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INDIAN RAILWAYS
CENTRE FOR ADVANCED MAINTENANCE TECHNOLOGY

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Handbook on

CONSTRUCTION OF EARTHQUAKE RESISTANT BUILDINGS
भूकंप प्रतिरोधी भवन निर्माण

पर हस्त पुस्तिका



अअमा सं RDSO
रेल अग्रदूत Transforming Railways

EXCELLENCE IN MAINTENANCE

केमटेक - महाराजपुर, ग्वालियर - ४७४००५
CAMTECH - Maharajpur, Gwalior - 474005

गुणवत्ता नीति

रेलों में यात्री और माल यातायात की बढ़ती माँग को पूरा करने के लिए गुणवत्ता प्रबंध प्रणाली में अनुसंधान, डिज़ाइनों और मानकों में उत्कृष्टता तथा सतत सुधारों के माध्यम से सांविधिक और नियामक अपेक्षाओं को पूरा करते हुए सुरक्षित, आधुनिक और किफ़ायती रेल प्रौद्योगिकी का विकास करना।



QUALITY POLICY

To develop safe, modern and cost effective Railway technology complying with Statutory and Regulatory requirements, through excellence in Research, Designs & Standards and Continual improvements in Quality Management System to cater to growing demand of passenger and freight traffic on the Railways.

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Handbook on

CONSTRUCTION OF EARTHQUAKE RESISTANT BUILDINGS

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पर हस्त पुस्तिका



Concept of Future SURAT Railway Station

अग्रगामी RDSO
रेल अग्रदूत Transforming Railways

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केमटेक - महाराजपुर, ग्वालियर - ४७४००५
CAMTECH - Maharajpur, Gwalior - 474005

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A.K. MITTAL

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FOREWORD

It is very heartening to know that a Handbook on "Construction of Earthquake Resistant Buildings" is being brought out by CAMTECH Directorate, Gwalior under the aegis of RDSO. The complex theory and practical issues related to Earthquake magnitude, its measurement, earthquake resistant design, preferred building layout, and design based on Earthquake spectrum analysis has been presented in a lucid and informative manner for adoption in the field by Civil Engineers. Solved examples have also been included, illustrating calculation of design forces in structural member for multi-storied building. Provisions contained in Seismic Code IS: 1893 & others have been brought out related to building layout, seismic forces calculation and reinforcement detailing.

Details of retro-fitment for buildings have been also included, which can adopted by serving Engineers to make buildings Earthquake Resistant in an effective and economical manner.

I congratulate ADG and ED/Works of RDSO for editing & Civil Engineers of CAMTECH for compilation of very informative Handbook.

New Delhi,
20th July, 2017

(A.K. Mittal)

जोगेश सिंह सोन्धी
अपर महानिदेशक
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F o r e w o r d

It is indeed very heartening to know that CAMTECH under the direction from RDSO has brought out a Handbook on “Construction of Earthquake Resistant Buildings”.

It is also worth mentioning that on IR, there were no comprehensive Guidelines or instructions regarding construction of Earthquake Resistant Buildings. This handbook shall bridge the gap & provide technical information on Earthquake phenomenon, assessment of magnitude of earthquake, general principles for earthquake resistance in Building-layout, dynamic response of Buildings.

Codal based procedure for determining lateral earthquake forces with special reference to ‘Ductility & Capacity Design Concepts’ has been brought out. Solved examples, illustrate calculation of design forces for structural members of multi-storied building. Provisions contained in Seismic Code IS: 1893 & others have been brought out related to buildings which shall help structural designers and project engineers. Chapter on Seismic Evaluation and Retrofitting gives in-sight to serving Engineers in the field to assess building for earthquake resistance, and action required thereof in economical manner.

Thanks are due to Dr. S.K. Thakkar, Professor (Retd.), IIT/Roorkee for technical review of this Handbook. I congratulate Works & Bridge Dte. of RDSO for editing, and Sh. D.K. Gupta, Jt. Director /Civil of CAMTECH involved in compilation of this Handbook, for their praise worthy efforts.

(J. S. Sondhi)
Addl. Director General
RDSO

Dt. 20.07.2017

प्राक्कथन

दुनिया के कई हिस्सों में हाल ही में आए भूकंपों ने इमारतों और जीवन को काफी नुकसान पहुंचाया है। भूकंप की दृष्टि से देखा जाए तो सबसे खतरनाक भवन निर्माण, unreinforced ईंट या concrete ब्लॉक का होता है। चार मंजिलों तक के अधिकांश घरों को प्रबलित कंक्रीट स्लैब के साथ burnt clay ईंट चिनाई से निर्मित किया जा रहा है। इसी तरह, कई नए चार या पांच मंजिला घर जो कि छोटे और बड़े शहरों में प्रबलित कंक्रीट फ्रेम से बनाए गए हैं, में एक उचित फ्रेम प्रणाली की कमी रहती है।

हाल ही में आए भूकंपों के कारण, भारत में इमारतों और घरों को कैसे सुरक्षित रखा जाये, इस पर प्रमुखता से चर्चा हुई है। भूकंपीय देशों में इंजीनियर्स को यह महत्वपूर्ण जिम्मेदारी सुनिश्चित करना है कि नए निर्माण भूकंप प्रतिरोधी हों और यह भी कि उन्हें मौजूदा कमजोर संरचनाओं द्वारा उत्पन्न समस्या का समाधान भी निकालना है।

यह आशा की जाती है कि कैमटेक द्वारा तैयार पुस्तिका सिविल संरचनाओं के निर्माण एवं रखरखाव की गतिविधियों में लगे भारतीय रेलवे के इंजीनियरिंग कर्मियों के लिए काफी मददगार होगी।

केमटेक/ ग्वालियर ***(ए. आर. तुपे)***

23 मई, 2017 ***कार्यकारी निदेशक***

FOREWORD

The recent earthquakes occurred in many parts of world has caused considerable damage to the buildings and lives. The most dangerous building construction, from an earthquake point of view, is unreinforced brick or concrete block. Most houses of upto four storeys are built of burnt clay brick masonry with reinforced concrete slabs. Similarly, many new four or five storey reinforced concrete frame building being constructed in small and large towns lack a proper frame system.

With the recent earthquakes, the discussion on how safe buildings and houses are in India has gained prominence. Engineers in seismic countries have the important responsibility to ensure that the new construction is earthquake resistant and also, they must solve the problem posed by existing weak structures.

It is expected that the handbook prepared by CAMTECH will be quite helpful to the engineering personnel of Indian Railways engaged in construction and maintenance activities of civil structures.

CAMTECH/Gwalior ***(A.R. Tupe)***
23 May, 2017 ***Executive Director***

भूमिका

भारतीय रेलवे एक बड़ा संगठन है जिसके पास सिविल इंजीनियरिंग संरचनाओं एवं भवनों की विशाल संपदा मौजूद है। भूकंप की विनाशकारी प्रकृति को ध्यान में रखते हुए यह आवश्यक है कि लगभग सभी भवनों चाहे वे आवासीय, संस्थागत, शैक्षणिक इत्यादि के हों उनकी योजना, डिजाइन, निर्माण तथा रखरखाव भूकंप प्रतिरोधी तरीकों को अपनाकर किया जाना चाहिए, जिससे कि भूकंप के कारण मानव जीवन व संपत्ति के नुकसान को न्यूनतम किया जा सके।

"भूकंप प्रतिरोधी भवनों के निर्माण" पर यह हस्तपुस्तिका, एक जगह पर पर्याप्त सामग्री प्रदान करने का एक प्रयास है ताकि व्यक्ति भवनों के भूकंप प्रतिरोधी निर्माण के लिए मूलभूत सिद्धांतों को विकसित कर सही तथा व्यवहारिक कार्यविधि को अमल में ला सके।

इस हस्तपुस्तिका की सामग्री को ग्यारह अध्यायों में विभाजित किया गया है; **अध्याय-1** परिचय तथा **अध्याय-2** भूकंप इंजीनियरिंग में प्रयुक्त शब्दावली परिभाषित करता है। **अध्याय-3** भूकंप व भूकंपी खतरों के बारे में बुनियादी ज्ञान को संक्षेप में वर्णित करता है। **अध्याय- 4** भूकंप परिमाण तथा तीव्रता के माप के साथ भारत के भूकंपीय ज़ोन मानचित्र, भूकंप की निगरानी के लिए एजेंसियों के बारे में जानकारी प्रदान करता है। **अध्याय-5 व 6** भवन लेआउट में भूकंप प्रतिरोध के सुधार के लिए व्यापक सिद्धांत को बताता है। **अध्याय-7** भवन की गतिशील प्रतिक्रिया को दर्शाता है। **अध्याय-8 और 9** में कोड पर आधारित पार्श्व बल निर्धारण का तरीका तथा बहुमंजिला भवन की "डक्टाइल डिटेलिंग तथा केपेसिटी डिजाइन" को ध्यान में रखते हुए डिजाइन का उदाहरण प्रस्तुत किया गया है। **अध्याय-10** में कम शक्ति की चिनाई द्वारा संरचनाओं के निर्माण को भूकंप प्रतिरोधी सिद्धांतों को ध्यान में रख वर्णित किया गया है। **अध्याय -11** में मौजूदा भवनों की भूकंप प्रतिरोधी आवश्यकताओं को पूरा करने के लिए भवनों के मौजूदा भूकंपरोधी मूल्यांकन और पुनः संयोजन पर प्रकाश डाला गया है।

यह हस्तपुस्तिका मुख्यतः भारतीय रेल के फील्ड तथा डिजाइन कार्यालय में कार्यरत जेई/एसएसई स्तर के लिए है। इस हस्तपुस्तिका को भारतीय रेल के सिविल इंजीनियर्स तथा अन्य विभागों के इंजीनियर्स द्वारा एक संदर्भ पुस्तिका के रूप में भी इस्तेमाल किया जा सकता है।

मैं, श्री एस. के. ठक्कर, प्रोफेसर (रिटायर्ड), आई. आई. टी. रुड़की, को उनके द्वारा दिये गए मार्गदर्शन तथा सुझावों के लिए अत्यन्त आभारी हूँ तथा श्री के. सी. शाक्य, एसएसई/सिविल को इस हस्तपुस्तिका के संकलन में उनके समर्पित सहयोग के लिए धन्यवाद देता हूँ।

यद्यपि इस हस्तपुस्तिका को तैयार करने में हर तरह की सावधानी बरती गई है, फिर भी कोई त्रुटि या चूक हो तो कृपया IRCAMTECH/Gwalior की जानकारी में लायी जा सकती है।

भारतीय रेल के सभी अधिकारियों और इकाइयों द्वारा पुस्तक की सामग्री में विस्तार तथा सुधार के लिए दिये जाने वाले सुझावों का स्वागत है।

केमटेक/ ग्वालियर ***(डी. के. गुप्ता)***

23 मई, 2017 ***संयुक्त निदेशक/सिविल***

PREFACE

Indian Railways is a big organisation having large assets of Civil Engineering Structures and Buildings. Keeping in mind the destructive nature of Earthquake, it is essential that almost all buildings whether residential, institutional, educational, assembly etc. should be planned, designed, constructed as well as maintained by adopting Earthquake Resistant features; so that loss due to earthquake to human lives and properties can be minimised.

This handbook on **"Construction of Earthquake Resistant Buildings"** is an attempt to provide enough material at one place for individual to develop the basic concept for correctly interpreting and using practices for earthquake resistant construction of Buildings.

Content of this handbook is divided into Eleven Chapters; **Chapter-1** is Introduction and **Chapter-2** defines Terminology frequently used in Earthquake Engineering. **Chapter-3** describes in brief Basic knowledge about Earthquake & Seismic Hazards. **Chapter-4** deals with Measurement of Earthquake magnitude & intensity with information about Seismic Zoning Map of India and Agencies for Earthquake monitoring. **Chapter-5 & 6** elaborates General Principle for improving Earthquake resistance in building layouts. **Chapter-7** features Dynamic Response of Building. In **Chapter-8 & 9** Codal based procedure for determining lateral loads and Design of multi-storeyed building with solved example considering Ductile Detailing and Capacity Design Concept is covered. **Chapter-10** describes Construction of Low strength Masonry Structure considering earthquake resistant aspect. **Chapter-11** enlighten "Seismic Evaluation & Retrofitting" for structural upgrading of existing buildings to meet the seismic requirements.

This handbook is primarily written for JE/SSE level over Indian Railways working in Field and Design office. This handbook can also be used as a reference book by Civil Engineers and Engineers of other departments of Indian Railways.

I sincerely acknowledge the valuable guidance & suggestion by Shri S.K. Thakkar, Professor (Retd.), IIT Roorkee and also thankful to Shri K.C. Shakya, SSE/Civil for his dedicated cooperation in compilation of this handbook.

Though every care has been taken in preparing this handbook, any error or omission may please be brought out to the notice of IRCAMTECH/Gwalior.

Suggestion for addition and improvement in the contents from all officers & units of Indian Railways are most welcome.

CAMTECH/Gwalior ***(D.K. Gupta)***
23 May, 2017 ***Joint Director/Civil***

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अध्याय / Chapter – 1

परिचय / Introduction

To avoid a great earthquake disaster with its severe consequences, special consideration must be given. Engineers in seismic countries have the important responsibility to ensure that the new construction is earthquake resistant and also, they must solve the problem posed by existing weak structures.

Most of the loss of life in past earthquakes has occurred due to the collapse of buildings, constructed with traditional materials like stone, brick, adobe (kachcha house) and wood, which were not particularly engineered to be earthquake resistant. In view of the continued use of such buildings, it is essential to introduce earthquake resistance features in their construction.

The problem of earthquake engineering can be divided into two parts, first to design new structures to perform satisfactorily during an earthquake and second to retrofit existing structures so as to reduce the loss of life during an earthquake. Every city in the world has a significant proportion of existing unsafe buildings which will produce a disaster in the event of a strong ground shaking. Engineers have the responsibility to develop appropriate methods of retrofit which can be applied when the occasion arises.

The design of new building to withstand ground shaking is prime responsibility of engineers and much progress has been made during the past 40 years. Many advances have been made such as the design of ductile reinforced concrete members. Methods of base isolation and methods of increasing the damping in structures are now being utilized for important buildings, both new and existing. Improvements in seismic design are continuing to be made such as permitting safe inelastic deformations in the event of very strong ground shaking.

A problem that the engineer must share with the seismologist/geologist is that of prediction of future occurrence of earthquake, which is not possible in current scenario.

Earthquake resistant construction requires seismic considerations at all stages; from architectural planning to structural design to actual constructions and quality control.

Problems pertaining to Earthquake engineering in a seismic country cannot be solved in a short time, so engineers must be prepared to continue working to improve public safety during earthquake. In time, they must control the performance of structures so that effect of earthquake does not create panic in society and its after effects are easily restorable.

To ensure seismic resistant construction, earthquake engineering knowledge needs to spread to a broad spectrum of professional engineers within the country, rather than confining it to a few organizations or individuals as if it were a super-speciality.

***

अध्याय / Chapter – 2

भूकंप इंजीनियरिंग के लिए शब्दावली / Terminology for Earthquake Engineering

2.1 फोकस या हाइपोसेंटर / Focus or Hypocenter

In an earthquake the waves emanate from a finite area of rocks. However, the point from which the waves first emanate or where the fault movement starts is called the earthquake focus or hypocenter.

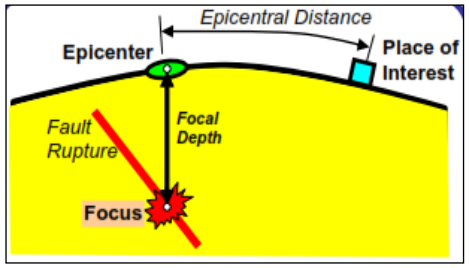


Fig. 2.1Basic terminology

2.2 इपीसेंटर / Epicentre

The point on the ground surface just above the focus is called the epicentre.

2.3 सतही फोकस भूकंप / Shallow Focus Earthquake

Shallow focus earthquake occurs where the focus is less than 70 km deep from ground surface.

2.4 इंटरमीडिएट फोकस भूकंप / Intermediate Focus Earthquake

Intermediate focus earthquake occurs where the focus is between 70 km to 300 km deep.

2.5 गहरा फोकस भूकंप / Deep Focus Earthquake

Deep focus earthquake occurs where the depth of focus is more than 300 km.

2.6 इपीसेंटर दूरी / Epicentre Distance

Distance between epicentre and recording station in km or in degrees is called epicentre distance.

2.7 पूर्व के झटके / Foreshocks

Fore shocks are smaller earthquakes that precede the main earthquake.

2.8 बाद के झटके / Aftershocks

Aftershocks are smaller earthquakes that follow the main earthquake.

2.9 परिमाण / Magnitude

The magnitude of earthquake is a number, which is a measure of energy released in an earthquake. It is defined as logarithm to the base 10 of the maximum trace amplitude, expressed in microns, which the standard short-period torsion seismometer (with a period of 0.8s,

magnification 2800 and damping nearly critical) would register due to the earthquake at an epicentral distance of 100 km.

2.10 तीव्रता / Intensity

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

2.11 परिमाण और तीव्रता के बीच बुनियादी फर्क / Basic difference between Magnitude and Intensity

Magnitude of an earthquake is a measure of its size, whereas intensity is an indicator of the severity of shaking generated at a given location. Clearly, the severity of shaking is much higher near the epicenter than farther away.

This can be elaborated by considering the analogy of an electric bulb. Here, the size of the bulb (*100-Watt*) is like the magnitude of an earthquake (M), and the illumination (measured in *lumens*) at a location like the intensity of shaking at that location (Fig. 2.2).

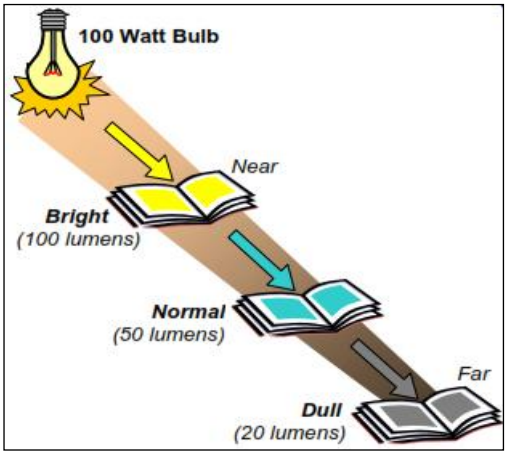


Fig. 2.2 Reducing illumination with distance from an electric bulb

2.12 द्रवण / Liquefaction

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

2.13 विवर्तनिक लक्षण / Tectonic Feature

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

2.14 भूकंपी द्रव्यमान / Seismic Mass

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

2.15 भूकंपी भार / Seismic Weight

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

2.16 आधार / Base

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

2.17 द्रव्यमान का केंद्र / Centre of Mass

The point, through which the resultant of the masses of a system acts, is called Centre of Mass. This point corresponds to the centre of gravity of masses of system.

2.18 कठोरता का केंद्र / Centre of Stiffness

The point, through which the resultant of the restoring forces of a system acts, is called Centre of stiffness.

2.19 बॉक्स प्रणाली / Box System

Box is a bearing wall structure without a space frame, where the horizontal forces are resisted by the walls acting as shear walls.

2.20 पट्टा / Band

A reinforced concrete, reinforced brick or wooden runner provided horizontally in the walls to tie them together, and to impart horizontal bending strength in them.

2.21 लचीलापन / Ductility

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

2.22 कतरनी दीवार / Shear Wall

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

2.23 तन्य का ब्यौरा / Ductile Detailing

Ductile Detailing is the preferred choice of location and amount of reinforcement in reinforced concrete structures to provide adequate ductility. In steel structures, it is the design of members and their connections to make them adequate ductile.

2.24 लचीला भूकंपी त्वरण गुणांक / Elastic Seismic Acceleration Co-Efficient *A*

This is the horizontal acceleration value, as a fraction of acceleration due to gravity, versus natural period of vibration *T* that shall be used in design of structures.

2.25 प्राकृतिक अवधि / Natural Period *T*

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

a) Fundamental Natural Period T_1: It is the highest modal time period of vibration along the considered direction of earthquake motion.
b) Modal Natural Period T_k: Modal natural period of mode *k* is the time period of vibration in mode *k*.

2.26 नॉर्मल मोड / Normal Mode

Mode of vibration at which all the masses in a structure attain maximum values of displacements and rotations, and also pass through equilibrium positions simultaneously.

2.27 ओवरस्ट्रेंग्थ / Overstrength

Strength considering all factors that may cause its increase e.g., steel strength being higher than the specified characteristic strength, effect of strain hardening in steel with large strains, and concrete strength being higher than specified characteristic value.

2.28 रिस्पांस कमी कारक/ Response Reduction Factor *R*

The factor by which the actual lateral force that would be generated, if the structure were to remain elastic during the most severe shaking that is likely at that site, shall be reduced to obtain the design lateral force.

2.29 रिस्पांस स्पेक्ट्रम / Response Spectrum

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

2.30 मिट्टी प्रोफ़ाइल फैक्टर / Soil Profile Factor *S*

A factor used to obtain the elastic acceleration spectrum depending on the soil profile below the foundation of structure.

***

अध्याय / Chapter – 3

भूकंप के बारे में / About Earthquake

3.1 भूकंप / Earthquake

- Vibrations of earth's surface caused by waves coming from a source of disturbance inside the earth are described as earthquakes.
- Earthquake is a natural phenomenon occurring with all uncertainties.
- During the earthquake, ground motions occur in a random fashion, both horizontally and vertically, in all directions radiating from epicentre.
- These cause structures to vibrate and induce inertia forces on them.

3.2 किन कारणों से होता है भूकंप / What causes Earthquake

Earthquakes may be caused by

- Tectonic activity
- Volcanic activity
- Land-slides and rock-falls
- Rock bursting in a mine
- Nuclear explosions

3.3 विवर्तनिक गतिविधि / Tectonic Activity

Tectonic activity pertains to geological formation of the bedrock in the earth's crust characterized by structural features, such as dislocation, distortion, faults, folding, thrusts, volcanoes directly involved in the earth movement.

As engineers we are interested in earthquakes that are large enough and close enough (to the structure) to cause concern for structural safety- usually caused by tectonic activity.

Earth (Fig. 3.1) consists of following segments –

- solid inner core (radius ~1290*km*) that consists of heavy metals (*e.g.*, nickel and iron)
- liquid outer core(thickness ~2200*km*)
- stiffer mantle(thickness ~2900*km*) that has ability to flow and
- crust(thickness ~5 to 40*km*) that consists of light materials (*e.g.*, basalts and granites)

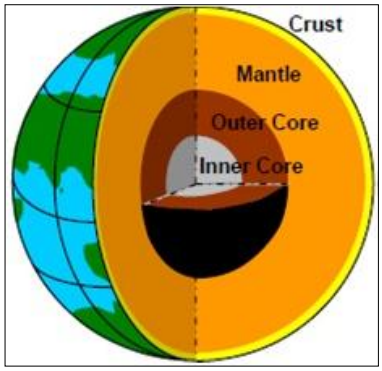


Fig. 3.1 Inside the Earth

At the Core, the temperature is estimated to be ~2500°*C*, the pressure ~4 million *atmospheres* and density ~13.5 *gm/cc*; this is in contrast to ~25°*C*, 1 *atmosphere* and 1.5 *gm/cc* on the surface of the Earth.

Due to prevailing high temperature and pressure gradients between the Crust and the Core, the local convective currents in mantle (Fig. 3.2) are developed. These convection currents result in a *circulation* of the earth's mass; hot molten lava comes out and the cold rock mass goes into the Earth. The mass absorbed eventually melts under high temperature and pressure and becomes a part of the Mantle, only to come out again from another location.

Near the bottom of the crust, horizontal component currents impose shear stresses on bottom of crust, causing movement of plates on earth's surface. The movement causes the plates to move apart in some places and to converge in others.

