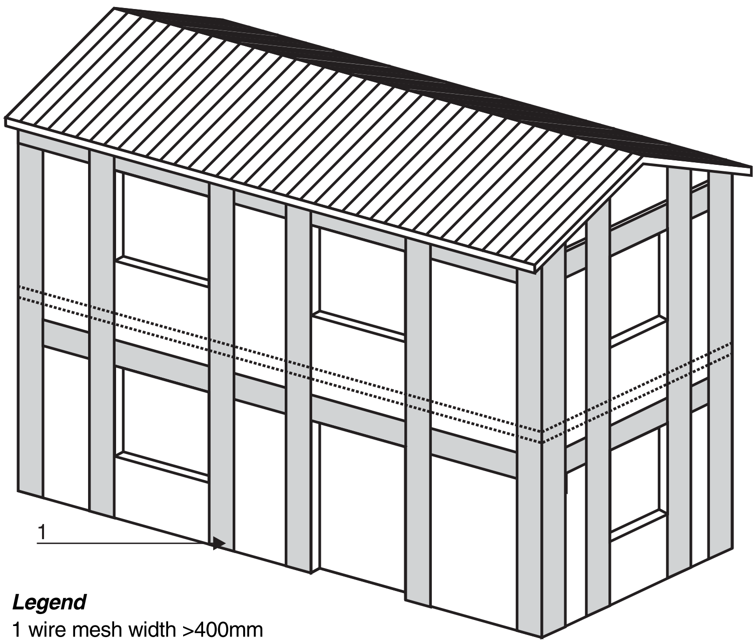
illustrate the ways in which the strengthening could be done.

- Strengthening of RC columns by jacketing, by providing additional cage of longitudinal and lateral tie reinforcement around the columns and casting a concrete ring. (see *Fig. 58*).
- Jacketing reinforced concrete beams in the above manner. For holding the stirrup in this case, holes will have to be drilled through the slab (see *Fig. 59*)



Legend
1 wall
2 door
3 window
4 mesh in seismic belt
5 overlap in mesh
6 first coat of plaster
7 external plaster, second coat

Fig. 56
Fixing of seismic belt



Legend
1 wire mesh width >400mm

Fig. 57
Splint and bandage strengthening technique

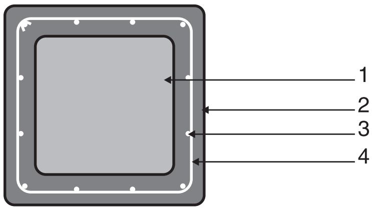
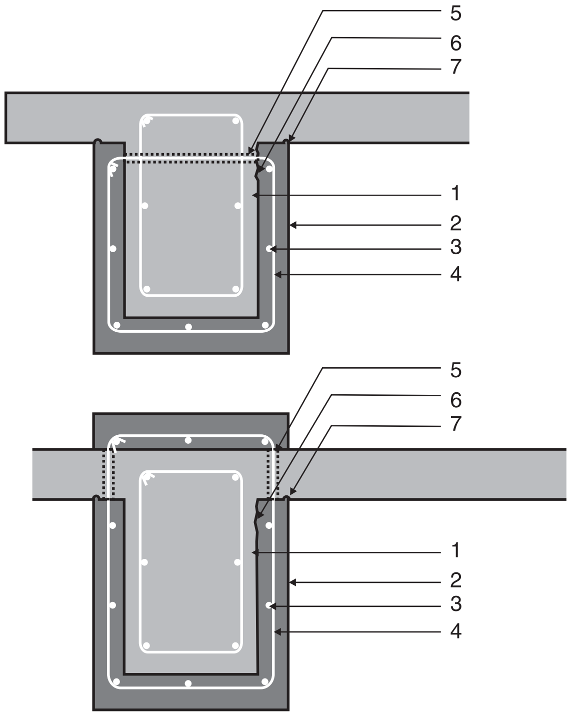


Fig. 58
Encasing a concrete column

Legend
1 Existing column section
2 Added section
3 New longitudinal bars
4 New tie bars, closely spaced



Legend
1 Old concrete
2 New concrete
3 New longitudinal bars
4 New stirrups
5 Holes for passing stirrups
6 Chip old surface
7 Groove in slab

Fig. 59
Increasing the section and reinforcement of existing beams

- Similar technique for strengthening R.C. shear walls.
- Applying prestress to RC beams so that opposite moments are caused to those applied. The wires will run on both sides of the web outside and anchored against the end of the beam through a steel plate.