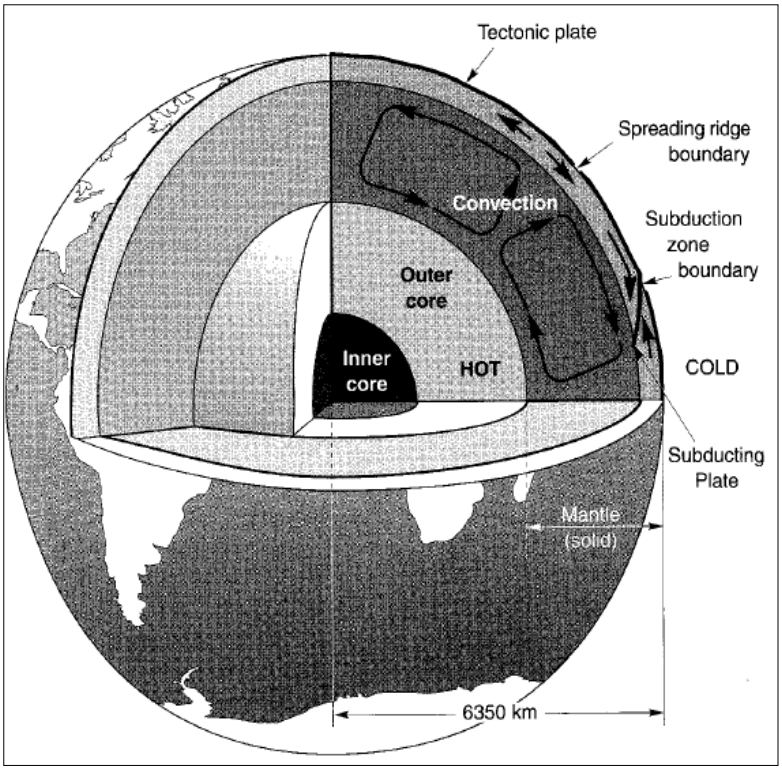


Fig 3.2 Convention current in mantle

3.4 विवर्तनिक प्लेट का सिद्धांत / Theory of Plate Tectonics

Tectonic Plates: Basic hypothesis of plate tectonics is that the earth's surface consists of a number of large, intact blocks called *plates* or *tectonic plates* and these plates move with respect to each other due to the convective flows of Mantle material, which causes the Crust and some portion of the Mantle, to slide on the hot molten outer core. The major plates are shown in Fig. 3.3.

The earth's crust is divided into six continental-sized plates (African, American, Antarctic, Australia-Indian, Eurasian, and Pacific) and about 14 of sub-continental size (e.g., Carribean, Cocos, Nazca, Philippine, etc.). Smaller *platelets* or *micro-plates also* have broken off from the larger plates in the vicinity of many of the major plate boundaries.

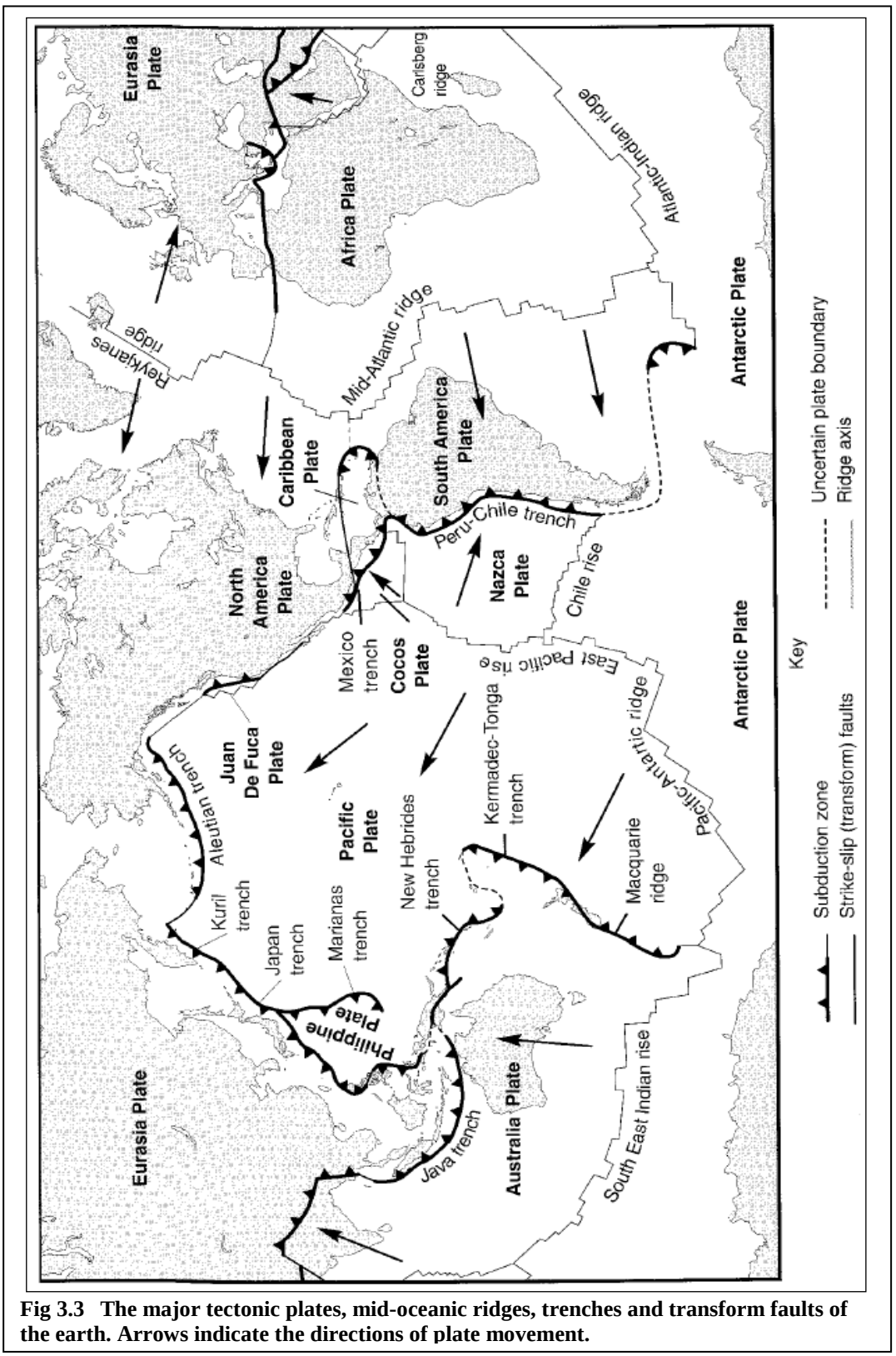


Fig 3.3 The major tectonic plates, mid-oceanic ridges, trenches and transform faults of the earth. Arrows indicate the directions of plate movement.

The relative deformation between plates occurs only in narrow zones near their boundaries. These deformations are:

1. *Aseismic deformation*: This deformation of the plates occurs slowly and continuously.

2. *Seismic deformation*: This deformation occurs with sudden outburst of energy in the form of earthquakes.

The boundaries are: (i) Convergent (ii) Divergent (iii) Transform

Convergent boundary: Sometimes, the plate in the front is slower. Then, the plate behind it comes and collides (and *mountains* are formed). This type of inter-plate interaction is the *convergent* boundary (Fig. 3.4).

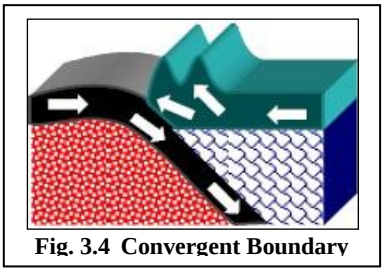


Fig. 3.4 Convergent Boundary

Divergent boundary: Sometimes, two plates move away from one another (and *rifts* are created). This type of inter-plate interaction is the divergent boundary (Fig. 3.5).

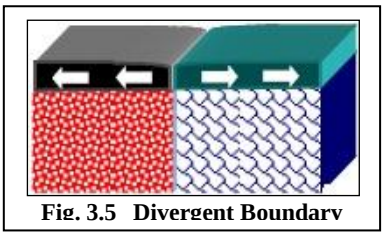


Fig. 3.5 Divergent Boundary

Transform boundary: Sometimes, two plates move side-by-side, along the same direction or in opposite directions. This type of inter-plate interaction is the transform boundary (Fig. 3.6).

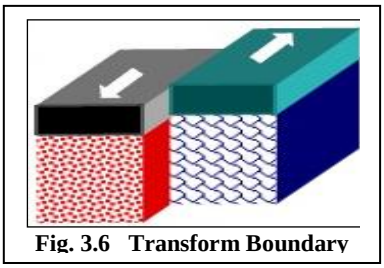


Fig. 3.6 Transform Boundary

Since the deformation occurs predominantly at the boundaries between the plates, it would be expected that the locations of earthquakes would be concentrated near plate boundaries. The map of earthquake epicentres shown in Fig. 3.7 provides strong support to confirm the theory of plate tectonics. The dots represent the epicentres of significant earthquakes. It is apparent that the locations of the great majority of earthquakes correspond to the boundaries between plates.

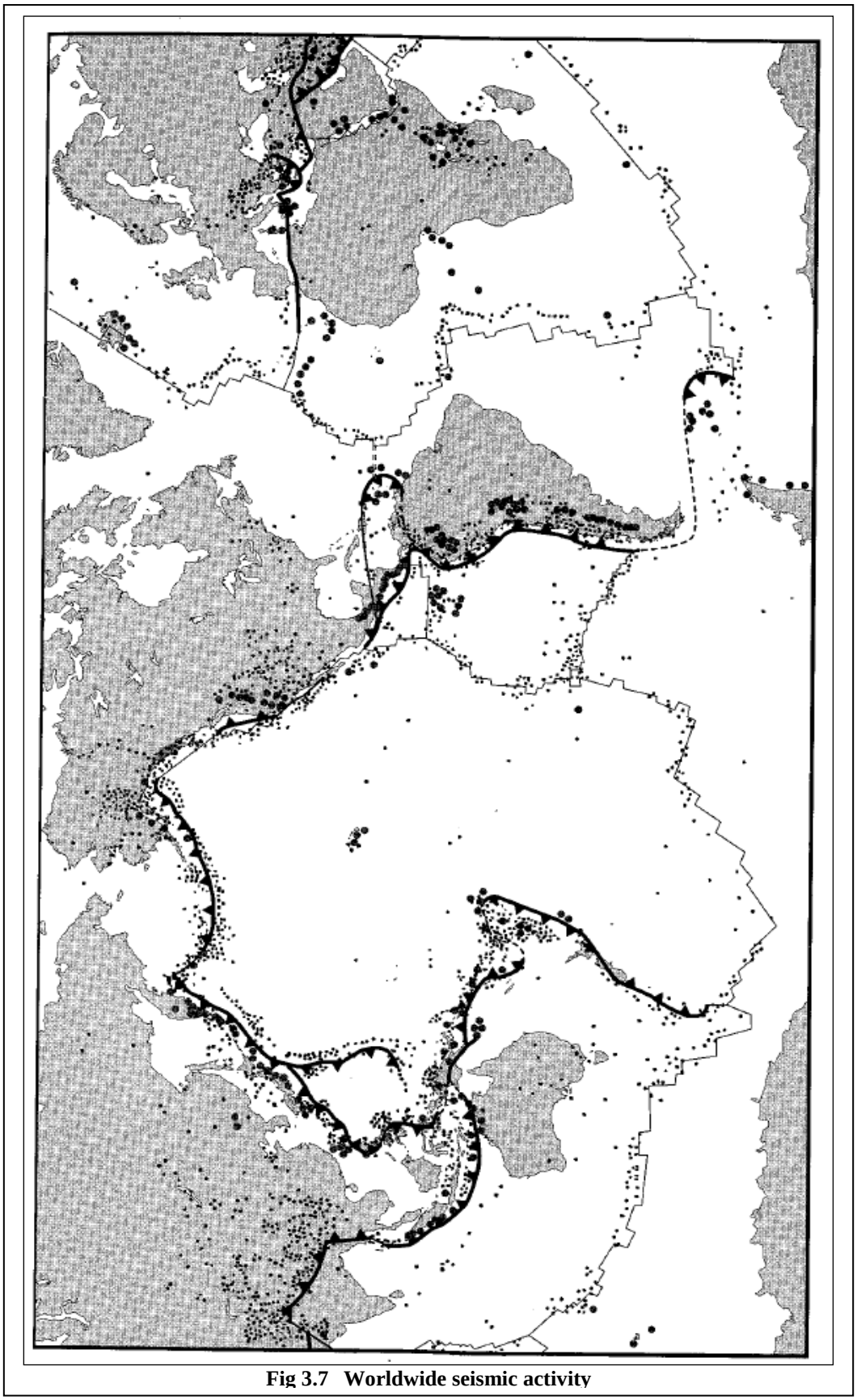


Fig 3.7 Worldwide seismic activity

3.5 लचीला रिबाउंड सिद्धांत / Elastic Rebound Theory

Earth crust for some reason is moving in opposite directions on certain faults. This sets up elastic strains in the rocks in the region near this fault. As the motion goes on, the stresses build up in the rocks until the stresses are large enough to cause slip between the two adjoining portions of rocks on either side. A rupture takes place and the strained rock rebounds back due to internal stress. Thus the strain energy in the rock is relieved partly or fully (Fig. 3.8).

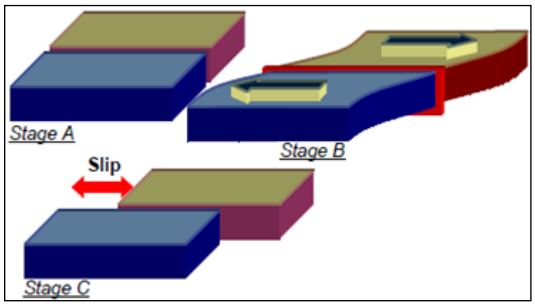


Fig. 3.8 Elastic Strain Build-Up and Brittle Rupture

Fault: The interface between the plates where the movement has taken place is called fault.

Slip: When the rocky material along the interface of the plates in the Earth's Crust reaches its strength, it fractures and a sudden movement called slip takes place.

The sudden slip at the fault causes the earthquake. A violent shaking of the Earth during which large elastic strain energy released spreads out in the form of seismic waves that travel through the body and along the surface of the Earth.

After elastic rebound there is a readjustment and reapportion of the remaining strains in the region. The stress grows on a section of fault until slip occurs again; this causes yet another even though smaller earthquake which is termed as *aftershock.* The aftershock activity continues until the stresses are below the threshold level everywhere in the rock.

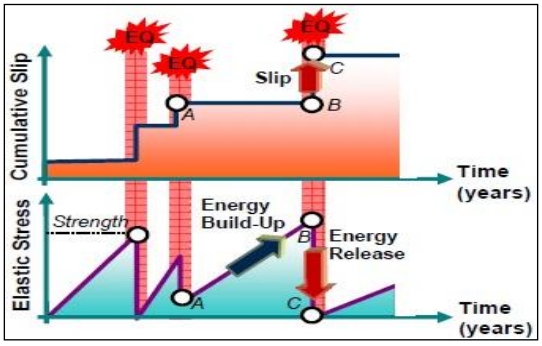


Fig. 3.9 Elastic Rebound Theory

After the earthquake is over, the process of strain build-up at this modified interface between the tectonic plates starts all over again. This is known as the *Elastic Rebound Theory* (Fig. 3.9).

3.6 भूकंप और दोष के प्रकार / Types of Earthquakes and Faults

Inter-plate Earthquakes: Most earthquakes occurring along the boundaries of the tectonic plates are called Inter-plate Earthquakes, (*e.g.*, 1897 Assam (India) earthquake).

Intra-plate Earthquakes: Numbers of earthquakes occurring within the plate itself but away from the plate boundaries are called Intra-plate Earthquakes, (*e.g.*, 1993 Latur (India) earthquake).

Note: *In both types of earthquakes, the slip generated at the fault during earthquakes is along*

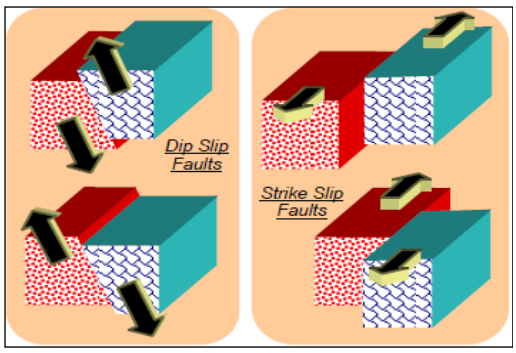


Fig. 3.10 Type of Faults

both vertical and horizontal directions (called Dip Slip) and lateral directions (called Strike Slip), with one of them dominating sometimes (Fig. 3.10).

3.7 जमीन कैसे हिलती है / How the Ground shakes

Seismic waves: Large strain energy released during an earthquake travels as seismic waves in all directions through the Earth's layers, reflecting and refracting at each interface (Fig. 3.11).

There are of two types of waves: 1) Body Waves
2) Surface Waves

Body waves are of two types:

a) Primary Waves (P-Wave)
b) Secondary Wave (S-Wave)

Surface waves are of two types, namely

a) Love Waves
b) Rayleigh Waves

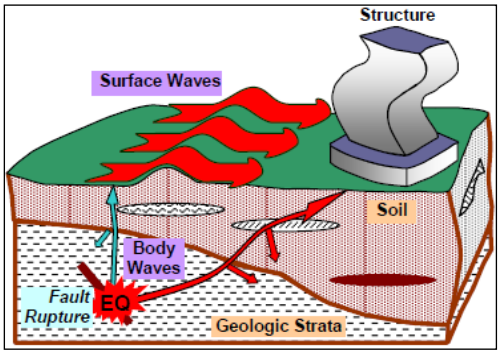


Fig. 3.11 Arrival of Seismic Waves at a Site

Body Waves: Body waves have spherical wave front. They consist of:

- **Primary Waves (P-waves):** Under P-waves [Fig. 3.11(a)], material particles undergo extensional and compressional strains along direction of energy transmission. These waves are faster than all other types of waves.

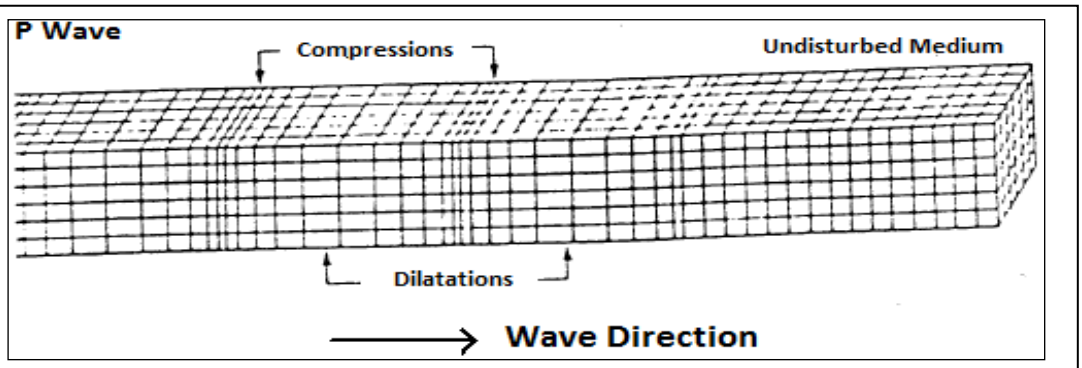


Fig. 3.11(a) Motions caused by Primary Waves

- **Secondary Waves (S-waves):** Under S-waves [Fig. 3.11(b)], material particles oscillate at

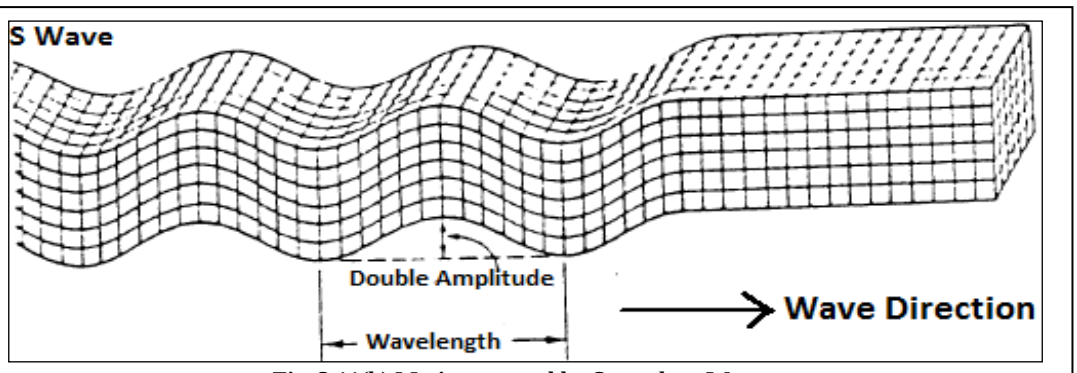


Fig. 3.11(b) Motions caused by Secondary Waves

right angles to direction of energy transmission. This type of wave shears the rock particle to the direction of wave travel. Since the liquid has no shearing resistance, these waves cannot pass through liquids.

Surface Waves: Surface waves have cylindrical wave front. They consist of:

- **Love Waves***:* In case of Love waves [Fig. 3.11(c)], the displacement is transverse with no vertical or longitudinal components (i.e. similar to secondary waves with no vertical component). Particle motion is restricted to near the surface. Love waves being transverse waves, these cannot travel in liquids.

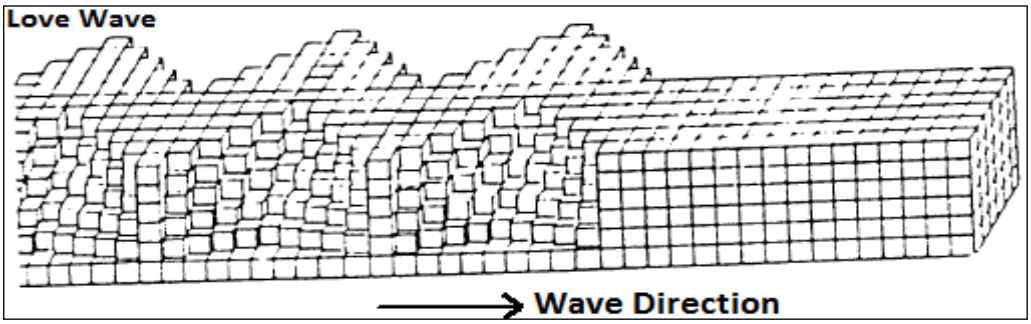


Fig. 3.11(c) Motions caused by Love Waves

- **Rayleigh Waves:** Rayleigh waves [Fig. 3.11(d)] make a material particle oscillate in an elliptic path in the vertical plane with horizontal motion along direction of energy transmission.

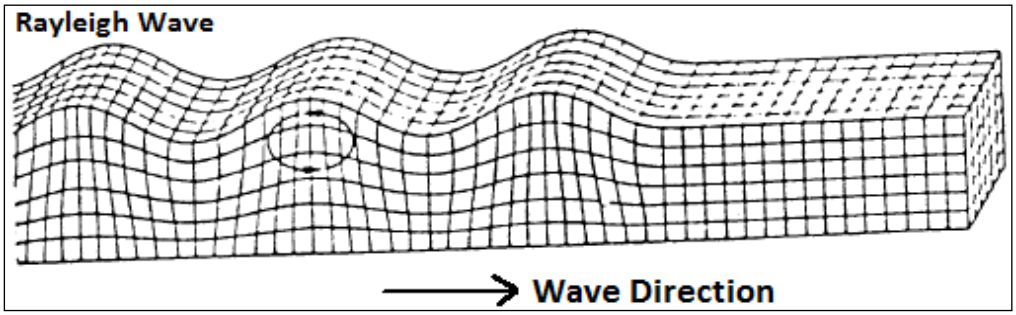


Fig. 3.11(d) Motions caused by Rayleigh Waves

Note: *Primary waves are fastest, followed in sequence by Secondary, Love and Rayleigh waves.*

3.8 भूकंप या भूकंपी खतरों का प्रभाव / Effects of Earthquake or Seismic Hazards

Basic causes of earthquake-induced damage are:

- Ground shaking
- Structural hazards
- Liquefaction
- Ground failure/ Landslides
- Tsunamis, and
- Fire

3.8.1 जमीन कंपन / Ground shaking

- Ground shaking can be considered to be the most important of all seismic hazards because all the other hazards are caused by ground shaking.
- When an earthquake occurs, seismic waves radiate away from the source and travel rapidly through the earth's crust.
- When these waves reach the ground surface, they produce shaking that may last from seconds to minutes.
- The strength and duration of shaking at a particular site depends on the size and location of the earthquake and on the characteristics of the site.
- At sites near the source of a large earthquake, ground shaking can cause tremendous damage.
- Where ground shaking levels are low, the other seismic hazards may be low or nonexistent
- Strong ground shaking can produce extensive damage from a variety of seismic hazards depending upon the characteristics of the soil.
- The characteristics of the soil can greatly influence the nature of shaking at the ground surface.
- Soil deposits tend to act as "filters" to seismic waves by attenuating motion at certain frequencies and amplifying it at others.
- Since soil conditions often vary dramatically over short distances, levels of ground shaking can vary significantly within a small area.
- One of the most important aspects of geotechnical earthquake engineering practice involves evaluation of the effects of local soil conditions on strong ground motion.