Note 1: In all cases of adding new concrete to old concrete, the original surface should be roughened and groves made in the appropriate direction for providing shear transfer. The ends of the additional steel are to be anchored in the adjacent beams or columns as the case may be.

2: Demonstration of retrofitting techniques undertaken recently are shown in Fig.60 & Fig. 61.

Fig.60
A retrofitted two storey Sub-Divisional Hospital Building at Kupwara, J&K

Fig.61 Retrofitting of School Buildings

REFERENCES

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2. "IS:800-2006, Code of Practice for General Construction in Steel".

3. "IS:883-1966, Code of Practice for Design of Structural Timber in Building".

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10. "IS:13920-1993, Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces - Code of Practice".

11. "IS:13935-1993, Repair and Seismic Strengthening of Buildings- Guidelines".

12. "IS:SP6(6)- 1972, ISI Handbook for Structural Engineers".

13. "IS:SP16-1980, Design Aids for Reinforced Concrete to IS:456-1978".

MSK Intensity Scale (Extract related to buildings)

V. Awakening:

a. The earthquake is felt indoors by all, outdoors by many-Many sleeping people awake. A few run outdoors. Animals become uneasy. Buildings tremble throughout. Hanging objects swing considerably. Pictures knock against walls or swing out of place. Occasionally pendulum clocks stop. Unstable objects may be overturned or shifted........

b. Slight damages in buildings of Type A are possible.

VI. Frightening:

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

b. Damage of Grade I 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.

VII. Damage of buildings:

a. 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.

b. 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 landslips of roadway on steep slopes; cracks in roads; seams of pipelines damaged; cracks in stone walls.

VIII. Destruction of buildings:

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

b. 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, and most buildings of Type A suffer damage of Grade 4. Occasional breaking of pipe seams. Memorials and monuments move and twist. Tomb stones overturn. Stone walls collapse.

IX. General damage to buildings:

a. General panic; considerable damage to furniture. Animals run to and fro in confusion and cry.

b. Many buildings of Type C suffer damage of Grade 3, and a few of Grade 4. Many buildings of Type B show 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.

Notes:

1. Type of Buildings

Type A	*Buildings in field-stone, rural structures, unburnt-brick houses, clay houses.*
Type B	*Ordinary brick buildings, buildings of the large block and prefabricated type, half timbered structures, buildings in natural hewn stone.*
Type C	*Reinforced buildings, well built wooden structures.*

2. Definition of Quantity:

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

3. Grades of damage

G1 to G5 are described in Table 3 in the text.

Source: IS:1893-2002 Appendix D-2

CONFINED MASONRY BUILDING CONSTRUCTION *

1. UNDERSTANDING CONFINED MASONRY CONSTRUCTION

Confined masonry construction is a building technology that offers an alternative to 'Minimally reinforced masonry' with 'RC Bands and Vertical Bars' as per sections **18.5** and **18.7** and the 'RC Frame Construction'. It consists of masonry walls (made either of clay brick or concrete block units) and horizontal and vertical RC *'confining members'* built on all four sides of the masonry wall panels. Vertical members are called *'tie-columns'* or *'practical columns'* and though they resemble columns in RC frame construction but are of much smaller cross-section. Horizontal elements, called *'tie-beams'*, resemble beams in RC frame construction, but of much smaller section. It must be understood that the confining elements are not beams and columns the way these are used in RC Frames but rather in the nature of horizontal and vertical *ties* or *bands for resisting tensile stresses* and may better be termed as such.

The structural components of a building using confined masonry walls are as follows: (see Figure 1):

- *Masonry walls* – are *load bearing elements,* and transmit the gravity loading from the slab (s) and walls above down to the foundation. The walls also work as *bracing panels* acting along with the confining tie elements which resist the horizontal earthquake forces.
- *Confining elements* – horizontal and vertical *tie elements* provide the necessary tensile strength and ductility to the masonry wall panels and protect them from disintegration in the specified major earthquakes.
- *Floor and roof slabs* – transmit both vertical gravity and lateral loads to the confined masonry walls. In an earthquake the slabs behave like rigid horizontal diaphragms.
- *Plinth band or tie-beam*- will transmit the vertical and horizontal loads from the walls down to the foundation. It also protects the ground floor walls from settlement in soft soil conditions.
- *Foundation* – transmits the loads from the structure to the ground.

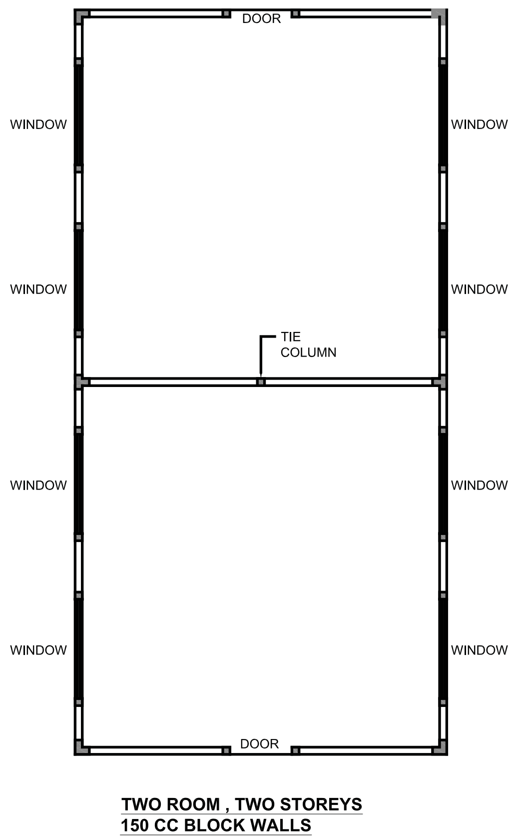

Fig. 1(a)
Typical Confined Masonry Building Plan

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* The masonry wall construction shall follow all detailed specifications given in sections **18, 19** and **20** except those to be modified as per this Annexure.