3.8.2 संरचनात्मक खतरे / Structural Hazards

- Without doubt the most dramatic and memorable images of earthquake damage are those of structural collapse, which is the leading cause of death and economic loss in many earthquakes.
- As the earth vibrates, all buildings on the ground surface will respond to that vibration in varying degrees.
- Earthquake induced accelerations, velocities and displacements can damage or destroy a building unless it has been designed and constructed or strengthened to be earthquake resistant.
- The effect of ground shaking on buildings is a principal area of consideration in the design of earthquake resistant buildings.
- Seismic design loads are extremely difficult to determine due to the random nature of earthquake motions.
- Structures need not collapse to cause death and damage. Falling objects such as brick facings and parapets on the outside of a structure or heavy pictures and shelves within a structure have caused casualties in many earthquakes. Interior facilities such as piping, lighting, and storage systems can also be damaged during earthquakes.
- However, experiences from past strong earthquakes have shown that reasonable and prudent practices can keep a building safe during an earthquake.
- Over the years, considerable advancement in earthquake-resistant design has moved from an emphasis on structural strength to emphases on both strength and ductility. In current design

practice, the geotechnical earthquake engineer is often consulted for providing the structural engineer with appropriate design ground motions.

3.8.3 द्रवीकरण / Liquefaction

In some cases, earthquake damage have occurred when soil deposits have lost their strength and appeared to flow as fluids. This phenomenon is termed as *liquefaction.* In *liquefaction,* the strength of the soil is reduced, often drastically, to the point where it is unable to support structures or remain stable. Because it only occurs in saturated soils, liquefaction is most commonly observed near rives, bays, and other bodies of water.

Soil liquefaction can occur in low density saturated sands of relatively uniform size. The phenomenon of liquefaction is particularly important for dams, bridges, underground pipelines, and buildings standing on such ground.

3.8.4 जमीन विफलता / लैंड स्लाइड / Ground Failure / Land slides

1) Earthquake-induced ground Failure has been observed in the form of ground rupture along the fault zone, landslides, settlement and soil liquefaction.
2) Ground rupture along a fault zone may be very limited or may extend over hundreds of kilometers.
3) Ground displacement along the fault may be horizontal, vertical or both, and can be measured in centimetres or even metres.
4) A building directly astride such a rupture will be severely damaged or collapsed.
5) Strong earthquakes often cause landslides.
6) In a number of unfortunate cases, earthquake-induced landslides have buried entire towns and villages.
7) Earthquake-induced landslides cause damage by destroying buildings, or disrupting bridges and other constructed facilities.
8) Many earthquake-induced landslides result from liquefaction phenomenon.
9) Others landslides simply represent the failures of slopes that were marginally stable under static conditions.
10) Landslide can destroy a building; the settlement may only damage the building.

3.8.5 सुनामी / Tsunamis

1) Tsunamis or seismic sea waves are generally produced by a sudden movement of the ocean floor.
2) Rapid vertical seafloor movements caused by fault rupture during earthquakes can produce long-period sea waves i.e. *Tsunamis.*
3) In the open sea, tsunamis travel great distances at high speeds but are difficult to detect – they usually have heights of less than 1 m and wavelengths (the distance between crests) of several hundred kilometres.
4) As a tsunami approaches shore, the decreasing water depth causes its speed to decrease and the height of the wave to increase.

5) As the water waves approach land, their velocity decreases and their height increases from 5 to 8 m, or even more.
6) In some coastal areas, the shape of the seafloor may amplify the wave, producing a nearly vertical wall of water that rushes far inland and causes devastating damage.
7) Tsunamis can be devastating for buildings built in coastal areas.

3.8.6 अग्नि / Fire

When the fire following an earthquake starts, it becomes difficult to extinguish it, since a strong earthquake is accompanied by the loss of water supply and traffic jams. Therefore, the earthquake damage increases with the earthquake-induced fire in addition to the damage to buildings directly due to earthquakes.

***

अध्याय /Chapter – 4

भूकंपी जोन और भूकंप का मापन /
Seismic Zone and Measurement of Earthquake

4.1 भूकंपी जोन / Seismic Zone

Due to convective flow of mantle material, crust of Earth and some portion of mantle slide on hot molten outer core. This sliding of Earth's mass takes place in pieces called Tectonic Plates. The surface of the Earth consists of seven major tectonic plates (Fig. 4.1).

They are

1. Eurasian Plate
2. Indo-Australian Plate
3. Pacific Plate
4. North American Plate
5. South American Plate
6. African Plate
7. Antarctic Plate

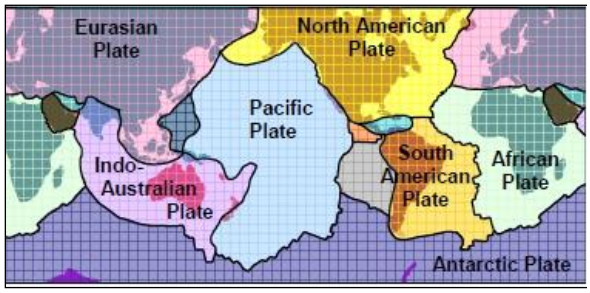


Fig. 4.1 Major Tectonic Plates on the Earth's surface

India lies at the northwestern end of the Indo Australian Plate (Fig. 4.2). This Plate is colliding against the huge Eurasian Plate and going under the Eurasian Plate. Three chief tectonic sub-regions of India are

- the mighty Himalayas along the north,
- the plains of the Ganges and other rivers, and
- the peninsula

Most earthquakes occur along the Himalayan plate boundary (these are inter-plate earthquakes), but a number of earthquakes have also occurred in the peninsular region (these are intra-plate earthquakes).

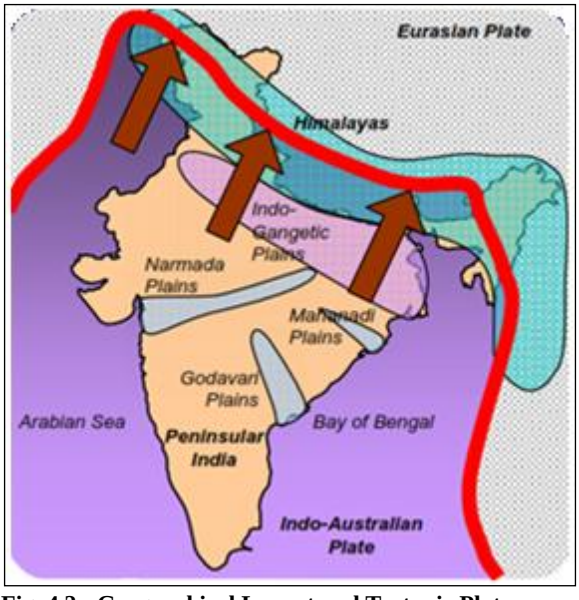


Fig. 4.2 Geographical Layout and Tectonic Plate Boundaries in India

Bureau of Indian Standards [IS1893 (part – 1) : 2002], based on various scientific inputs from a number of agencies including earthquake data supplied by Indian Meteorological Department (IMD), has grouped the country into four seismic zones viz., Zone II, III, IV and V. Of these, Zone V is rated as the most seismically prone region, while Zone II is the least (Fig. 4.3).

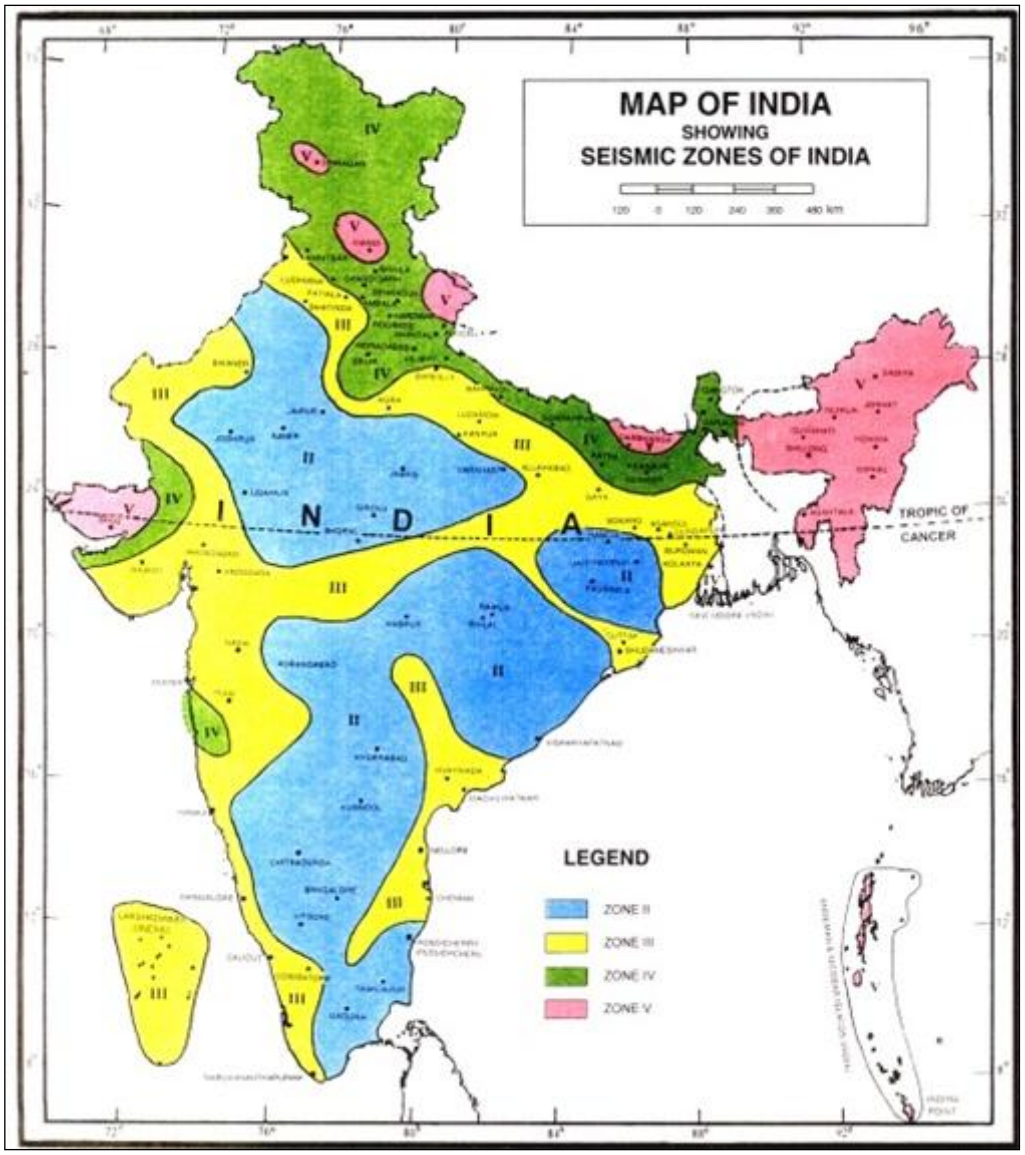


Fig. 4.3 Map showing Seismic Zones of India [IS 1893 (Part 1): 2002]

Indian Seismic code (IS 1893:2002) divides the country into four seismic zones based on the expected intensity of shaking in future earthquake. The four zones correspond to areas that have potential for shaking intensity on MSK scale as shown in the table.

Seismic Zone	Intensity on MSK scale	% of total area
II (Low intensity zone)	VI (or less)	43%
III (Moderate intensity zone)	VII	27%
IV (Severe intensity zone)	VIII	18%
V (Very Severe intensity zone)	IX (and above)	12%

4.2 भूकंप का मापन / Measurement of Earthquake

4.2.1 मापन उपकरण / Measuring Instruments

- **Seismograph:** The instrument that measures earthquake shaking is known as a seismograph (Fig. 4.4). It has three components –
  - **Sensor –** It consists of pendulum mass, string, magnet and support.
  - **Recorder –** It consists of drum, pen and chart paper.
  - **Timer –** It consists of the motor that rotates the drum at constant speed.

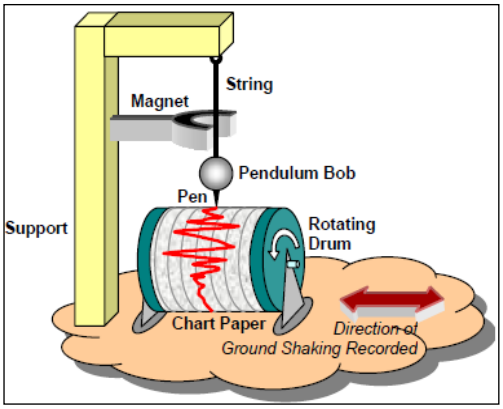


Fig. 4.4 Schematic of Early Seismograph

- **Seismoscopes:** Some instruments that do not have a timer device provide only the maximum extent (or scope) of motion during the earthquake.

- **Digital instruments:** The digital instruments using modern computer technology records the ground motion on the memory of the microprocessor that is in-built in the instrument.

Note: *The analogue instruments have evolved over time, but today, digital instruments are more commonly used.*

4.2.2 मापन के स्केल / Scale of Measurement

The Richter Magnitude Scale (also called Richter scale) assigns a magnitude number to quantify the energy released by an earthquake. Richter scale is a base 10 logarithmic scale, which defines magnitude as the logarithm of the ratio of the amplitude of the seismic wave to an arbitrary minor amplitude.

The magnitude M of an Earthquake is defined as

$$M = \log_{10} A - \log_{10} A_0$$

Where,

A = Recorded trace amplitude for that earthquake at a given distance as written by a standard type of instrument (say Wood Anderson instrument).

A_0 = Same as A, but for a particular earthquake selected as standard.

This number M is thus independent of distance between the epicentre and the station and is a characteristic of the earthquake. The standard shock has been defined such that it is low enough to make the magnitude of most of the recorded earthquakes positive and is assigned a magnitude of zero. Thus, if $A = A_0$,

$$M = \log_{10} A_0 - \log_{10} A_0 = 0$$

Standard shock of magnitude zero: It is defined as one that records peak amplitude of one thousandths of a millimetre at a distance of 100 km from the epicentre.

1) Zero magnitude does not mean that there is no earthquake.
2) Magnitude of an earthquake can be a negative number also.
3) An earthquake that records peak amplitude of 1 mm on a standard seismograph at 100 km will have its magnitude as

$$M = \log_{10} (1) - \log_{10} (10^{-3}) = 0 - (-3) = 3$$

Magnitude of a local earthquake: It is defined as the logarithm to base 10 of the maximum seismic wave amplitude (in thousandths of a mm) recorded on Wood Anderson seismograph at a distance of 100 kms from the earthquake epicentre.

1) With increase in magnitude by 1.0, the energy released by an earthquake increases by a factor of about 31.6.

2) A magnitude 8.0 earthquake releases about 31.6 times the energy released by a magnitude 7.0 earthquake or about 1000 times the energy released by a 6.0 earthquake.

3) With increase in magnitude by 0.2, the energy released by the earthquake doubles.

4.3 भूकंप परिमाण स्केल के प्रकार / Types of Earthquake Magnitude Scales

Several scales have historically been described as the "Ritcher Scale". The Ritcher local magnitude (M_L) is the best known magnitude scale, but it is not always the most appropriate scale for description of earthquake size. The Ritcher local magnitude does not distinguish between different types of waves.

At large epicentral distances, body waves have usually been attenuated and scattered sufficiently that the resulting motion is dominated by surface waves.

Other magnitude scales that base the magnitude on the amplitude of a particular wave have been developed. They are:

a) Surface Wave Magnitude (M_S)

b) Body Wave Magnitude (M_b)

c) Moment Magnitude (M_w)

4.3.1 सतह लहर परिमाण / Surface Wave Magnitude (M_S)

The *surface wave magnitude* (Gutenberg and Ritcher, 1936) is a worldwide magnitude scale based on the amplitude of Rayleigh waves with period of about 20 sec. The surface wave magnitude is obtained from

$$M_S = \log A + 1.66 \log \Delta + 2.0$$

Where A is the maximum ground displacement in micrometers and Δ is the epicentral distance of the seismometer measured in degrees (360^0 corresponding to the circumference of the earth).

The surface wave magnitude is most commonly used to describe the size of shallow (less than about 70 km focal depth), distant (farther than about 1000 km) moderate to large earthquakes.

4.3.2 बॉडी लहर परिमाण / Body Wave Magnitude (M_b)

For deep-focus earthquakes, surface waves are often too small to permit reliable evaluation of the surface wave magnitude. The *body wave magnitude* (Gutenberg, 1945) is a worldwide magnitude scale based on the amplitude of the first few cycles of p-waves which are not strongly influenced by the focal depth (Bolt, 1989). The body wave magnitude can be expressed as

$$M_b = \log A - \log T + 0.01\Delta + 5.9$$

Where, A is the p-wave amplitude in micrometers and T is the period of the p-wave (usually about one sec).

Saturation

For strong earthquakes, the measured ground-shaking characteristics become less sensitive to the size of the earthquake than the smaller earthquakes. This phenomenon is referred to as *saturation* (Fig. 4.5).

The body wave and the Ritcher local magnitudes saturate at magnitudes of 6 to 7 and the surface wave magnitude saturates at about $M_s = 8$.

To describe the size of a very large earthquake, a magnitude scale that does not depend on ground-shaking levels, and consequently does not saturate, would be desirable.

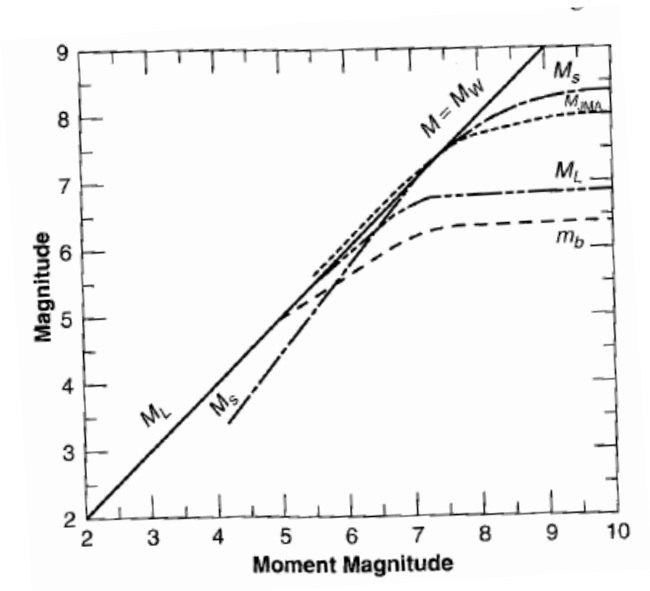


Fig. 4.5 Saturation of various magnitude scale: M_w (Moment Magnitude), M_L (Ritcher Local Magnitude), M_S (Surface Wave Magnitude), m_b (Short-period Body Wave Magnitude), m_B (Long-period Body Wave Magnitude), and M_{JMA} (Japanese Meteorological Agency Magnitude).

4.3.3 पल परिमाण / Moment Magnitude (M_w)

The only magnitude scale that is not subject to saturation is the moment magnitude.

The moment magnitude is given by:

$$M_w = [(\log M_0)/1.5] - 10.7$$

Where, M_0 is the seismic moment in dyne-cm.

4.4 भूकंप तीव्रता / Earthquake Intensity

Earthquake magnitude is simply a measure of the size of the earthquake reflecting the elastic energy released by the earthquake. It is usually referred by a certain real number on the Ritcher scale (e.g. magnitude 6.5 earthquake).

On the other hand, earthquake intensity indicates the extent of shaking experienced at a given location due to a particular earthquake. It is usually referred by a Roman numeral on the Modified Mercalli Intensity (MMI) scale as given below:

I	Not felt except by a very few under especially favourable circumstances.
II	Felt by only a few persons at rest, especially on upper floors of buildings; delicately suspended objects may swing.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake; standing motor cars may rock slightly; vibration like passing of truck; duration estimated.
IV	During the day felt indoors by many, outdoors by few; at night some awakened; dishes, windows, doors disturbed; walls make cracking sound; sensation like heavy truck striking building; standing motor cars rocked noticeably.
V	Felt by nearly everyone, many awakened; some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned; disturbances of trees, piles, and other tall objects sometimes noticed; pendulum clocks may stop.
VI	Felt by all, many frightened and run outdoors; some heavy furniture moved; a few instances of fallen plaster or damaged chimneys; damage slight.
VII	Everybody runs outdoors; damage negligible in buildings of good design and construction, slight to moderate in well-built ordinary structures, considerable in poorly built or badly designed structures; some chimneys broken; noticed by persons driving motor cars.
VIII	Damage slight in specially designed structures, considerable in ordinary substantial buildings, with partial collapse, great in poorly built structures; panel walls thrown out of frame structures; fall of chimneys, factory stacks, columns, monuments, walls; heavy furniture overturned; sand and mud ejected in small amounts; changes in well water; persons driving motor cars disturbed.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings, with partial collapse; buildings shifted off foundations; ground cracked conspicuously; underground pipes broken.

X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked; rails bent; landslides considerable from river banks and steep slopes; shifted sand and mud; water splashed over banks.
XI	Few, if any (masonry) structures remain standing; bridges destroyed; broad fissures in ground; underground pipelines completely out of service; earth slumps and land slips in soft ground; rails bent greatly.
XII	Damage total; practically all works of construction are damaged greatly or destroyed; waves seen on ground surface; lines of sight and level are destroyed; objects thrown into air.

4.4.1 MSK तीव्रता स्केल / MSK Intensity Scale

The MSK intensity scale is quite comparable to the Modified Mercalli intensity scale but is more convenient for application in field and is widely used in India. In assigning the MSK intensity scale at a site, due attention is paid to:

- Type of Structures (Table – A)
- Percentage of damage to each type of structure (Table – B)
- Grade of damage to different types of structures (Table – C)
- Details of Intensity Scale (Table – D)

The main features of MSK intensity scale are as follows:

Table – A : Types of Structures (Buildings)

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

Table – B : Definition of Quantity

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

Table – C : Classification of Damage to Buildings

Grade	Definitions	Descriptions
G1	Slight damage	Fine cracks in plaster; fall of small pieces of plaster.
G2	Moderate damage	Small cracks in plaster; fall of fairly large pieces of plaster; pantiles slip off cracks in chimneys parts of chimney fall down.
G3	Heavy damage	Large and deep cracks in plaster; fall of chimneys.
G4	Destruction	Gaps in walls; parts of buildings may collapse; separate parts of the buildings lose their cohesion; and inner walls collapse.
G5	Total damage	Total collapse of the buildings.

Table – D : Details of Intensity Scale

Intensity		Descriptions
I	Not noticeable	The intensity of the vibration is below the limits of sensibility; the tremor is detected and recorded by seismograph only.
II	Scarcely noticeable (very slight)	Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.
III	Weak, partially observed only	The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects, somewhat more heavily on upper floors.
IV	Largely observed	The earthquake is felt indoors by many people, outdoors by few. Here and there people awake, but no one is frightened. The vibration is like that due to the passing of a heavily loaded truck. Windows, doors, and dishes rattle. Floors and walls crack. Furniture begins to shake. Hanging objects swing slightly. Liquid in open vessels are slightly disturbed. In standing motor cars, the shock is noticeable.
V	Awakening	a) The earthquake is felt indoors by all, outdoors by many. Many 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 overturn or shift. Open doors and windows are thrust open and slam back again. Liquids spill in small amounts from well-filled open containers. The sensation of vibration is like that due to heavy objects falling inside the buildings. b) Slight damages in buildings of Type A are possible. c) Sometimes changes in flow of springs.
VI	Frightening	a) Felt by most indoors and outdoors. Many people in buildings are frightened and run outdoors. A few persons loose their balance. Domestic animals run out of their stalls. In few instances, dishes and glassware may break, and books fall down. Heavy furniture may possibly move and small steeple bells may ring. b) Damage of Grade 1 is sustained in single buildings of Type B and in many of Type A. Damage in few buildings of Type A is of Grade 2. c) In few cases, cracks up to widths of 1cm possible in wet ground; in mountains occasional landslips; change in flow of springs and in level of well water are observed.
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, landslides of roadway on steep slopes; crack inroads; seams of pipelines damaged; cracks in stone walls. c) Waves are formed on water, and is made turbid by mud stirred up. Water levels in wells change and the flow of springs changes. Sometimes dry springs have their flow resorted and existing springs stop flowing. In isolated instances parts of sand and gravelly banks slip off.
VIII	Destruction of buildings	a) Fright and panic; also persons driving motor cars are disturbed. Here and there branches of trees break off. Even heavy furniture moves and partly overturns. Hanging lamps are damaged in part. 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. Most buildings of Type A suffer damage of Grade 4. Occasional breaking of pipe seams. Memorials and monuments move and twist. Tombstones overturn. Stone walls collapse. c) Small landslips in hollows and on banked roads on steep slopes; cracks in ground up to widths of several centimetres. Water in lakes becomes turbid. New reservoirs come into existence. Dry wells refill and existing wells become dry. In many cases, change in flow and level of water is observed.
IX	General damage of 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 a damage of Grade 4 and a few of Grade 5. Many buildings of Type A suffer damage of Grade 5. Monuments and columns fall. Considerable damage to reservoirs; underground pipes partly broken, In individual cases, railway lines are bent and roadway damaged. c) On flat land overflow of water, sand and mud is often observed. Ground cracks to widths of up to 10 cm, on slopes and river banks more than 10 cm. Furthermore, a large number of slight cracks in ground; falls of rock, many landslides and earth flows; large waves in water. Dry wells renew their flow and existing wells dry up.
X	General destruction of building	a) Many buildings of Type C suffer damage of Grade 4, and a few of Grade 5. Many buildings of Type B show damage of Grade 5. Most of Type A have destruction of Grade 5. Critical damage to dykes and dams. Severe damage to bridges. Railway lines are bent slightly. Underground pipes are bent or broken. Road paving and asphalt show waves. b) In ground, cracks up to widths of several centimetres,

		sometimes up to 1m. Parallel to water courses occur broad fissures. Loose ground slides from steep slopes. From river banks and steep coasts, considerable landslides are possible. In coastal areas, displacement of sand and mud; change of water level in wells; water from canals, lakes, rivers. etc. thrown on land. New lakes occur.
XI	Destruction	a) Severe damage even to well built buildings. bridges, water dams and railway lines. Highways become useless Underground pipes destroyed. b) Ground considerably distorted by broad cracks and fissures, as well as movement in horizontal and vertical directions. Numerous landslips and falls of rocks. The intensity of the earthquake requires to be investigated specifically.
XII	Landscape changes	a) Practically all structures above and below ground are greatly damaged or destroyed. b) The surface of the ground is radically changed. Considerable ground cracks with extensive vertical and horizontal movements are observed. Falling of rock and slumping of river banks over wide areas, lakes are dammed; waterfalls appear and rivers are deflected. The intensity of the earthquake requires to be investigated specially.

4.4.2 विभिन्न स्केलों की तीव्रता मूल्यों की तुलना / Comparison of Intensity Values of Different Scales

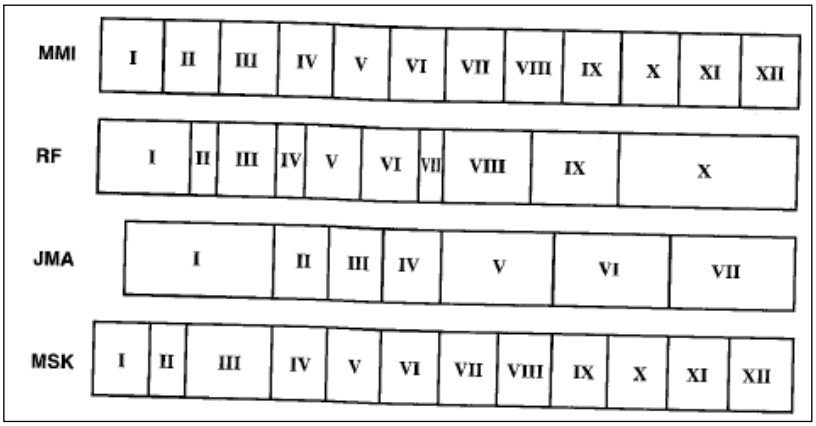


Fig. 4.5 Comparison of Intensity Values of Different Scales

4.4.3 विभिन्न परिमाण और तीव्रता के भूकंप का प्रभाव / Effect of Earthquake of various Magnitude and Intensity

The following describes the typical effects of earthquakes of various magnitudes near the epicenter. The values are typical only. They should be taken with extreme caution, since intensity and thus ground effects depend not only on the magnitude, but also on the distance to the epicenter, the depth of the earthquake's focus beneath the epicenter, the location of the epicenter and geological conditions (certain terrains can amplify seismic signals).

Magnitude	Description	Mercalli intensity	Average earthquake effects	Average frequency of occurrence (estimated)
1.0-1.9	Micro	I	Micro earthquakes, not felt, or felt rarely. Recorded by seismographs.	Continual/several million per year
2.0-2.9	Minor	I to II	Felt slightly by some people. No damage to buildings.	Over one million per year
3.0-3.9		III to IV	Often felt by people, but very rarely causes damage. Shaking of indoor objects can be noticeable.	Over 100,000 per year
4.0-4.9	Light	IV to VI	Noticeable shaking of indoor objects and rattling noises. Felt by most people in the affected area. Slightly felt outside. Generally causes none to minimal damage. Moderate to significant damage very unlikely. Some objects may fall off shelves or be knocked over.	10,000 to 15,000 per year
5.0-5.9	Moderate	VI to VIII	Can cause damage of varying severity to poorly constructed buildings. At most, none to slight damage to all other buildings. Felt by everyone.	1,000 to 1,500 per year
6.0-6.9	Strong	VII to X	Damage to a moderate number of well-built structures in populated areas. Earthquake-resistant structures survive with slight to moderate damage. Poorly designed structures receive moderate to severe damage. Felt in wider areas; up to hundreds of miles/kilometers from the epicenter. Strong to violent shaking in epicentral area.	100 to 150 per year
7.0-7.9	Major	VIII or Greater	Causes damage to most buildings, some to partially or completely collapse or receive severe damage. Well-designed structures are likely to receive damage. Felt across great distances with major damage mostly limited to 250 km from epicenter.	10 to 20 per year
8.0-8.9	Great		Major damage to buildings, structures likely to be destroyed. Will cause moderate to heavy damage to sturdy or earthquake-resistant buildings. Damaging in large areas. Felt in extremely large regions.	One per year
9.0 and greater			At or near total destruction – severe damage or collapse to all buildings. Heavy damage and shaking extends to distant locations. Permanent changes in ground topography.	One per 10 to 50 years

4.5 भूकंप निगरानी और सेवाओं के लिए एजेंसियों / Agencies for Earthquake Monitoring and Services

- Centre for Seismology (CS) in Indian Meteorological Department (IMD) under Ministry of Earth Sciences is nodal agency of Government of India dealing with various activities in the field of seismology and allied disciplines, and is responsible for monitoring seismic activity in and around the country.

- The major activities currently being pursued by the Centre for Seismology (CS) include:

  a) Earthquake monitoring on 24X7 basis, including real time seismic monitoring for early warning of tsunamis
  b) Operation and maintenance of national seismological network and local networks
  c) Seismological data centre and information services
  d) Seismic hazard and risk related studies
  e) Field studies for aftershock / swarm monitoring, site response studies
  f) Earthquake processes and modelling, etc.

- These activities are being managed by various units/groups of the Centre for Seismology (CS) as detailed below:

  1) Centre for Seismology (CS) is maintaining a country wide National Seismological Network (NSN), consisting of a total of 82 seismological stations, spread over the entire length and breadth of the country. This includes:

     a) 16-station V-SAT based digital seismic telemetry system around National Capital Territory (NCT) of Delhi.
     b) 20-station VSAT based real time seismic monitoring network in North East region of the country.
     (c) 17-station Real Time Seismic Monitoring Network (RTSMN) to monitor and report large magnitude under-sea earthquakes capable of generating tsunamis on the Indian coastal regions.

  2) The remaining stations are of standalone/ analog type.
  3) A Control Room is in operation, on a 24X7 basis, at premises of IMD Headquarters in New Delhi, with state-of-the art facilities for data collection, processing and dissemination of information to the concerned user agencies.
  4) India, represented by CS/IMD, is a permanent Member of the International Seismological Centre (ISC), UK.
  5) Seismological Bulletins of CS/IMD are shared regularly with International Seismological Centre (ISC), UK for incorporation in the ISC's Monthly Seismological Bulletins, which contain information on earthquakes occurring all across the globe.
  6) Towards early warning of tsunamis, real-time continuous seismic waveform data of three IMD stations, viz., Portblair, Minicoy and Shillong, is shared with global community, through IRIS (Incorporated Research Institutions of Seismology), Washington D.C., USA.

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अध्याय / Chapter – 5

भवन में भूकंप प्रतिरोध में सुधार के लिए सामान्य सिद्धांत / General Principle for improving Earthquake Resistance in Building

5.1 हल्कापन / 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.

5.2 निर्माण की निरंतरता / Continuity of Construction

- As far as possible, all parts of the building should be tied together in such a manner that the building acts as one unit.
- For integral action of building, roof and floor slabs should be continuous throughout as far as possible.
- Additions and alterations to the structures should be accompanied by the provision of positive measures to establish continuity between the existing and the new construction.

5.3 प्रोजेक्टिंग एवं सस्पेंडेड पार्ट्स / Projecting and Suspended Parts

- Projecting parts should be avoided as far as possible. If the projecting parts cannot be avoided, they should be properly reinforced and firmly tied to the main structure.
- Ceiling plaster should preferably be avoided. When it is unavoidable, the plaster should be as thin as possible.
- Suspended ceiling should be avoided as far as possible. Where provided, they should be light and adequately framed and secured.

5.4 भवन की आकृति / Shape of Building

- In order to minimize torsion and stress concentration, the building should have a simple rectangular plan.
- It should be symmetrical both with respect to mass and rigidity so that the centre of mass and rigidity of the building coincide with each other.
- It will be desirable to use separate blocks of rectangular shape particularly in seismic zones V and IV.

5.5 सुविधा जनक बिल्डिंग लेआउट / Preferred Building Layouts

Buildings having plans with shapes like, L, T, E and Y shall preferably be separated into rectangular parts by providing separation sections at appropriate places. Typical examples are shown in Fig. 5.1.

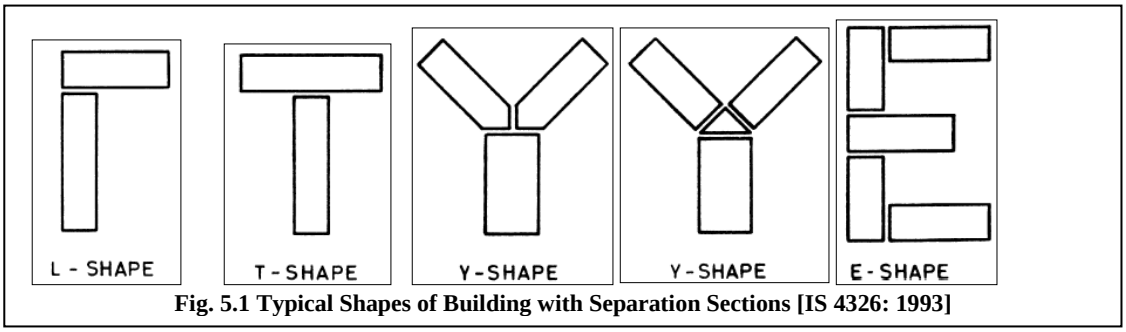


Fig. 5.1 Typical Shapes of Building with Separation Sections [IS 4326: 1993]

5.6 विभिन्न दिशाओं में शक्ति / Strength in Various Directions

The structure shall have adequate strength against earthquake effects along both the horizontal axes considering the reversible nature of earthquake forces.

5.7 नींव / Foundations

- For the design of foundations, the provisions of IS 1904: 1986 in conjunctions with IS 1893: 1984 shall generally be followed.
- The sub-grade below the entire area of the building shall preferably be of the same type of the soil. Wherever this is not possible, a suitably located separation or crumple section shall be provided.
- Loose fine sand, soft silt and expansive clays should be avoided. If unavoidable, the building shall rest either on a rigid raft foundation or on piles taken to a firm stratum. However, for light constructions the following measures may be taken to improve the soil on which the foundation of the building may rest:

  a) Sand piling, and b) Soil stabilization.

- Structure shall not be founded on loose soil, which will subside or liquefy during an earthquake resulting in large differential settlement.

5.8 छतें एवं मंजिलें / Roofs and Floors

5.8.1 सपाट छत या फर्श / Flat roof or floor

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

5.8.2 ढलान वाली छतें / Pitched Roofs

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

5.8.3 संवृत छतें / Pent Roofs

All roof trusses should be supported on and fixed to timber band reinforced concrete band or reinforced brick band. The holding down bolts should have adequate length as required for earthquake and wind forces.

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

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

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

5.8.4 जैक मेहराब / Jack Arches

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

5.9 सीढ़ियां / Staircases

- The interconnection of the stairs with the adjacent floors should be appropriately treated by providing sliding joints at the stairs to eliminate their bracing effect on the floors.
- Ladders may be made fixed at one end and freely resting at the other.
- Large stair halls shall preferably be separated from rest of the building by means of separation or crumple section.

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

5.9.1 अलग सीढ़ियां / Separated Staircases

One end of the staircase rests on a wall and the other end is carried by columns and beams which have no connection with the floors. The opening at the vertical joints between the floor and the staircase may be covered either with a tread plate attached to one side of the joint and sliding on the other side, or covered with some appropriate material which could crumple or fracture during an earthquake without causing structural damage.

The supporting members, columns or walls, are isolated from the surrounding floors by means of separation or crumple sections. A typical example is shown in Fig. 5.2.

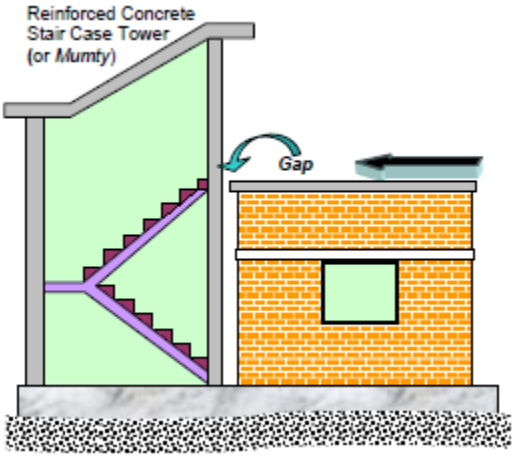


Fig. 5.2 Separated Staircase

5.9.2 बिल्ट-इन सीढ़ियां / Built-in Staircase

When stairs are built monolithically with floors, they can be protected against damage by providing rigid walls at the stair opening. An arrangement, in which the staircase is enclosed by two walls, is given in Fig. 5.3 (a). In such cases, the joints, as mentioned in respect of separated staircases, will not be necessary.

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

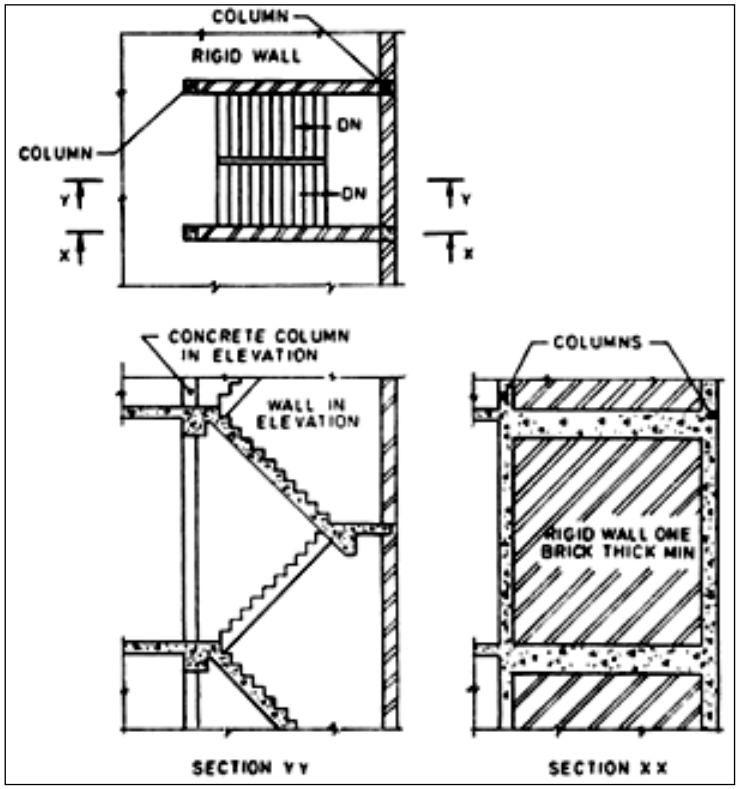


Fig. 5.3 (a) Rigidly Built-In Staircase [IS 4326: 1993]

5.9.3 स्लाइडिंग जोड़ों वाली सीढ़ियां / Staircases with Sliding Joints

In case it is not possible to provide rigid walls around stair openings for built-in staircase or to adopt the separated staircases, the staircases shall have sliding joints so that they will not act as diagonal bracing. (Fig. 5.3 (b))

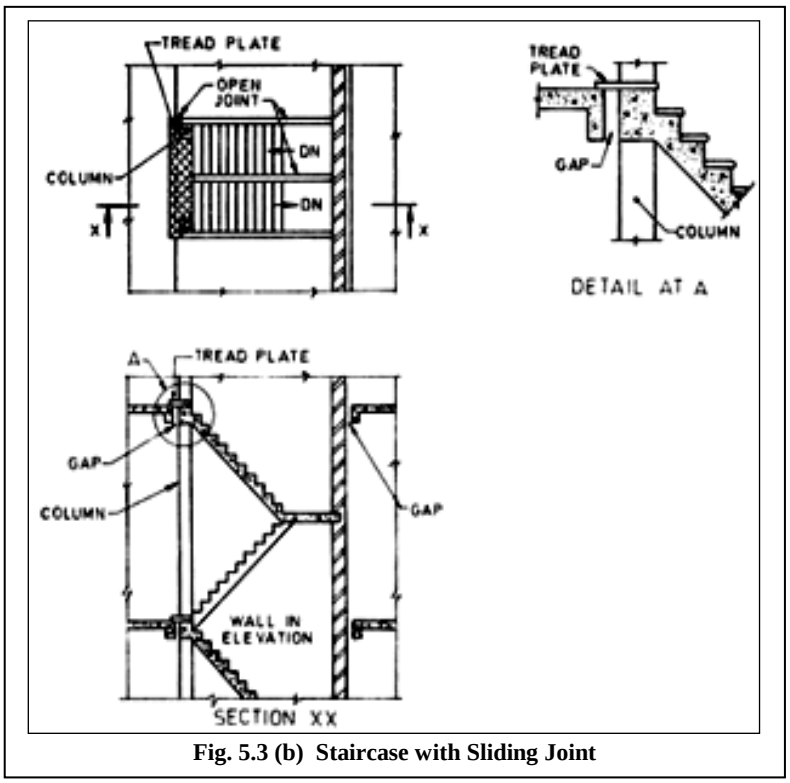


Fig. 5.3 (b) Staircase with Sliding Joint

5.10 बॉक्स प्रकार निर्माण / Box Type Construction

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

5.11 अग्नि सुरक्षा / Fire Safety

Fire frequently follows an earthquake and therefore buildings should be constructed to make them fire resistant in accordance with the provisions of relevant Indian Standards for fire safety. The relevant Indian Standards are IS 1641 : 1988, IS 1642 : 1989, IS 1643 : 1988, IS 1644 : 1988 and IS 1646 : 1986.

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अध्याय / Chapter – 6

भूकंप के दौरान आर सी भवनों के प्रदर्शन पर स्ट्रक्चरल अनियमितताओं का प्रभाव / Effect of Structural Irregularities on Performance of RC Buildings during Earthquakes

6.1 स्ट्रक्चरल अनियमितताओं का प्रभाव / Effect of Structural Irregularities

There are numerous examples of past earthquakes in which the cause of failure of reinforced concrete building has been ascribed to irregularities in configurations.

Irregularities are mainly categorized as

(i) Horizontal Irregularities

(ii) Vertical Irregularities

6.2 क्षैतिज अनियमितताएं / Horizontal Irregularities

Horizontal irregularities refer to asymmetrical plan shapes (e.g. L-, T-, U-, F-) or discontinuities in the horizontal resisting elements (diaphragms) such as cut-outs, large openings, re-entrant corners and other abrupt changes resulting in torsion, diaphragm deformations, stress concentration.