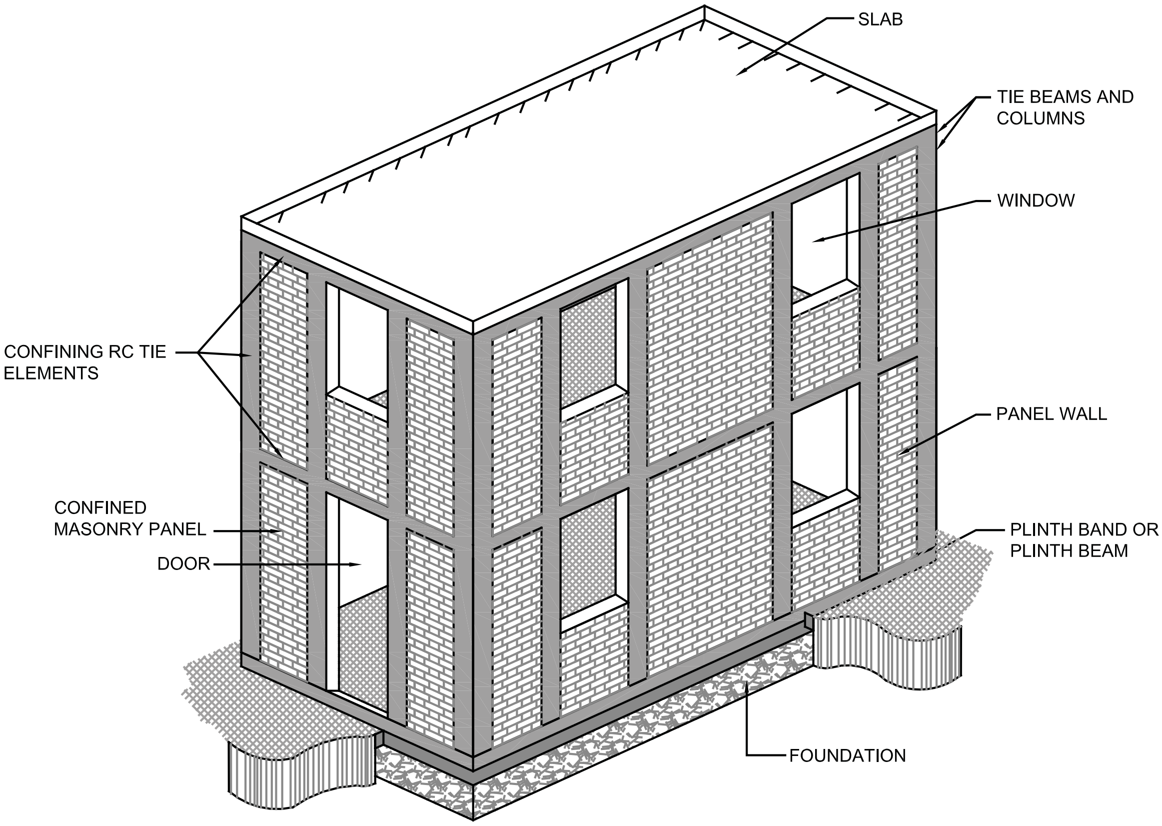
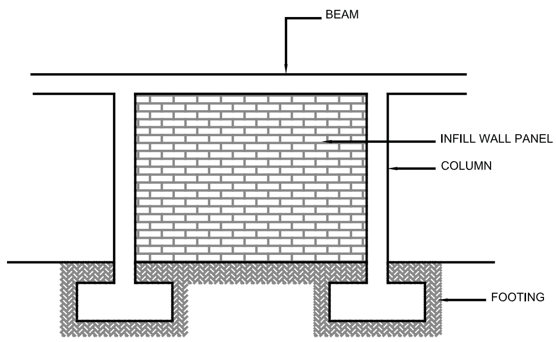


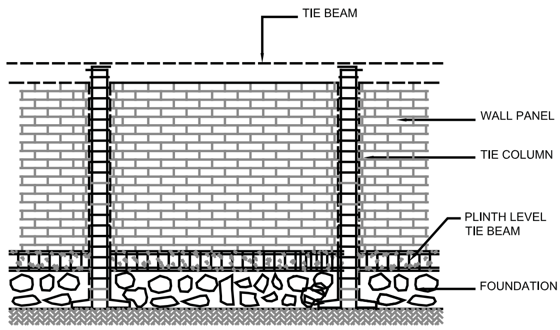
Fig. 1(b)
A Typical 2-Storey Confined Masonry Building

Confined masonry walls can be constructed using different types of masonry units, such as hollow clay tiles, burnt clay bricks, concrete blocks of hollow or solid types or dressed rectangularised stones. In confined masonry, the reinforcement is concentrated in vertical and horizontal confining elements whereas the masonry walls are usually free of reinforcement, but may be connected with the confining elements using steel dowels.

Difference between Confined Masonry and RC Frame Construction. The appearance of finished confined masonry construction and a RC frame construction with masonry infills, may look alike but in load carrying scheme they are very different. The main differences are related to the *construction sequence*, as well as to the manner in which these structures resist gravity and lateral loads. See Fig. 2. Whereas the columns and beams carry the vertical gravity as well as the lateral loads from earthquakes or wind storms *unaided* by the masonry infills, in the case of Confined masonry buildings, the



(a) RC FRAME CONSTRUCTION



(b) CONFINED MASONRY CONSTRUCTION

Fig. 2
RC Frame and Confined Masonry Construction

wall panels are the main load carrying elements (both vertical and horizontal) *aided* by the confining elements for resisting tensile forces.