Table – 6.1 Definitions of Irregular Buildings – Plan Irregularities (Fig. 6.1)

S. No.	Irregularity Type and Description
(i)	***Torsion Irregularity*:** To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure

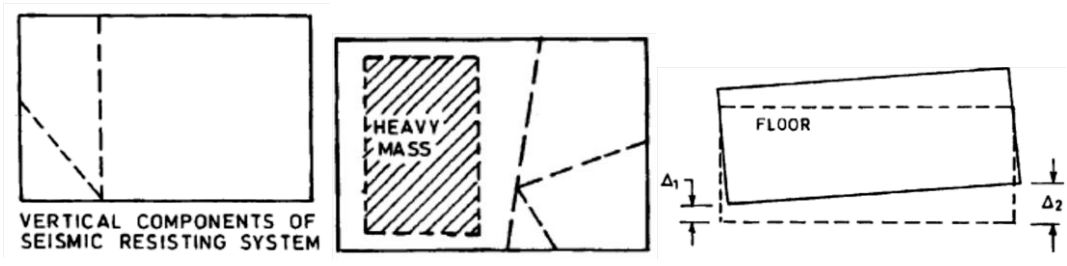


Fig. 6.1 (a)

(ii) ***Re-entrant Corners*:** Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction

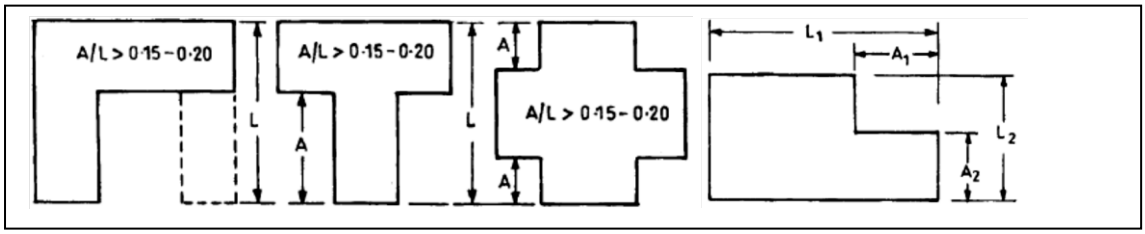


Fig. 6.1 (b)

(iii) ***Diaphragm Discontinuity*:** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next

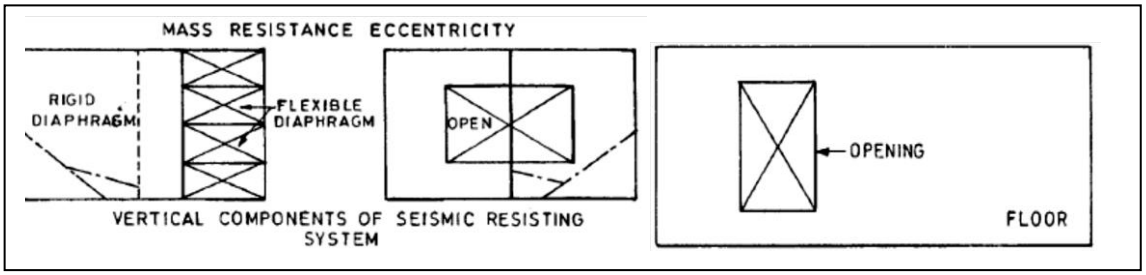


Fig. 6.1 (c)

(iv) ***Out-of-Plane Offsets*:** Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements

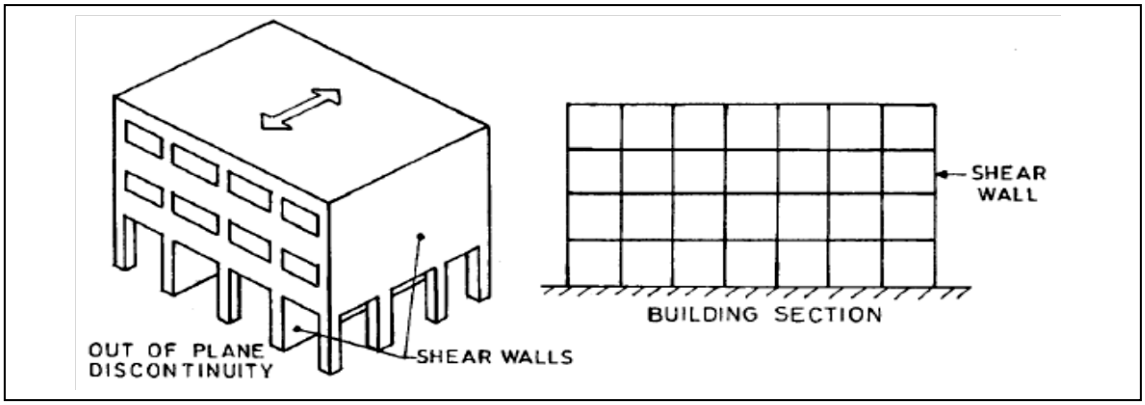


Fig. 6.1 (d)

(v) ***Non-parallel Systems*:** The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements

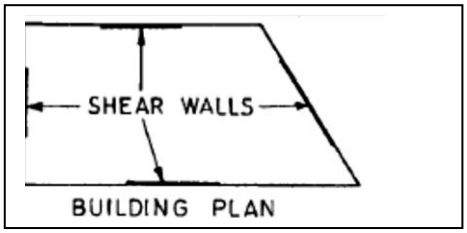
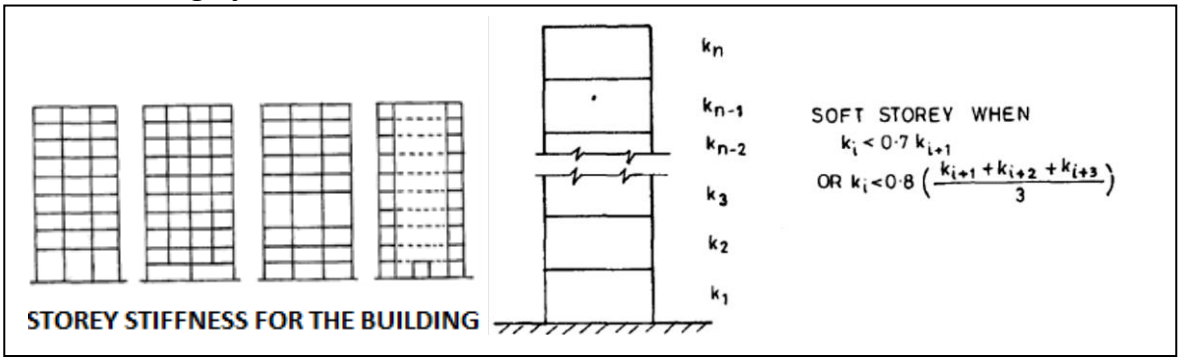
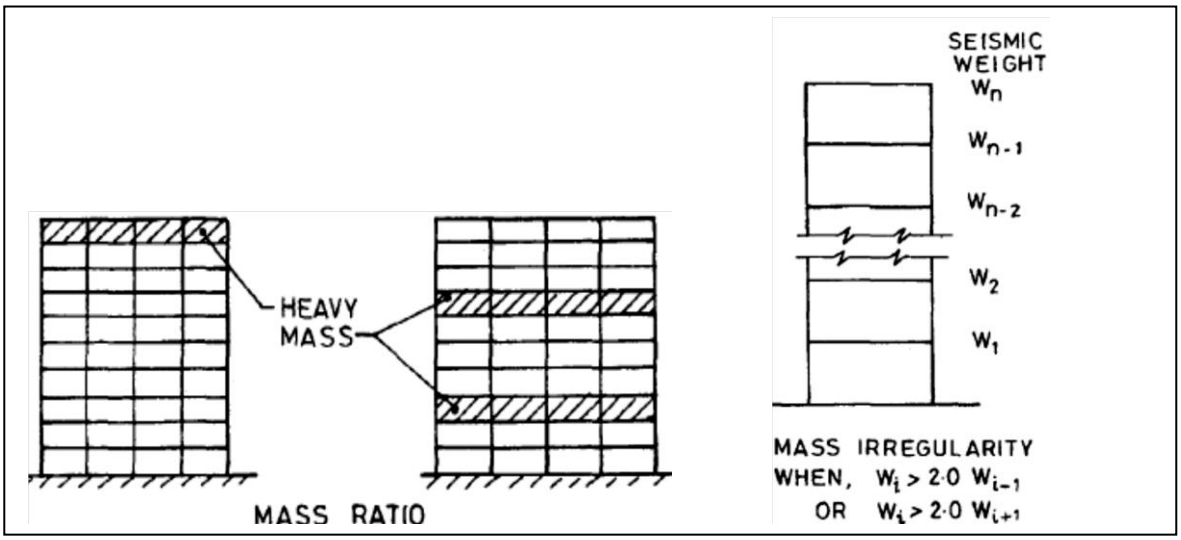


Fig. 6.1 (e)

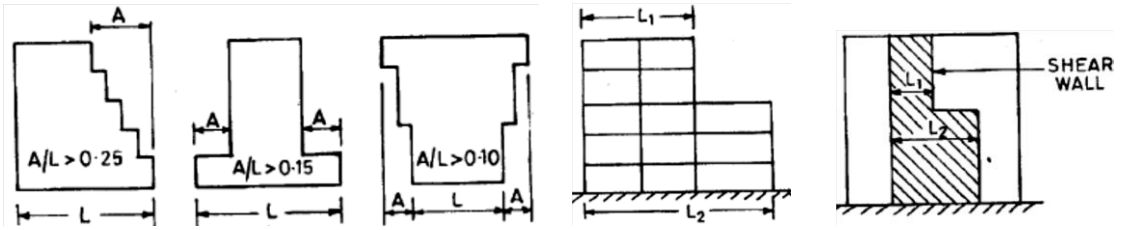
6.3 ऊर्ध्वाधर अनियमितताएं / Vertical Irregularities

Vertical irregularities, referring to sudden change of strength, stiffness, geometry and mass, result in irregular distribution of forces and/ or deformation over the height of building.

Table – 6.2 Definition of Irregular Buildings – Vertical Irregularities (Fig. 6.2)

S. No.	Irregularity Type and Description
(i)	a) ***Stiffness Irregularity – Soft Storey*:** A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above. b) ***Stiffness Irregularity – Extreme Soft Storey*:** A extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent of the average stiffness of the three storeys above. For example, buildings on STILTS will fall under this category.  **Fig. 6.2 (a)**
(ii)	**Mass *Irregularity*:** Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs  **Fig. 6.2 (b)**

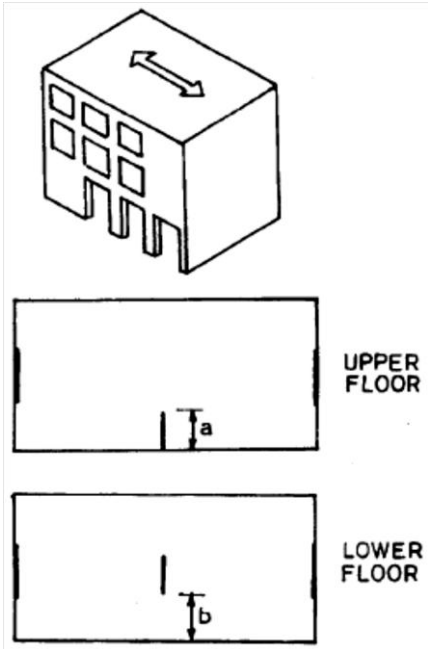
(iii) ***Vertical Geometric Irregularity*:** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than150 percent of that in its adjacent storey



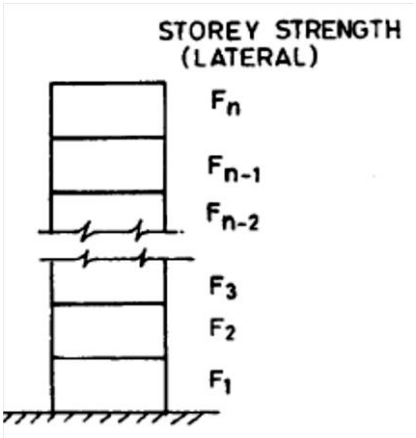
Vertical Geometric Irregularity when L2>1.5 L_1

Fig. 6.2 (c)

(iv) ***In-Plane Discontinuity in Vertical Elements Resisting Lateral Force*:** A in-plane offset of the lateral force resisting elements greater than the length of those elements



In-Plane Discontinuity in Vertical Elements Resisting Lateral Force when b >*a*



Weak Storey when *F*i<0.8*F*i+ 1

Fig. 6.2 (d)

(v) ***Discontinuity in Capacity – Weak Storey*:** A weak storey is one in which the storey lateral strength is less than 80 percent of that in the storey above, The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

6.4 भवन विन्यास अनियमितताएं – समस्याएँ, विश्लेषण एवं निदान के उपाय / Building Irregularities – Problems, Analysis and Remedial Measures

The influence of irregularity on performance of building during earthquakes is presented to account for the effects of these irregularities in analysis of problems and their solutions along with the design.

Architectural problems	Structural problems	Remedial measures
Extreme height/depth ratio	High overturning forces, large drift causing non-structural damage, foundation stability	Revive properties or special structural system
Extreme plan area	Built-up large diaphragm forces	Subdivide building by seismic joints
Extreme length depth ratio	Built-up of large lateral forces in perimeter, large differences in resistance of two axes Experience greater variations in ground movement and soil conditions	Subdivide building by seismic joints
Variation in perimeter strength-stiffness	Torsion caused by extreme variation in strength and stiffness	Add frames and disconnect walls, or use frames and lightweight walls
False symmetry	Torsion caused by stiff asymmetric core	Disconnect core, or use frame with non-structural core walls
Re-entrant corners	Torsion, stress concentrations at the notches	Separate walls, uniform box, centre box, architectural relief, diagonal reinforcement
Mass eccentricities	Torsion, stress concentrations	Reprogram, or add resistance around mass to balance resistance and mass
Vertical setbacks and reverse setbacks	Stress concentration at notch, different periods for different parts of building, high diaphragm forces to transfer at setback	Special structural systems, careful dynamic analysis
Soft storey frame	Causes abrupt changes of stiffness at point of discontinuity	Add bracing, add columns, braced
Variation in column stiffness	Causes abrupt changes of stiffness, much higher forces in stiffer columns	Redesign structural system to balance stiffness
Discontinuous shear wall	Results in discontinuities in load path and stress concentration for most heavily loaded elements	Primary concern over the strength of lower level columns and connecting beams that support the load of discontinuous frame
Weak column – strong beam	Column failure occurs before beam, short column must try and accommodate storey height displacement	Add full walls to reduce column forces, or detach spandrels from columns, or use light weight curtain wall with frame
Modification of primary structure	Most serious when masonry in-fill modifies structural concept, creation of short, stiff columns result in stress concentration	Detach in-fill, or use light-weight materials
Building separation (Pounding)	Possibility of pounding dependent on building period, height, drift, distance	Ensure adequate separation, assuming opposite building vibrations
Coupled	Incompatible deformation between walls and links	Design adequate link
Random Openings	Seriously degrade capacity at point of maximum force transfer	Careful designing, adequate space for reinforcing design

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अध्याय / Chapter – 7

भवन की डायनामिक विशेषताएँ / Dynamic Characteristics of Building

7.1 डायनामिक विशेषताएँ / Dynamic Characteristics

Buildings oscillate during earthquake shaking. The oscillation causes inertia force to be induced in the building. The intensity and duration of oscillation, and the amount of inertia force induced in a building depend on features of buildings, called dynamic characteristics of building.

The important dynamic characteristics of buildings are:

a) Modes of Oscillation

b) Damping

A mode of oscillation of a building is defined by associated Natural Period and Deformed Shape in which it oscillates. Every building has a number of natural frequencies, at which it offers minimum resistance to shaking induced by external effects (like earthquakes and wind) and internal effects(like motors fixed on it). Each of these natural frequencies and the associated deformation shape of a building constitute a Natural Mode of Oscillation.

The mode of oscillation with the smallest natural frequency (and largest natural period) is called the **Fundamental Mode**; the associated natural period $T_1$is called the **Fundamental Natural Period**.

7.2 प्राकृतिक अवधि / Natural Period

Natural Period (T_n) of a building is the time taken by it to undergo one complete cycle of oscillation. It is an inherent property of a building controlled by its mass m and stiffness k. These three quantities are related by

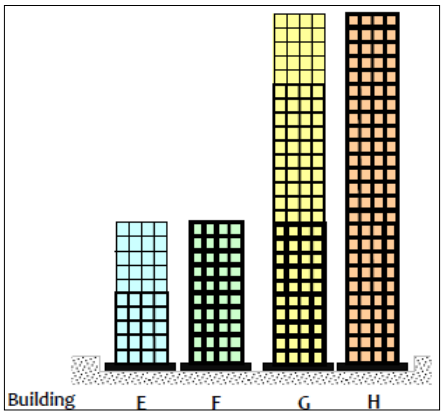
$$T_n = 2\pi\sqrt{\frac{m}{k}}$$ Its unit is second (s).

7.3 प्राकृतिक आवृत्ति / Natural Frequency

The reciprocal ($1/T_n$) of natural period of a building is called the Natural Frequency f_n; its unit is Hertz (Hz).

7.4 प्राकृतिक अवधि को प्रभावित करने वाले कारक / Factors influencing Natural Period

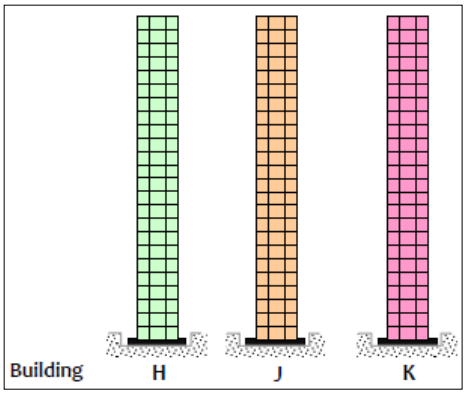
7.4.1 कठोरता का प्रभाव / Effect of Stiffness: Stiffer buildings have smaller natural period.



Building E	Building F
T_{X1} = 1.32 s	T_{X1} = 1.31 s
T_{Y1} = 1.36 s	T_{Y1} = 1.35 s
Building G	**Building H**
T_{X1} = 2.89 s	T_{X1} = 2.98 s
T_{Y1} = 3.04 s	T_{Y1} = 3.14 s

Fig. 7.1 Effect of Stiffness

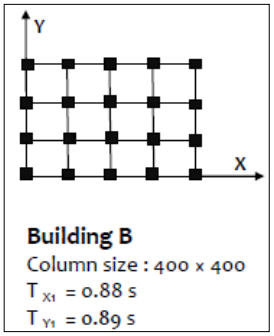
7.4.2 द्रव्यमान का प्रभाव / Effect of Mass: Heavier buildings have larger natural period.

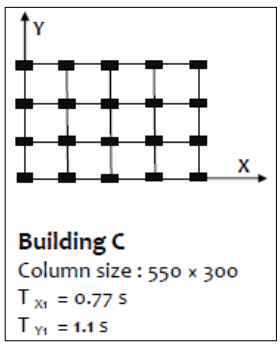


Building H	Building J	Building K
Mass m	Mass 1.1m	Mass 1.2m
T_{X1} = 2.98 s	T_{X1} = 3.12 s	T_{X1} = 3.25 s
T_{Y1} = 3.14 s	T_{Y1} = 3.29 s	T_{Y1} = 3.43 s

Fig. 7.2 Effect of Mass

7.4.3 कॉलम अभिविन्यास का प्रभाव / Effect of Column Orientation: Buildings with larger column dimension oriented in the direction reduces the translational natural period of oscillation in that direction.





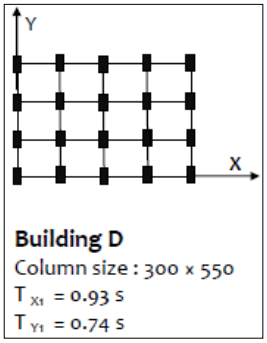


Fig. 7.3 Effect of Column Orientation

7.4.4 भवन की ऊंचाई का प्रभाव / Effect of Building Height: Taller buildings have larger natural period.

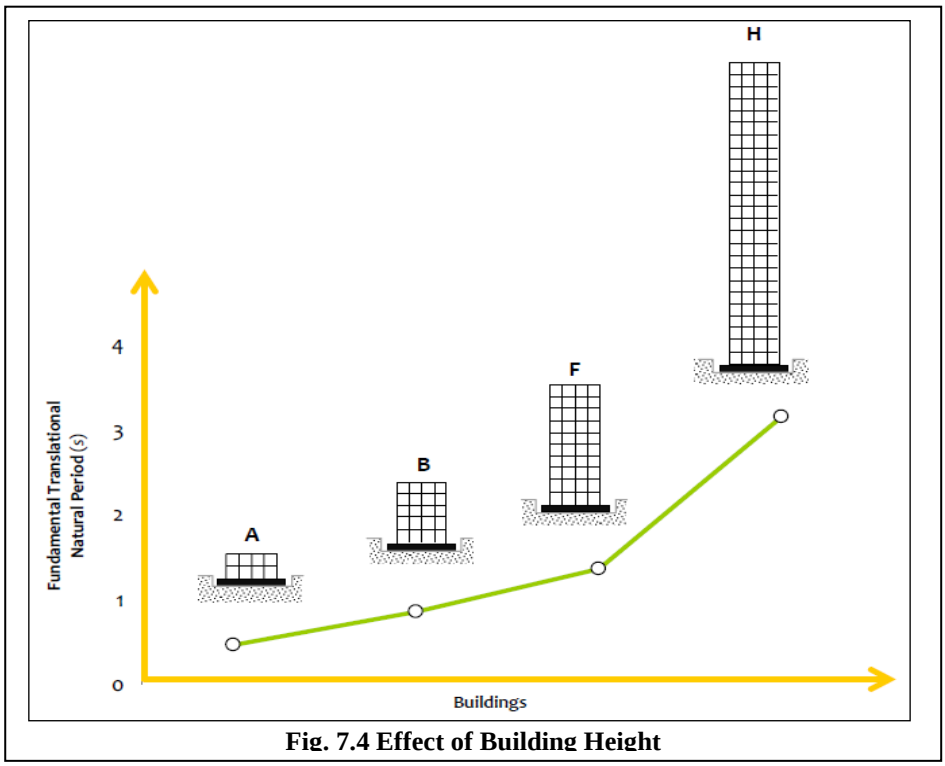


Fig. 7.4 Effect of Building Height

7.4.5 Unreinforced चिनाई भराव का प्रभाव / Effect of Unreinforced Masonry Infills: Natural Period of building is lower when the stiffness contribution of URM infill is considered

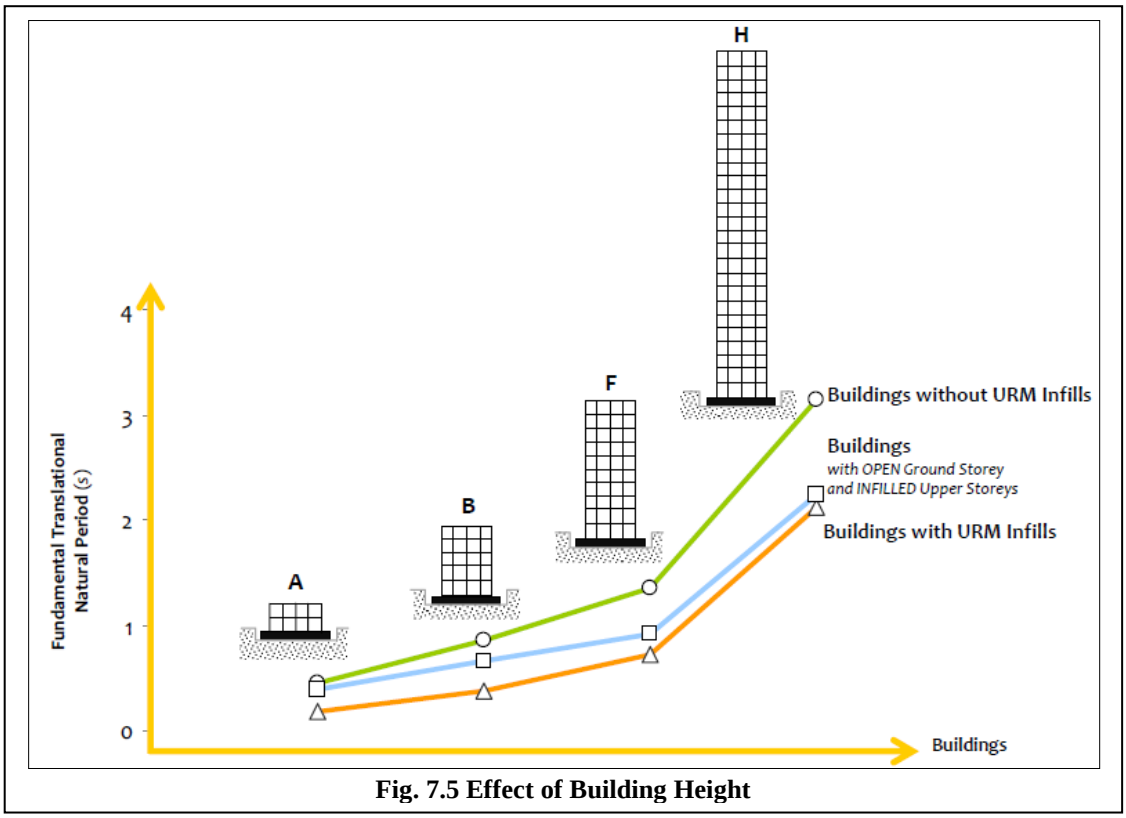


Fig. 7.5 Effect of Building Height

7.5 Mode आकृति / Mode Shape

Mode shape of oscillation associated with a natural period of a building is the deformed shape of the building when shaken at the natural period. Hence, a building has as many mode shapes as the number of natural periods.

The deformed shape of the building associated with oscillation at *fundamental* natural period is termed its *first mode shape*. Similarly, the deformed shapes associated with oscillations at *second*, *third*, and other higher natural periods are called *second mode shape*, *third mode shape*, and so on, respectively.

Fundamental Mode Shape of Oscillation

As shown in Fig. 7.6, there are three basic modes of oscillation, namely,

1. Pure translational along X-direction,
2. Pure translational along Y-direction and
3. Pure rotation about Z-axis

Regular buildings

These buildings have pure mode shapes.The *Basic modes of oscillation* i.e. two translational and one rotational mode shapes.

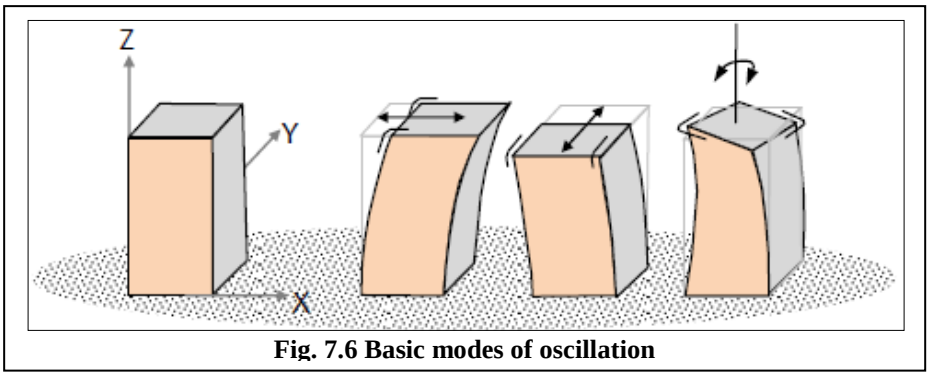


Fig. 7.6 Basic modes of oscillation

Irregular buildings

These buildings that have irregular geometry, non-uniform distribution of mass and stiffness in plan and along the height have mode shapes, which are a mixture of these pure mode shapes. Each of these mode shapes is independent, implying, it cannot be obtained by combining any or all of the other mode shapes.

a) **Fundamental and two higher translational modes of oscillation along X-direction of a five storey benchmark building**: First modes shape has one zero crossing of the un-deformed position, second two, and third three.

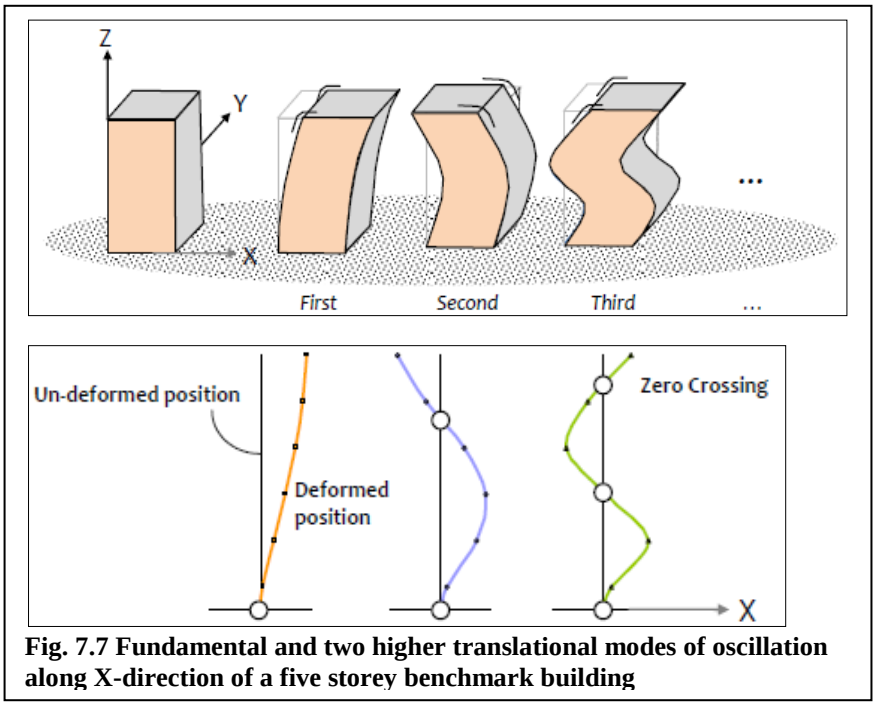


Fig. 7.7 Fundamental and two higher translational modes of oscillation along X-direction of a five storey benchmark building

b) Diagonal modes of oscillation: First three modes of oscillation of a building symmetric in both directions in plan; first and second are diagonal translational modes and third rotational

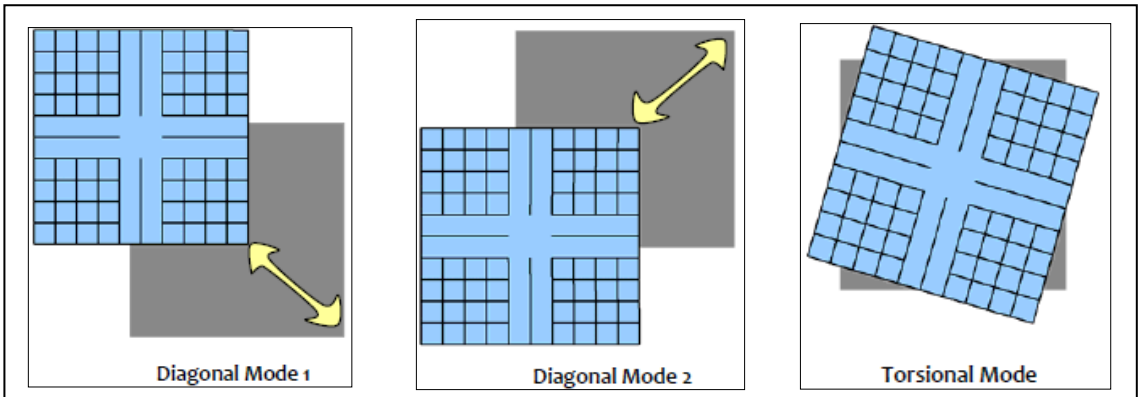


Fig. 7.8 Diagonal modes of oscillation

c) Effect of modes of oscillation on column bending: Columns are severely damaged while bending about their diagonal direction

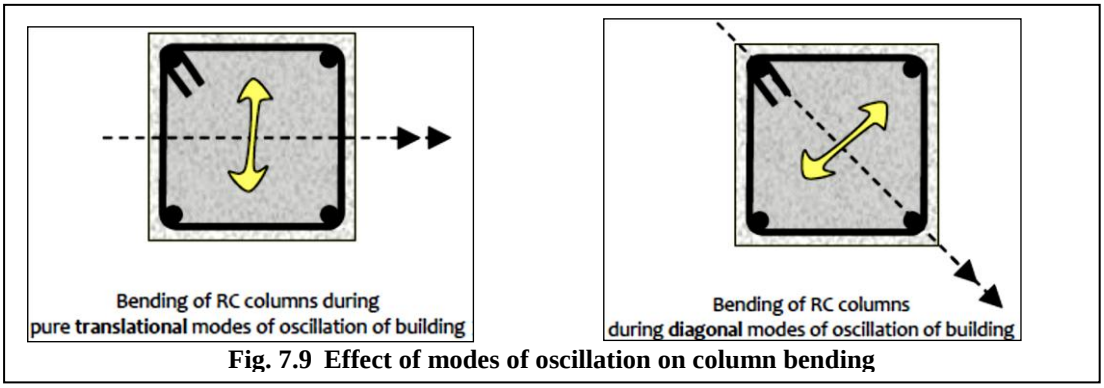


Fig. 7.9 Effect of modes of oscillation on column bending

7.6 Mode आकृतियों को प्रभावित करने वाले कारक / Factors influencing Mode Shapes

7.6.1 Effect of relative flexural stiffness of structural elements: Fundamental translational mode shape changes from flexural-type to shear-type with increase in beam flexural stiffness relative to that of column

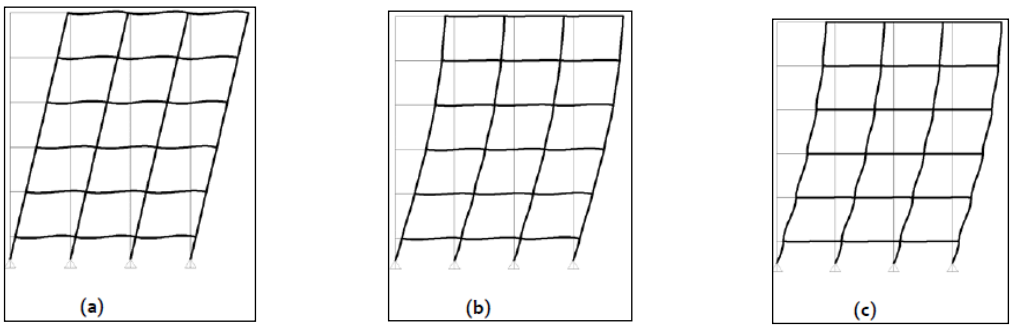


Fig. 7.10 Effect of relative flexural stiffness of structural elements

7.6.2 Effect of axial stiffness of vertical members: Fundamental translational mode shape changes from flexure-type to shear-type with increase in axial stiffness of vertical members.

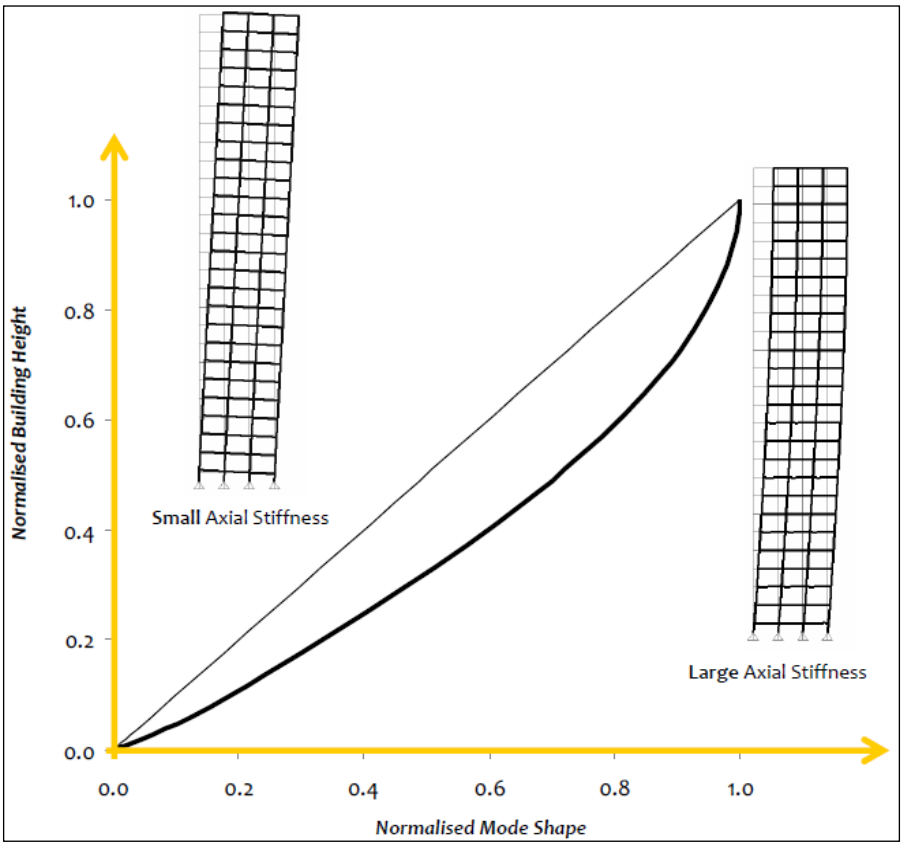


Fig. 7.11 Effect of axial stiffness of vertical members

7.6.3 Effect of degree of fixity at member ends: Lack of fixity at beam ends induces flexural-type behaviour, while the same at column bases induces shear-type behaviour to the fundamental translational mode of oscillation.

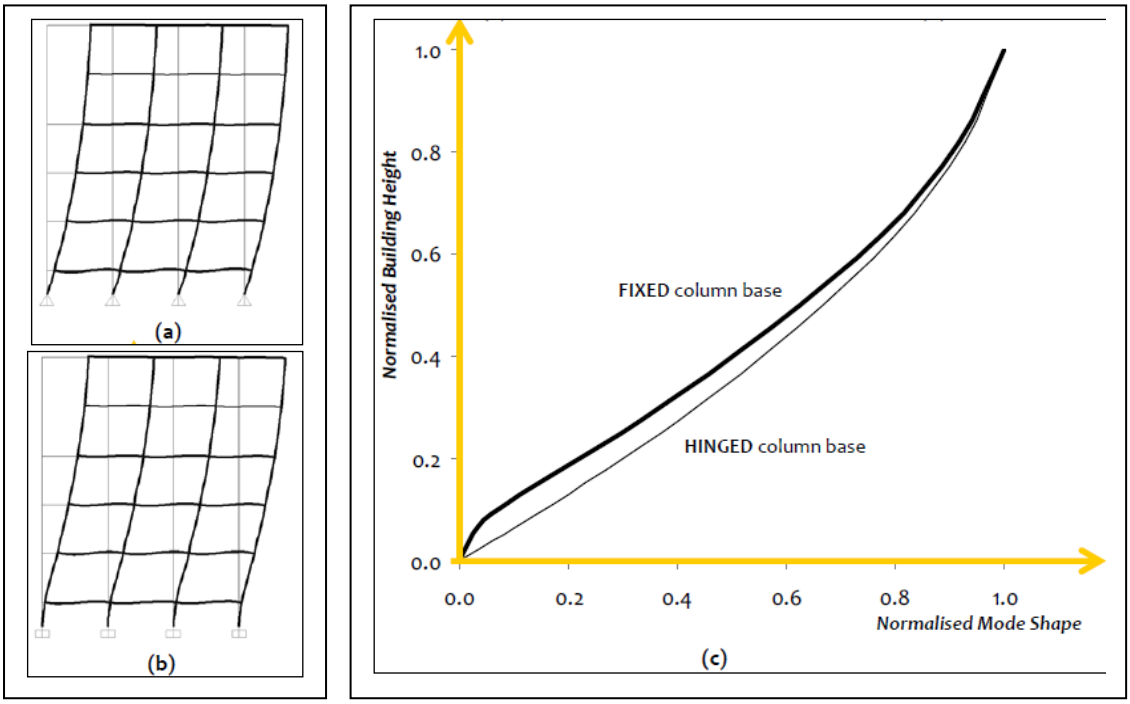


Fig. 7.12 Effect of degree of fixity at member ends

7.6.4 Effect of building height: Fundamental translational mode shape of oscillation does not change significantly with increase in building height, unlike the fundamental translational natural period, which does change.

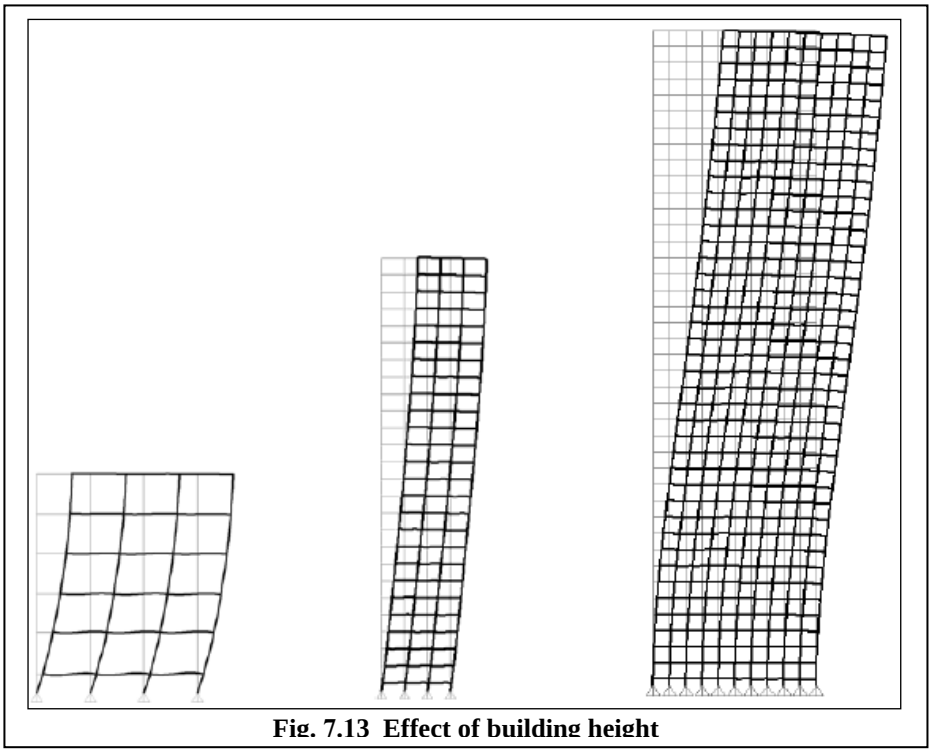


Fig. 7.13 Effect of building height

7.6.5 Influence of URM Infill Walls in Mode Shape of RC frame Buildings: Mode shape of a building obtained considering stiffness contribution of URM is significantly different from that obtained without considering the same.

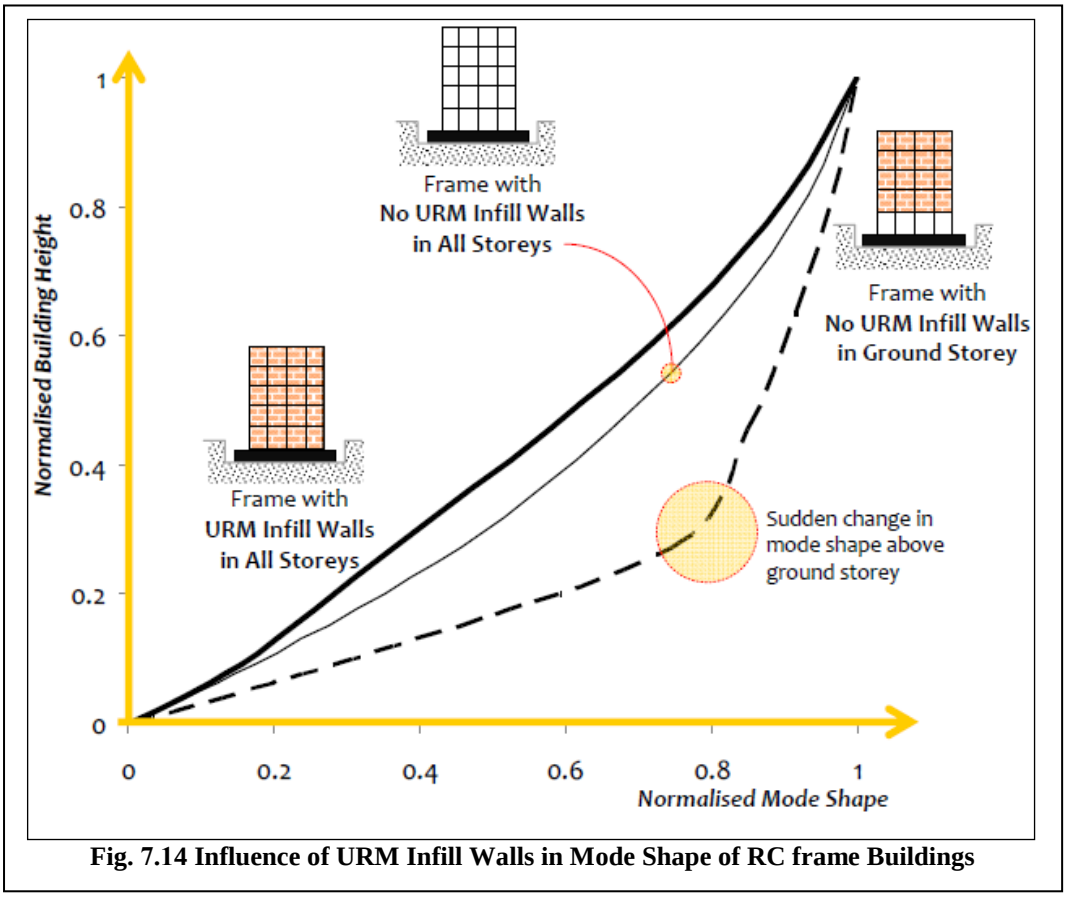


Fig. 7.14 Influence of URM Infill Walls in Mode Shape of RC frame Buildings

7.7 संरचना की प्रतिक्रिया / Response of Structure

The earthquakes cause vibratory motion which is cyclic about the equilibrium. The structural response is vibratory (Dynamic) and it is cyclic about the equilibrium position of structure. The fundamental natural frequency of most civil engineering structures lie in the range of 0.1 sec to 3.0 sec or so. This is also the range of frequency content of earthquake-generated ground motions. Hence, the ground motion imparts considerable amount of energy to the structures. Initially, the structure responds elastically to the ground motion; however, as its yield capacity is exceeded, the structure responds in an inelastic manner. During the inelastic response, stiffness and energy dissipation properties of the structure are modified.

Response of the structure to a given strong ground motion depends not only on the properties of input ground motion, but also on the structural properties.

7.8 डिजाइन स्पेक्ट्रम / Design Spectrum

The design spectrum is a design specification which is arrived at by considering all aspects. The design spectrum may be in terms of acceleration, velocity, or displacement.

Since design spectrum is a specification for design, it cannot be viewed in isolation without considering the other factors that go into the design process. One must concurrently specify

a) The procedure to calculate natural period of the structure.
b) The damping to be used for a given type of structure.
c) The permissible stresses and strains, load factors, etc.

Unless this information is part of a design spectrum, the design specification is incomplete.

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अध्याय / Chapter – 8

डिजाइन पार्श्व बलों के निर्धारण के लिए कोड आधारित तरीका / Code based Procedure for Determination of Design Lateral Loads

8.1 भूकंपी डिजाइन की फिलोस्फ़ी / Philosophy of Seismic Design

Design of earthquake effect is not termed as Earthquake Proof Design. Actual forces that appear on structure during earthquake are much greater than the design forces. Complete protection against earthquake of all size is not economically feasible and design based alone on strength criteria is not justified. Earthquake demand is estimated only based on concept of probability of exceedance. Design of earthquake effect is, therefore, termed as Earthquake Resistant Design against the probable value of demand.

Maximum Considered Earthquake (MCE): The earthquake corresponding to the Ultimate Safety Requirement is often called as Maximum Considered Earthquake.

Design Basis Earthquake (DBE): It is defined as the Maximum Earthquake that reasonably can be expected to experience at the site during lifetime of structure.

The philosophy of seismic design is to ensure that structures possess at least a minimum strength to

(i) resist minor (< DBE), which may occur frequently, without damage
(ii) resist moderate earthquake (DBE) without significant structural damage through some non-structural damage
(iii) resist major earthquake (MCE) without collapse

8.2 भूकंपी विश्लेषण के लिए तरीके / Methods for Seismic Analysis

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

Analysis is carried out by:

a- Dynamic analysis procedure [Clause 7.8 of IS 1893 (Part I): 2002]

b- Simplified method referred as Lateral Force Procedure [Clause 7.5 of IS 1893 (Part I): 2002] also recognized as Equivalent Lateral Force Procedure or Equivalent Static Procedure in the literature.

The main difference between the equivalent lateral force procedure and dynamic analysis procedure lies in the magnitude and distribution of lateral forces over the height of the buildings.

In the dynamic analysis procedure the lateral forces are based on the properties of the natural vibration modes of the building, which are determined by the distribution of mass and stiffness over height. In the equivalent lateral force procedures the magnitude of forces is based on an estimation of the fundamental period and on the distribution of forces, as given by simple formulae appropriate for regular buildings.

8.3 डायनामिक विश्लेषण / Dynamic Analysis

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

a) Regular buildings – Those greater than 40 m in height in Zones IV and V, and those greater than 90 m in height in Zones II and III. Modelling as per Para 7.8.4.5 of IS 1893 (Part 1): 2002 can be used.

b) Irregular buildings (as defined in Table – 6.1 and Table – 6.2 of Chapter - 6**) –** All framed buildings higher than 12m in Zones IV and V, and those greater than 40m in height in Zones II and III.