2. GUIDE TO EARTHQUAKE-RESISTANT CONFINED MASONRY CONSTRUCTION

The satisfactory earthquake performance of confined masonry is due to the *joint action* of masonry walls and the reinforced concrete confining elements. Properly designed and built confined masonry buildings are expected to exhibit good performance even in Maximum Considered Earthquake (MCE) wherein, moderate cracking in the elements is not ruled out, but collapse of building will be highly improbable. Depending on the crushing strength of the building unit, confined masonry buildings may be constructed upto five storeys in height for various Building Categories **(Table 7 in Section 16)** as suggested below:

Categories **B** and **C**	upto 5 storeys
Categories **D** and **E**	upto 4 storeys

2.1 Building Configuration

The architectural configuration concepts as highlighted in **Section 12** are necessary in confined masonry construction also.

2.2 Confining Elements

The *tie-beams* should be placed at plinth and every floor level. Vertical spacing of tie-beams should not exceed 3 m. The *tie-columns* should be placed at a maximum spacing of 4 m in 200 mm or thicker walls and 3 m in 100-114 mm thick walls, as well as at the following locations:

a) at the corners of rooms and all wall-to-wall intersections,
b) at the free end of a wall,
c) at the jambs of doors / windows of 900 mm or wider openings.

2.3 Walls

The wall thickness may be kept 100 or 114 mm in the ease of one to two storey high residential buildings. But for all *important* buildings **(Section 12)** and those of more than two storeys height, the thickness should be 200 or 230 mm, and the mortar shall be as per **Section 18.3.** At least two fully confined panels should be provided in each direction of the building. The earthquake performance of a confined masonry building depends on the shear resistance of masonry walls. Therefore, it is essential to provide an adequate number of walls in each direction. The walls should be placed preferably at the periphery so as to minimize torsion of the building in an earthquake.

2.4 Wall Density

Wall density can be defined as the *total cross sectional areas of all confined wall panels in one direction* divided by *the sum of the floor plan areas of all floors* in a building. Wall density of at least 2% in each of two orthogonal directions is required to ensure good earthquake performance of confined masonry building in Seismic Zone III of India. For the aim of achieving adequate earthquake performance in the higher Seismic Zones, the wall density of 3% in Zone IV and 4.5% in Zone V should at least be used.

3. CONSTRUCTION DETAILS OF CONFINED MASONRY

3.1. Construction of walls

The aim should be to use good quality building materials, that is, the build units and the mortar; as well as good quality workmanship

- Minimum wall thickness is 100 or 114 mm. Wall panel height to thickness ratio should not exceed 30.
- Toothed edges should be left on each side of the wall; the tooth projection may be kept 40 mm to achieve full concrete filling in the teeth space. Use of horizontal dowels instead or in addition to teething could be made at the wall-to-column interface.
- Concrete is to be poured in the *tie-columns* upon completion of desirable wall height (see Fig. 2(b). *Bricks must be made wet before casting of concrete*.
- Formwork support must be provided on two sides of the wall. See Fig. 3. The concrete needs to be vibrated to fill the teeth space thoroughly.

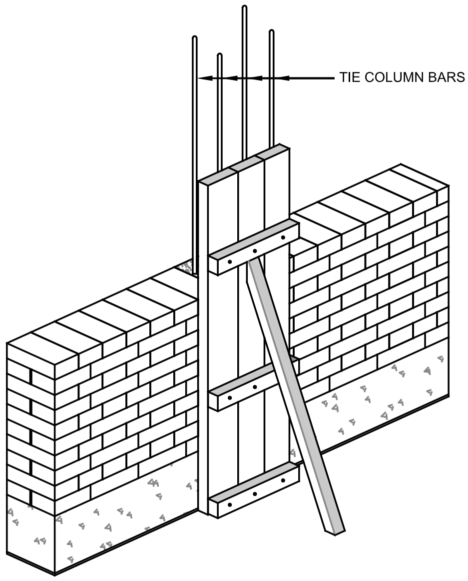


Fig. 3
Formwork for Tie Columns

3.2 Tie-columns

- The minimum *tie-column* cross sectional dimensions are 100 mm by 100 mm or 114 mm x 114 mm in 100 or 114 mm thick walls respectively and 150 x 200 or 150 x 230 in 200 or 230 mm thick walls at intermediate points but of square section of 200 or 230 mm side at the corners.
- Reinforcement for the ground storey *tie-columns* should be assembled before the foundation construction takes place. See Fig. 4.

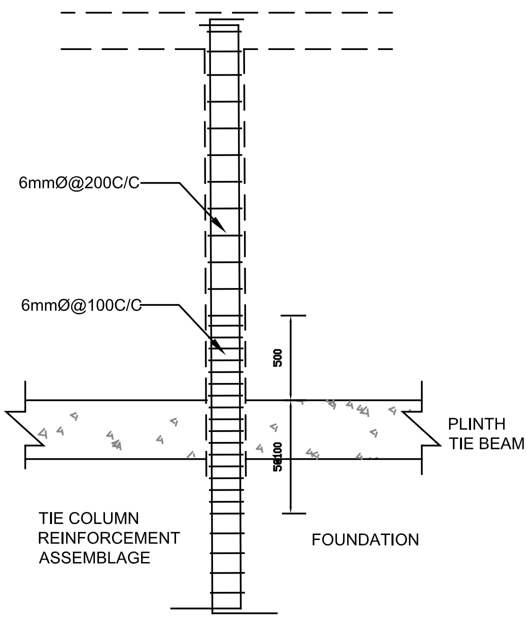


Fig. 4(a)
Tie Column Reinforcement set in position prior to the Construction of Foundation and Walls

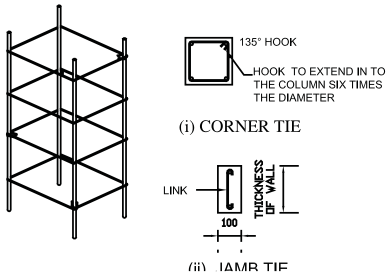


Fig. 4(b)
Reinforcement Detail of Tie Column

- The reinforcement in *tie-columns* at the corners should consist of four high strength deformed bars (HSD / TOR) for longitudinal reinforcement, and 6 mm ties at 100 mm spacing in the end 500 mm height at top and bottom of column and 200 mm apart in the remaining height. Vertical bars should be lapped by a minimum of 50 times the longitudinal bar diameter. The longitudinal bar may be kept as follows:

Categories **B** and **C**	8 mm dia upto four storeys and 10 mm dia in 5 storey building
Categories **D** and **E**	10 mm dia upto 3 storeys and 12 mm dia in 4 storeys building

- The *tie-columns* at jambs of windows / doors may be 100 mm x thickness of wall in size and have only two longitudinal bars each of same dia as in corner columns with 6 mm link ties spaced as in the corner columns.