8.4 पार्श्व बल प्रक्रिया / Lateral Force Procedure

The random earthquake ground motions, which cause the structure to vibrate, can be resolved in any three mutually perpendicular directions. The predominant direction of ground vibration is usually horizontal.

The codes represent the earthquake-induced inertia forces in the form of design equivalent static lateral force. This force is called as the Design Seismic Base Shear V_B. V_B remains the primary quantity involved in force-based earthquake-resistant design of buildings.

The Design Seismic Base Shear V_B is given by:

$$V_B = A_h W = \frac{ZI}{2R}\left(\frac{S_a}{g}\right)W$$

Where, A_h = Design horizontal seismic coefficient for a structure

$$= \frac{ZI}{2R}\left(\frac{S_a}{g}\right)$$

Z = Zone Factor

It is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone.

Generally Design Basis Earthquake (DBE) is half of Maximum Considered Earthquake (MCE). The factor 2 in the denominator of Z is used so as to reduce the MCE zone factor to the factor for DBE.

The value of A_h will not be taken less than Z/2, whatsoever the value of I/R.

The value of Zone Factor is given in Table – 8.1.

Table – 8.1 Zone Factor, Z[IS 1893 (Part 1): 2002]

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

I = Importance Factor

Value of importance factor depends upon the functional use of the structures, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance (as given in Table – 8.2).

Table – 8.2 Importance Factors, I [IS 1893 (Part 1): 2002]

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

Note:
1. The design engineer may choose values of importance factor I greater than those mentioned above.
2. Buildings not covered in S. No. (i) and (ii) above may be designed for higher value of I, depending on economy, strategy considerations like multi-storey buildings having several residential units.
3. This does not apply to temporary structures like excavations, scaffolding etc of short duration.

R = **Response Reduction Factor**

To make normal buildings economical, design code allows some damage for reducing the cost of construction. This philosophy is introduced with the help of Response reduction factor, R.

The ratio (I/R***)*** shall not be greater than 1.0.

Depending on the perceived seismic damage performance of the structure by ductile or brittle deformations, the values of R[1)] for buildings are given in Table – 8.3 below:

Table – 8.3 Response Reduction Factor[1)], R, for Building Systems [IS 1893 (Part 1): 2002]

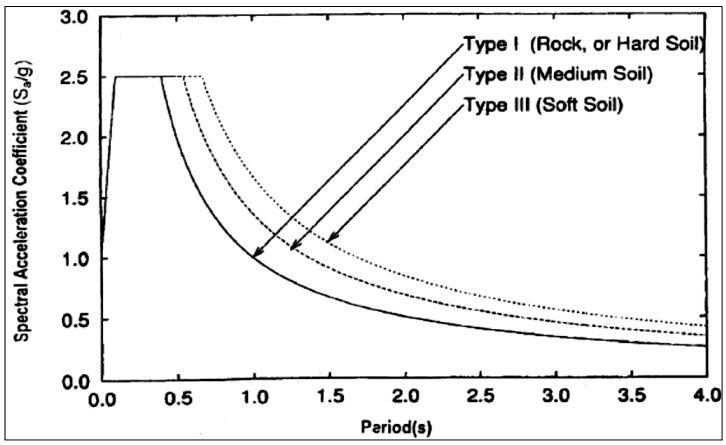
S. No.	Lateral Load Resisting System	R
Building Frame Systems		
(i)	Ordinary RC moment-resisting frame (OMRF)[2)]	3.0
(ii)	Special RC moment-resisting frame (SMRF)[3)]	5.0
(iii)	Steel frame with	
	a) Concentric braces	4.0
	b) Eccentric braces	5.0
(iv)	Steel moment resisting frame designed as per SP 6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

1) The values of response reduction factor are to be used for buildings with lateral load resisting elements, and not just for the lateral load resisting elements built in isolation.
2) OMRF (Ordinary Moment-Resisting Frame) are those designed and detailed as per IS 456 or IS 800 but not meeting ductile detailing requirement as per IS 13920 or SP 6 (6) respectively.
3) SMRF (Special Moment-Resisting Frame) defined in 4.15.2.
   As per 4.15.2, SMRF is a moment-resisting frame specially detailed to provide ductile behaviour and comply with the requirements given in IS 4326 or IS 13920 or SP 6 (6).
4) Buildings with shear walls also include buildings having shear walls and frames, but where:
   a) frames are not designed to carry lateral loads, or
   b) frames are designed to carry lateral loads but do not fulfil the requirements of 'dual systems'
5) Reinforcement should be as per IS 4326.
6) Prohibited in zones IV and V.
7) Ductile shear walls are those designed and detailed as per IS 13920.
8) Buildings with dual systems consist of shear walls (or braced frames) and moment resisting frames such that:
   a) the two systems are designed to resist the total design force in proportion to their lateral stiffness considering the interaction of the dual system at all floor levels, and
   b) the moment resisting frames are designed to independently resist at least 25 percent of the design seismic base shear

Sa/g = Average Response Acceleration Coefficient

Net shaking of a building is a combined effect of the energy carried by the earthquake at different frequencies and the natural period (T) of the building. Code reflects this by introducing a structural flexibility factor (Sa/g), also termed as Design Acceleration Coefficient.

Design Acceleration Coefficient (Sa/g) corresponding to 5% damping for different soil types, normalized to Peak Ground Acceleration (PAG), corresponding to natural period (T) of structure considering soil structure interaction given by Fig. 8.1 and associated expression given below:



For rocky, or hard soil sites

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

For medium soil sites

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

For soft soil sites

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

Fig. 8.1 Response Spectra for Rock and Soil Sitesfor5 % Damping [IS 1893 (Part 1): 2002]

Table – 8.4 gives multiplying factors for obtaining spectral values for various other damping.

Table – 8.4 Multiplying Factors for Obtaining Values for Other Damping [IS 1893 (Part 1): 2002]

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

8.5 कंपन की मौलिक प्राकृतिक अवधि / Fundamental Natural Period of Vibration

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:

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

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

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

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

Where, h = Height of building, in m as defined above.

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

8.6 डिजाइन पार्श्व बल / Design Lateral Force

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

$$V_B = A_h W$$

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

W= Seismic weight of the building

The design lateral force shall first be computed for the building as a whole. This design lateral force shall then be distributed to the various floor levels.

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

8.7 डिजाइन बल का वितरण / Distribution of Design Force

8.7.1 Vertical Distribution of Base Shear to Different Floor Levels

The Design Seismic Base Shear (***V_B)*** as computed above shall be distributed along the height of the building as per the following expression:

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

Where,

Q_i= Design lateral force at floor ***i,***
W_i = Seismic weight of floor i,
h_i = Height of floor ***i*** measured from base, and
n= Number of storeys in the building is the number of levels at which the masses are located.

8.7.2 Distribution of Horizontal Design Lateral Force to Different Lateral Force Resisting Elements

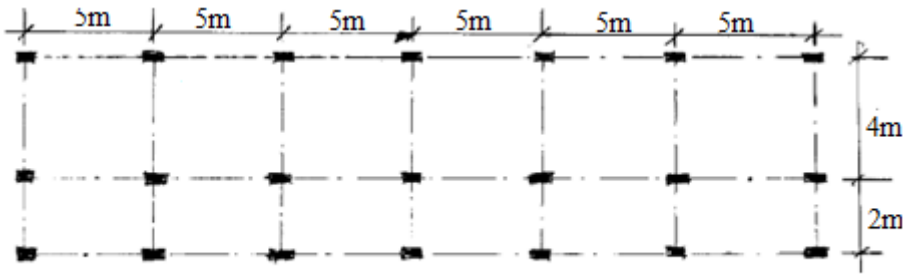
1. In case of buildings whose floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane shall be distributed to the various vertical elements of lateral force resisting system, assuming the floors to be infinitely rigid in the horizontal plane.
2. In case of building whose floor diaphragms cannot be treated as infinitely rigid in their own plane, the lateral shear at each floor shall be distributed to the vertical elements resisting the lateral forces, considering the in-plane flexibility of the diaphragms.

Notes:

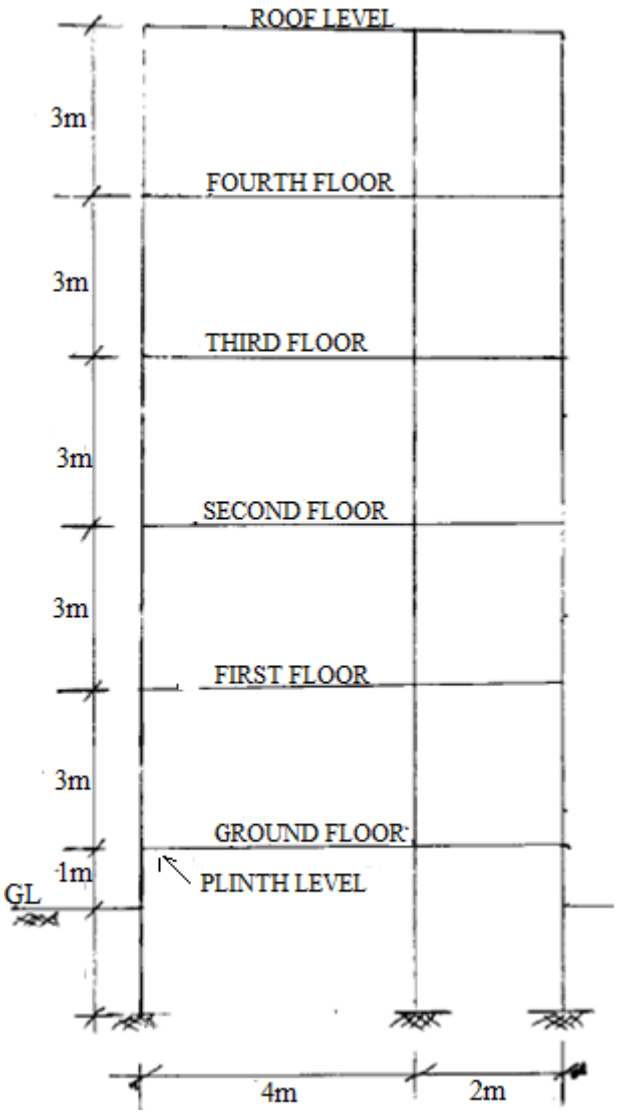
1. A floor diaphragm shall be considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.5 times the average displacement of the entire diaphragm.
2. Reinforced concrete monolithic slab-beam floors or those consisting of prefabricated/ precast elements with topping reinforced screed can be taken rigid diaphragms.

8.8 डिजाइन उदाहरण / Design Example – To determine Base Shear and its distribution along Height of Building

Exercise – 1: Determine the total base shear as per IS 1893(Part 1):2002 and distribute the base shear along the height of building to be used as school building in Bhuj, Gujrat and founded on Medium Soil. Basic parameters for design of building are as follows:



PLAN



ELEVATION

Solution:

Basic Data:

Following basic data is considered for analysis:

i)	Grade of Concrete	:	M-25
ii)	Grade of Steel	:	Fe – 415 Tor Steel
iii)	Density of Concrete	:	25 KN/m^3
iv)	Density of Brick Wall	:	20 KN/m^3
v)	Live Load for Roof	:	1.5 KN/m^2
vi)	Live Load for Floor	:	5.0 KN/m^2
vii)	Slab Thickness	:	150 mm
viii)	Beam Size		

(a) 5.00 m Span : 250 mm X 600 mm
(b) 4.00 m Span : 250 mm X 550 mm
(c) 2.00 m Span : 250 mm X 400 mm

ix) Column Size
(a) For 5.00 m Span : 300 mm X 600 mm
(b) For 2.00 m Span : 300 mm X 500 mm

Load Calculations:

1. **Dead Load:** Building is of G+4 Storeys.

Approximate Covered Area of Building on GF = 30 X 8 = 240 m^2
Approximate Covered Area of 1st, 2nd , 3rd & 4th Floor = 240 m^2
Total Floor Area = 5 X 240 = 1200 m^2
Roof Area = 1 X 240 = 240 m^2

(I) Slab:

Self Wt. of Slab = 0.15 X 25 = 3.75 KN/m^2
Wt. of Floor Finish = 1.25 KN/m^2

Total = 5.00 KN/m^2

Dead Load of Slab per Floor = 240 X 5 = 1200 KN
Dead Load of Slab on Roof = 240 X 5 = 1200 KN

(II) Beam:

Wt. per m of 250 X 600 mm beam = 0.25 X 0.60 X 25 = 3.75 KN/m
Wt. per m of 250 X 550 mm beam = 0.25 X 0.55 X 25 = 3.44 KN/m
Wt. per m of 250 X 400 mm beam = 0.25 X 0.40 X 25 = 2.50 KN/m

Weight of Beam per Floor
= (2 X 30 X 3.75) + (4 X 6 + 30) X 3.44 + (2 X 6 X 2.50)
= 225 + 185.76 + 30 = 440.76 KN [Say 441.00 KN]

(III) Column:

Wt. per m of 300 X 600 mm column = 0.30 X 0.60 X 25 = 4.50 KN/m
Wt. per m of 300 X 500 mm column = 0.30 X 0.50 X 25 = 3.75 KN/m

Weight of Column per Floor
= (12 X 3 X 4.50) + (6 X 3 X 3.75)
= 162 + 67.50 = 229.50 KN [Say 230.00 KN]

Walls:

250 mm thick wall (including plaster) are provided. Assuming 20% opening in the wall –

Wt. of Wall per m = 0.25 (Wall Thickness) X 0.80 (Reduction) X 20 (Density) X 2.50 (Clear Height)
= 10.00 KN/m

Wt. of Parapet Wall per m = 0.125 (Wall Thickness) X 20 (Density) X 1.00 (Clear Height) = 2.50 KN/m

Wt. of Wall per Floor = 10.00 X [30 X 3 + 2 X 2] = 940 KN

Wt. of Wall at Roof = 2.50 X [30 X 2 + 8 X 2] = 190 KN

Total Dead Load –

(i) For Floor = Slab + Beam + Column + Wall
= 1200 + 441 + 230 + 940 = 2811 KN

(ii) For Roof = 1200 (Slab) + 441 (Beam) + 190 (Parapet) = 1831 KN

2. **Live Load:** Live Load on Floor = 4.0 KN/m^2

As per Table – 8 in Cl. 7.3.1 of IS 1893 (Part 1):2002, "%age of Imposed Load to be considered in Seismic Weight calculation":

(i) Up to & including 3.00 KN/m^2 = 25%
(ii) Above 3.00 KN/m^2 = 50%

Live Load on Floors to be considered for Earthquake Force = 2.00 KN/m^2 [i.e. 50% of 4.0 KN/m^2]

As per Cl. 7.3.2 of IS 1893 (Part 1):2002, for calculating the design seismic force of the structure, the imposed load on roof need not be considered.

Therefore, Live Load on Roof = 0.00 KN

Seismic Weight due to Live Load

(i) For Floor = 240 X 2 = 480 KN
(ii) For Roof = 0.00 KN

3. **Seismic Weight of Building:**

As per Cl. 7.4 of IS 1893 (Part 1):2002

(i) For Floor = D.L. of Floor + L.L. on Floor
= 2811 + 480 = 3291 KN
(ii) For Roof = 1831 + 0.00 = 1831 KN

Total Seismic Weight of Building = 5 X 3291 + 1 X 1831

W = 18286 KN

4. **Determination of Base Shear:**

As per Cl. 7.5 of IS 1893 (Part 1):2002, $V_B = A_h W$

Where,

V_B = Base Shear

A_h = Design Horizontal Acceleration Spectrum

$= \frac{Z}{2} \frac{I}{R} \frac{Sa}{g}$

W = Seismic Wt. of Building

Total height of Building above Ground Level = 15.00 m

As per Cl. 7.6 of IS 1893 (Part 1):2002, Fundamental Natural Period of Vibration for RC Frame Building is

$T_a = 0.075\ h^{0.75}$

$= 0.075\ (15)^{0.75}$

$= 0.572$ Sec.

Average Response Acceleration Coefficient for 5% damping and Type II soil = 2.5

Bhuj, Gujrat is in Seismic Zone V.

As per Table – 2 of IS 1893 (Part 1):2002,

Zone Factor, Z = *0.36*

As per Table – 6 of IS 1893 (Part 1):2002,

Impedance Factor, I = 1.50

As per Table – 7 of IS 1893 (Part 1):2002,

Response Reduction Factor for Ordinary RC Moment-resisting Frame (OMRF) Building R = 3.00

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

$= (0.36/2)\ X\ (1.5/3.0)\ X\ (2.5)$

$= 0.225$

Base Shear, $V_B = A_h W$

$= 0.225\ X\ 18286$

$= 4114.35$ KN [Say 4114.00 KN]

5. Vertical Distribution of Base Shear to Different Floors Levels

As per Cl. 7.7.1 of IS 1893 (Part 1):2002,

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

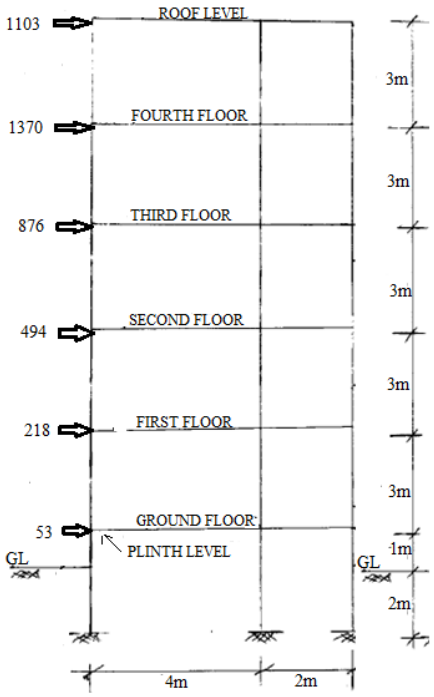
Where,

Q_i= Design lateral force at floor *i*,
W_i = Seismic weight of floor *i*,
h_i = Height of floor *i* measured from base, and
n= Number of storeys in the building is the number of levels at which the masses are located.

V_B = 4114 KN

Storey No.	Mass No.	W_i	h_i	$W_i h_i^2$	$f = \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$	$Q_i = V_B \times f$ (KN)	V_i (KN)
Roof	1	1831	18	593244	0.268	1103	1103
4th Floor	2	3291	15	740475	0.333	1370	2473
3rd Floor	3	3291	12	473904	0.213	876	3349
2nd Floor	4	3291	9	266571	0.120	494	3843
1st Floor	5	3291	6	118476	0.053	218	4061
Ground	6	3291	3	29619	0.013	53	4114

$\sum_{j=1}^{n} W_j h_j^2$ = 2222289



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अध्याय /Chapter – 9

ढाँचागत संरचना का निर्माण / Construction of Framed Structure

9.1 गुरुत्वाकर्षण लोडिंग और भूकंप लोडिंग में आर सी बिल्डिंग का व्यवहार / Behaviour of RC Building in Gravity Loading and Earthquake Loading

In recent times, *reinforced concrete* buildings have become common in India, particularly in towns and cities. A typical RC building consists of horizontal members (*beams* and *slabs*) and vertical members (*columns* and *walls*). The system is supported by *foundations* that rest on ground. The RC frame participates in resisting the gravity and earthquake forces as illustrated in Fig. 9.1.

- *Gravity Loading*

1. Load due to self weight and contents on buildings cause RC frames to bend resulting in stretching and shortening at various locations.
2. Tension is generated at surfaces that stretch and compression at those that shorten.
3. Under gravity loads, tension in the beams is at the bottom surface of the beam in the central location and is at the top surface at the ends.

- *Earthquake Loading*

1. It causes tension on beam and column faces at locations different from those under gravity loading; the relative levels of this tension (in technical terms, bending moment) generated in members are shown in Figure.
2. The level of bending moment due to earthquake loading depends on severity of shaking and can exceed that due to gravity loading.
3. Under strong earthquake shaking, the beam ends can develop tension on either of the top and bottom faces.
4. Since concrete cannot carry this tension, steel bars are required on both faces of beams to resist reversals of bending moment.
5. Similarly, steel bars are required on all faces of columns too.

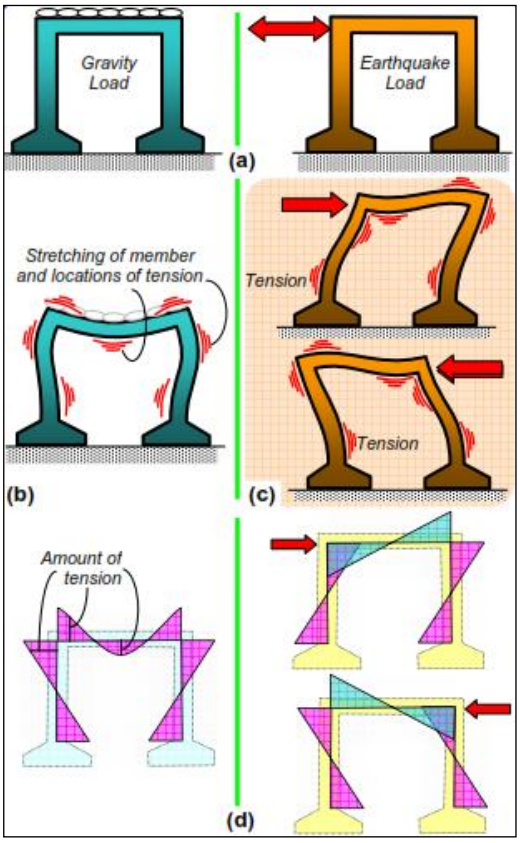


Fig. 9.1 Earthquake shaking reverses tension and compression in members – reinforcement is required on both faces of members

9.2 प्रबलित कंक्रीट इमारतों पर क्षैतिज भूकंप का प्रभाव / Effect of Horizontal Earthquake Force on RC Buildings

Earthquake shaking generates inertia forces in the building, which are proportional to the building mass. Since most of the building mass is present at floor levels, earthquake-induced inertia forces primarily develop at the floor levels. These forces travel downwards - through *slab and beams* to *columns and walls*, and then to the foundations from where they are dispersed to the ground (Fig. 9.2).

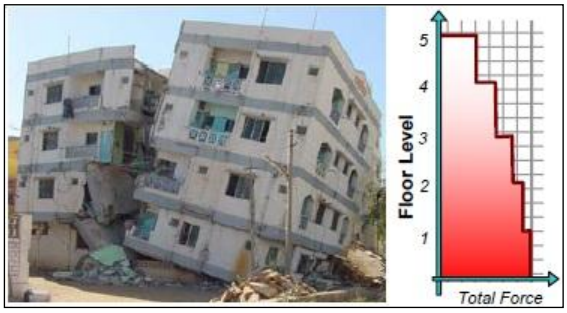


Fig. 9.2 Total horizontal earthquake force in a building increase downwards along its height

As inertia forces accumulate downwards from the top of the building, the columns and walls at lower storeys experience higher earthquake-induced forces and are therefore designed to be stronger than those in storeys above.

9.3 क्षमता डिजाइन संकल्पना / Capacity Design Concept

(i) Let us take two bars of same length & Cross-sectional area
1st bar – Made up of Brittle Material
2nd bar – Made up of Ductile Material

(ii) Pull both the bars until they break

(iii) Plot the graph of bar force F *versus* bar elongation. Graph will be as given in Fig. 9.3.

(iv) It is observed that –

a) Brittle bar breaks suddenly on reaching its maximum strength at a relatively small elongation.

b) Ductile bar elongates by a large amount before it breaks.