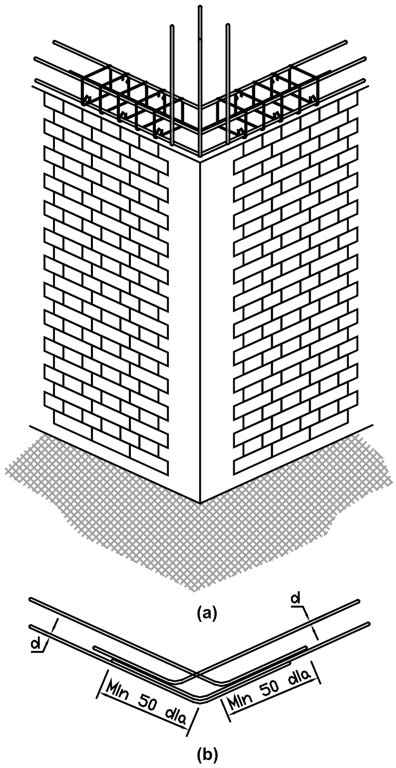


Fig. 5
Tie-Beam and Column

3.3 Tie-beams

- The minimum *tie-beam* cross-sectional dimensions are 100 x 100mm in 100 mm thick walls; 150 x 200 in 200 mm thick walls and 150 x 230 in 230 mm thick walls.
- Tie-beams are constructed on top of the walls at each floor level.
- The tie-beam reinforcement should consist of four high strength deformed bars (HSD/TOR) for longitudinal reinforcement, and 6 mm stirrups at 200 mm spacing. The tie-beam reinforcement needs to be continuous, with longitudinal reinforcement bars lapped by at least 50 times the bar diameter. The following bar diameter may be adopted:

Building Categories Band C	8 mm dia.
Building Categories D and E	10 mm dia.

- The lintel level bands may be provided as per **Section 18.5**. Very wide windows may require stronger lintels.
- Proper detailing of the *tie-beam* and its connection to *tie-column* is a must for satisfactory earthquake performance **(see Fig. 5)**.

3.4 Foundation and plinth construction

A *plinth band* should be constructed on top of the foundation (same as in **Section 18.5**). Instead of the specified band in **18.5**, a *tie beam* as specified in 3.3 above will be preferable for proper confinement of masonry panel above it.

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ABOUT BMTPC

Building Materials & Technology Promotion Council under the auspices of Ministry of Housing & Urban Poverty Alleviation is an autonomous organization dedicated to promote and popularize cost effective, eco-friendly and energy efficient building materials and disaster resistant construction technology. BMTPC works as a technology transfer council and helps various stake holders involved in the construction industry for technology development, production, mechanization, implementation, standardization, certification & evaluation, training & capacity building, certification and entrepreneur development. Over the last two decades, BMTPC has expanded its activities and made commendable efforts in the area of disaster mitigation and management.

Ever since 1991 Uttarkashi earthquake, BMTPC has been pro-actively involved not only in seismic rehabilitation but also in the area of prevention, mitigation preparedness as regards earthquake safety is concerned. The widely popularized publication of BMTPC entitled 'Vulnerability Atlas of India' is one of its kind which depicts the vulnerability of various man made constructions in different districts of India not only from earthquake hazards but also from Wind/Cyclone and Flood hazards. Efforts of BMTPC were applauded well and the Council in the process received UN Habit Award for the same. It is being BMTPC's endeavour to constantly publish guidelines, brochures, pamphlets on natural hazards so as to educate the common man and create capacities within India to handle any disaster. BMTPC has recently published the following documents:-

1. Guidelines on 'Improved Earthquake Resistance of Housing'.
2. Guidelines on 'Improved Flood Resistance of Housing'.
3. Manual on Basics of Ductile Detailing.
4. Building a Hazard Resistant House, a Common Man's Guide.
5. Manual for Restoration and Retrofitting of Buildings in Uttarakhand & Himachal Pradesh.

These documents are important tools for safety against natural hazards for various stake holders involved in disaster management. Apart from publications, the council is also involved in construction of disaster resistant model houses and retrofitting of existing life line buildings such as Schools/Hospitals to showcase different disaster resistant technologies and also spread awareness amongst artisans and professionals regarding retrofitting and disaster resistant construction.

BMTPC joined hands with Ministry of Home Affairs to draft Building Bye-laws incorporating disaster resistance features so that State/UT Governments prepare themselves against natural hazards. One of the very basic publications of BMTPC with IIT, Kanpur has been 'Earthquake Tips' which were specially designed and published to spread awareness regarding earthquake amongst citizens of India in a simple, easy to comprehend language. The tips are being published in other languages also so that there is greater advocacy and public out reach regarding earthquake safety.

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