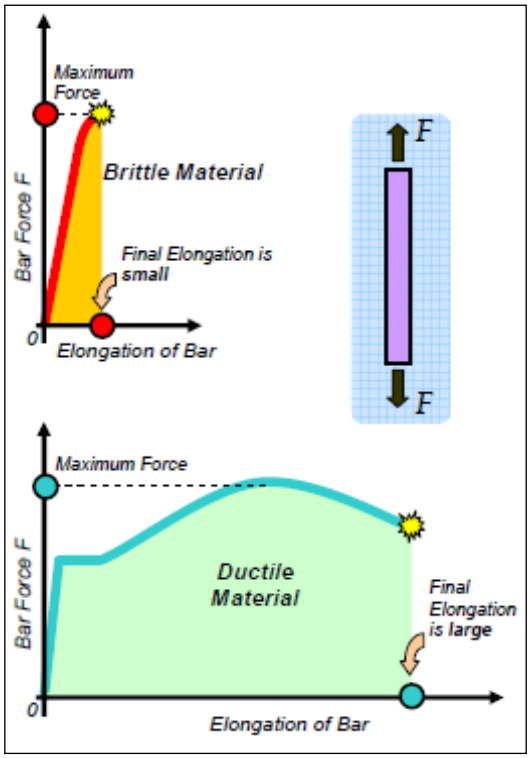


Fig. 9.3 Tension Test on Materials – *ductile versus brittle materials*

Materials used in building construction are steel, masonry and concrete. Steel is ductile material while masonry and concrete are brittle material.

Capacity design concept ensures that the brittle element will remain elastic at all loads prior to the failure of ductile element. Thus, brittle mode of failure i.e. sudden failure has been prevented.

The concept of capacity design is used to ensure post-yield ductile behaviour of a structure having both ductile and brittle elements. In this method, the ductile elements are designed and detailed for the design forces. Then, an upper-bound strength of the ductile elements is obtained. It is then expected that if the seismic force keeps increasing, a point will come when these ductile elements will reach their upper-bound strength and become plastic. Clearly, it is necessary to ensure that even at that level of seismic force, the brittle elements remain safe.

9.4 लचीलापन और ऊर्जा का अपव्यय / Ductility and Energy Dissipation

From strength point of view, overdesigned structures need not necessarily demonstrate good ductility. By ductility of Moment Resisting Frames (MRF), one refers to the capacity of the structure and its elements to undergo large deformations without loosing either strength or stiffness. It is important for a building in a seismic zone to be resilient i.e. absorb the shock from the ground and dissipate this energy uniformly throughout the structure.

In MRFs, the dissipation of the input seismic energy takes place in the form of flexural yielding, and resulting in the formation of plastic moment hinges. Due to cyclic nature of the flexural effects, both positive and negative plastic moment hinges may be formed.

9.5 'मजबूत स्तंभ - कमजोर बीम' फ्लोस्फ़ी / 'Strong Column – Weak Beam' Philosophy

Because beams are usually capable of developing large ductility than columns which are subjected to significant compressive loads, many building frames are designed based on the 'strong column – weak beam' philosophy. Figure shows that for a frame designed according to the 'strong column – weak beam' philosophy to form a failure mechanism, many more plastic hinges have to be formed than a frame designed according to the "weak column – strong beam' philosophy. The frames designed by the former approach dissipate greater energy before failure.

When this strategy is adopted in design, damage is likely to occur *first* in beams. When beams are detailed properly to have large ductility, the building as a whole can deform by large amounts despite progressive damage caused due to consequent yielding of beams.

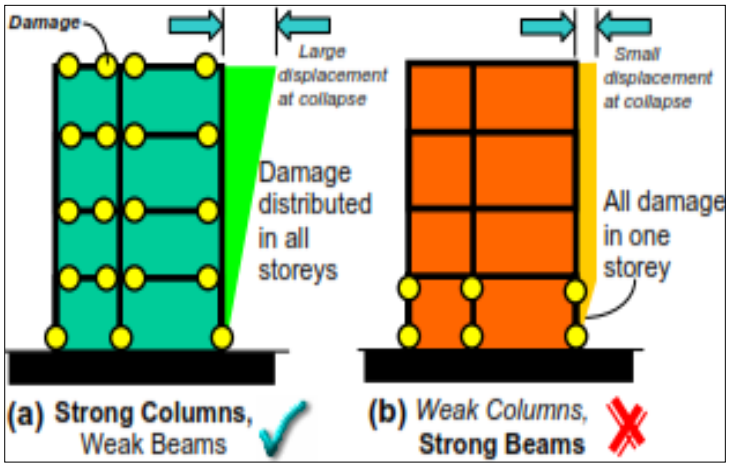


Fig. 9.4 Two distinct designs of buildings that result in different earthquake performances–columns should be stronger than beams

Note: If columns are made weaker, they suffer severe local damage, at the top and bottom of a particular storey. This localized damage can lead to collapse of a building, although columns at storeys above remain almost undamaged (Fig. 9.4).

For a building to remain safe during earthquake shaking, columns (which receive forces from beams) should be stronger than beams, and foundations (which receive forces from columns) should be stronger than columns.

9.6 कठोर डायाफ्राम क्रिया / Rigid Diaphragm Action

When beams bend in the vertical direction during earthquakes, these thin slabs bend along with them. And, when beams move with columns in the horizontal direction, the slab usually forces the beams to move together with it. In most buildings, the geometric distortion of the slab is negligible in the horizontal plane; this behaviour is known as the *rigid diaphragm action*. This aspect must be considered during design. (Fig. 9.5)

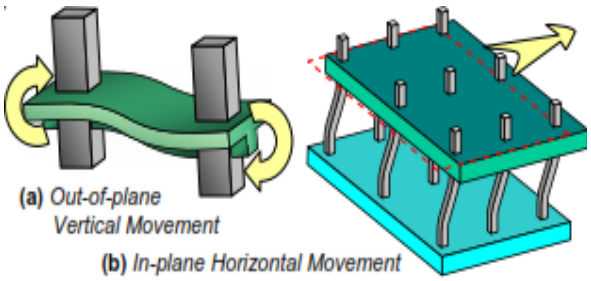


Fig. 9.5 Floor bends with the beam but moves all columns at that level together.

9.7 सॉफ्ट स्टोरी बिल्डिंग के साथ - ओपन ग्राउंड स्टोरी बिल्डिंग जो कि भूकंप के समय कमजोर होती है / Building with Soft storey – Open Ground Storey Building that is vulnerable in Earthquake

The buildings that have been constructed in recent times with a special feature - the ground storey is left *open* for the purpose of parking, *i.e.*, columns in the ground storey do not have any partition walls (of either masonry or RC) between them, are called *open ground storey buildings* or *buildings on stilts*.

An open ground storey building (Fig. 9.6), having *only columns* in the ground storey and *both partition walls and columns* in the upper storeys, have two distinct characteristics, namely:

(a) It is relatively *flexible* in the ground storey, *i.e.*, the relative horizontal displacement it undergoes in the ground storey is much larger than what each of the storeys above it does. This flexible ground storey is also called *soft storey*.

(b) It is relatively *weak* in ground storey, *i.e.*, the total horizontal earthquake force it can carry in the ground storey is significantly smaller than what each of the storeys above it can carry. Thus, the open ground storey may also be a *weak storey*.

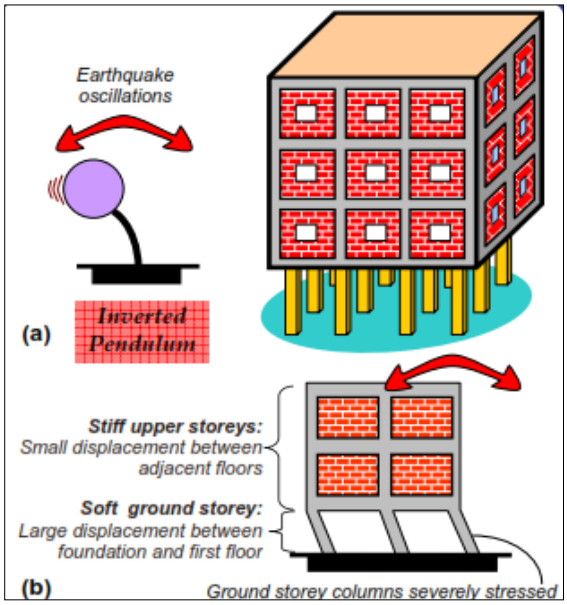


Fig. 9.6 Upper storeys of open ground storey building move together as a single block – such buildings are like inverted pendulums

The collapse of more than a hundred RC frame buildings with open ground storeys at Ahmedabad (~225km away from epicenter) during the 2001 Bhuj earthquake has emphasized that such buildings are *extremely* vulnerable under earthquake shaking.

After the collapses of RC buildings in 2001 Bhuj earthquake, the Indian Seismic Code IS 1893 (Part 1): 2002 has included special design provisions related to soft storey buildings.

- Firstly, it specifies when a building should be considered as a *soft* and a *weak storey building*.
- Secondly, it specifies higher design forces for the soft storey as compared to the rest of the structure.

The Code suggests that the forces in the columns, beams and shear walls (if any) under the action of seismic loads specified in the code, may be obtained by considering the *bare frame* building (without any infills). However, beams and columns *in the open ground storey* are required to be designed for 2.5 times the forces obtained from this bare frame analysis. (Fig. 9.7)

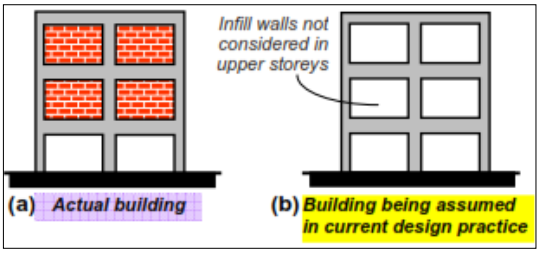


Fig. 9.7 Open ground storey building –assumptions made in current design practice are not consistent with the actual structure

For all *new RC frame buildings*, the best option is to avoid such sudden and large decrease in stiffness and/or strength in any storey; it would be ideal to build walls (either masonry or RC walls) in the ground storey also. Designers can avoid dangerous effects of flexible and weak ground storeys by ensuring that too many walls are not discontinued in the ground storey, *i.e.*, the drop in stiffness and strength in the ground storey level is not abrupt due to the absence of infill walls. (Fig. 9.8)

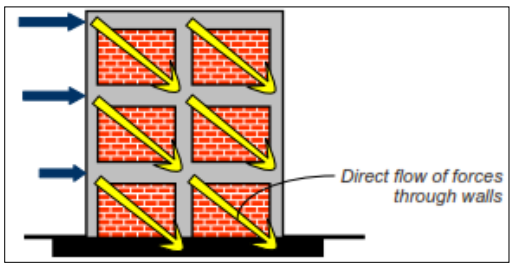


Fig. 9.8 Avoiding open ground storey problem – continuity of walls in ground storey is preferred

The *existing open ground storey buildings* need to be strengthened suitably so as to prevent them from collapsing during strong earthquake shaking. The owners should seek the services of qualified structural engineers who are able to suggest appropriate solutions to increase seismic safety of these buildings.

9.7.1 भरी हुई दीवारें / In-Fill Walls

When columns receive horizontal forces at floor levels, they try to move in the horizontal direction, but masonry walls tend to resist this movement. Due to their heavy weight and thickness, these walls attract rather large horizontal forces. However, since masonry is a brittle material, these walls develop cracks once their ability to carry horizontal load is exceeded. Thus, infill walls act like sacrificial fuses in buildings; they develop

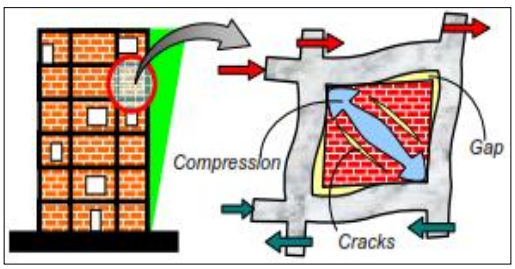


Fig. 9.9 Infill walls move together with the columns under earthquake shaking

cracks under severe ground shaking but help share the load of the beams and columns until cracking. Earthquake performance of infill walls is enhanced by mortars of good strength, making proper masonry courses, and proper packing of gaps between RC frame and masonry infill walls (Fig. 9.9).

9.8 भूकंप के दौरान लघु कॉलम वाली इमारतों का व्यवहार / Behavior of Buildings with Short Columns during Earthquakes

During past earthquakes, reinforced concrete (RC) frame buildings that have columns of different heights within one storey, suffered more damage in the shorter columns as compared to taller columns in the same storey.

Two examples of buildings with short columns are shown in Fig. 9.10 – (a) buildings on a sloping ground and (b) buildings with a mezzanine floor

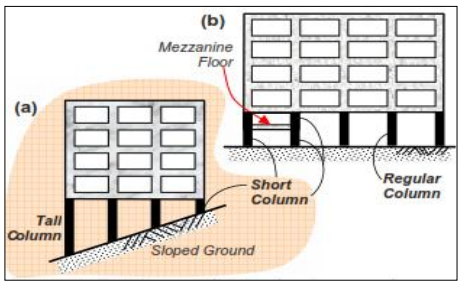


Fig. 9.10 Buildings with short columns – two explicit examples of common occurrences

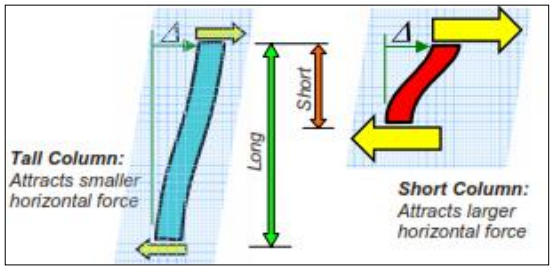


Fig. 9.11 Short columns are stiffer and attract larger forces during earthquakes – this must be accounted for in design

Poor behaviour of short columns is due to the fact that in an earthquake, a tall column and a short column of same cross-section move horizontally by same amount.

However, the short column is stiffer as compared to the tall column, and it attracts larger earthquake force. Stiffness of a column means resistance to deformation – the larger is the stiffness, larger is the force required to deform it. If a short column is not adequately designed for such a large force, it can suffer significant damage during an earthquake. This behaviour is called *Short Column Effect*. (Fig. 9.11)

In new buildings, *short column effect* should be avoided to the extent possible during *architectural design* stage itself. When it is not possible to avoid short columns, this effect must be addressed in structural design. The IS:13920-1993for ductile detailing of RC structures requires special confining reinforcement to be provided over the *full height* of columns that are likely to sustain short column effect.

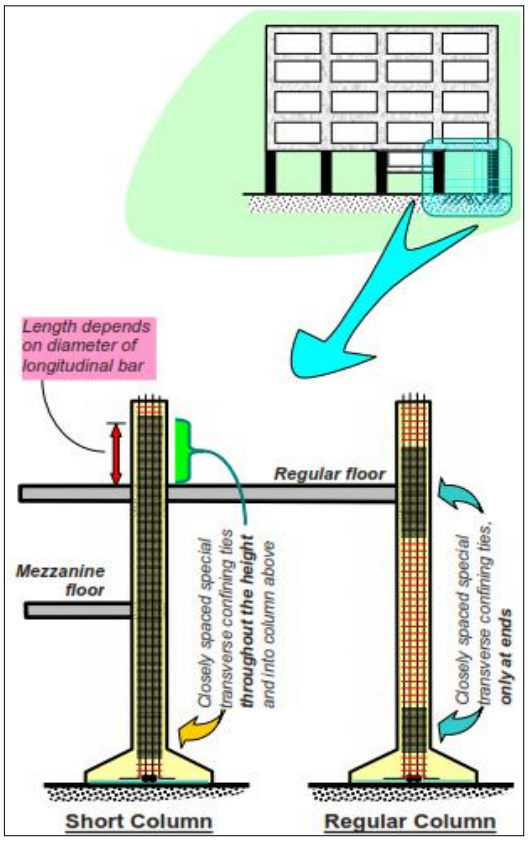


Fig. 9.12 Details of reinforcement in a building with short column effect in some columns –additional special requirements are given in IS:13920- 1993 for the short columns

The special confining reinforcement (*i.e.,* closely spaced closed ties) must extend beyond the short column into the columns vertically above and below by a certain distance as shown in Fig. 9.12.

In existing buildings with short columns, different retrofit solutions can be employed to avoid damage in future earthquakes. Where walls of partial height are present, the simplest solution is to close the openings by building a wall of full height – this will eliminate the short column effect. If that is not possible, short columns need to be strengthened using one of the well established retrofit techniques. The retrofit solution should be designed by a qualified structural engineer with requisite background.

9.9 भूकंप प्रतिरोधी इमारतों की लचीलापन आवश्यकताएँ / Ductility requirements of Earthquake Resistant Buildings

The primary members of structure such as beams and columns are subjected to stress reversals from earthquake loads. The reinforcement provided shall cater to the needs of reversal of moments in beams and columns, and at their junctions.

Earthquake motion often induces forces large enough to cause inelastic deformations in the structure. If the structure is brittle, sudden failure could occur. But if the structure is made to behave ductile, it will be able to sustain the earthquake effects better with some deflection larger than the yield deflection by absorption of energy. Therefore, besides the design for strength of the frame, ductility is also required as an essential element for safety from sudden collapse during severe shocks.

The ductility requirements will be deemed to be satisfied if the conditions given as in the following are achieved.

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

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

However, high strength deformed steel bars, produced by the thermo-mechanical treatment process, of grades Fe 500 and Fe 550, having elongation more than 14.5 percent and conforming to other requirements of IS 1786 : 1985 may also be used for the reinforcement.

9.10 बीम जिन्हें आर सी इमारतों में भूकंप बलों का विरोध करने के लिए डाला जाता है / Beams that are required to resist Earthquake Forces in RC Buildings

In RC buildings, the vertical and horizontal members (*i.e.,* the *columns* and *beams)* are built integrally with each other. Thus, under the action of loads, they act together as a *frame* transferring forces from one to another.

Beams in RC buildings have two sets of steel reinforcement (Fig. 9.13), namely:

(a) long straight bars (called *longitudinal bars*) placed along its length, and

(b) closed loops of small diameter steel bars (called *stirrups*)placed vertically at regular intervals along its full length.

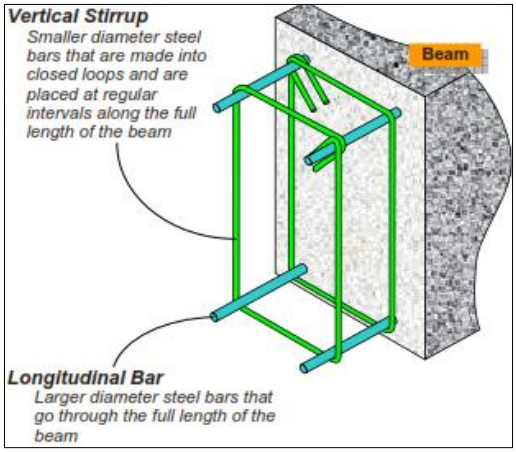


Fig. 9.13 Steel reinforcement in beams – stirrups prevent longitudinal bars from bending outwards

Beams sustain two basic types of failures, namely:

(i) *Flexural (or Bending) Failure*

As the beam sags under increased loading, it can fail in two possible ways (Fig. 9.14).

- If relatively more steel is present on the tension face, concrete *crushes in compression*; this is a *brittle* failure and is therefore undesirable.

- If relatively less steel is present on the tension face, the steel yields first (it *keeps elongating* but does not snap, as steel has ability to stretch large amounts before it snaps and redistribution occurs in the beam until eventually the concrete *crushes in compression*; this is a *ductile* failure and hence is desirable. *Thus, more steel on tension face is not necessarily desirable.* The ductile failure is characterized with many vertical cracks starting from the stretched beam face, and going towards its mid-depth.

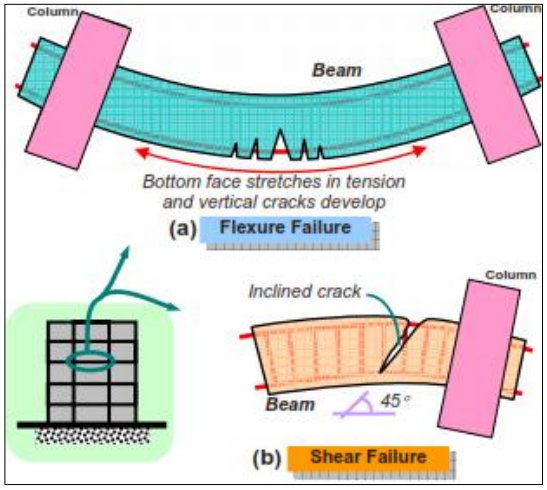


Fig. 9.14 Two types of damage in a beam: flexure damage is preferred. Longitudinal bars resist the tension forces due to bending while vertical stirrups resist shear forces

(ii) *Shear Failure*

A beam may also fail due to shearing action. A shear crack is inclined at 45° to the horizontal; it develops at mid-depth near the support and grows towards the top and bottom faces. Closed loop stirrups are provided to avoid such shearing action. Shear damage occurs when the area of these stirrups is *insufficient*. Shear failure is brittle, and therefore, shear failure must be avoided in the design of RC beams.

Longitudinal bars are provided to resist flexural cracking on the side of the beam that stretches. Since both top and bottom faces stretch during strong earthquake shaking, longitudinal steel bars are required on both faces at the ends and on the bottom face at mid-length (Fig. 9.15).

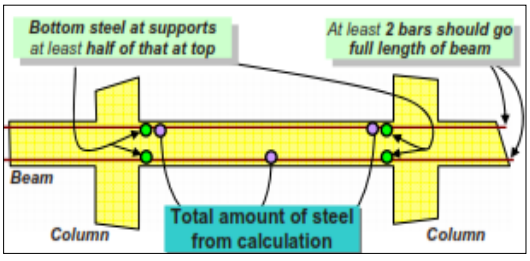


Fig. 9.15 Location and amount of longitudinal steel bars in beams – these resist tension due to flexure

- Designing a beam involves the selection of *its material properties* (*i.e.*, grades of steel bars and concrete) and *shape and size*; these are usually selected as a part of an overall design strategy of the whole building.
- The *amount and distribution of steel* to be provided in the beam must be determined by performing design calculations as per IS: 456-2000 and IS: 13920-1993.

9.11 फ्लेक्स्चरल मेम्बर्स के लिए सामान्य आवश्यकताएँ / General Requirements for Flexural Members

These members shall satisfy the following requirements.

- The member shall preferably have a width-to-depth ratio of more than 0.3.
- The width of the member shall not be less than 200 mm.
- The depth D of the member shall preferably be not more than 1/4 of the clear span.
- The factored axial stress on the member under earthquake loading shall not exceed $0.1f_{ck}$.

9.11.1 अनुदैर्ध्य सुदृढीकरण / Longitudinal Reinforcement

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

b) The tension steel ratio on any face, at any section, shall not be less than $\rho_{min} = 0.24$; where f_{ck} and f_y are in MPa.

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

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

- In an external joint, both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90 degree bend(s) (as shown in Fig. 9.16). In an internal joint, both face bars of the beam shall be taken continuously through the column.

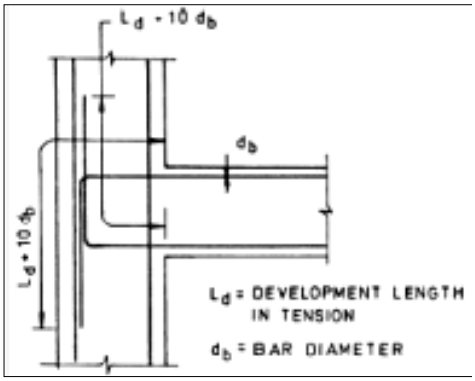


Fig. 9.16 Anchorage of Beam Bars in an External Joint (IS 13920: 1993)

9.11.2 अनुदैर्ध्य सुदृढीकरण की स्प्लाइसिंग / Splicing of longitudinal reinforcement

- The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at a spacing not exceeding 150 mm (as shown in Fig. 9.17). The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided (a) within a joint, (b) within a distance of 2*d* from joint face, and (c) within a quarter length of the member where flexural yielding may generally occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.

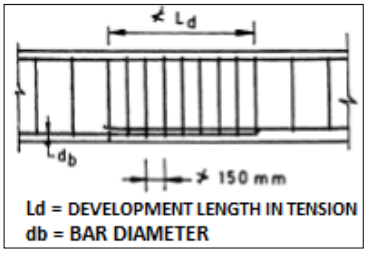


Fig. 9.17 Lap, Splice in Beam (IS 13920: 1993)

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

9.11.3 वेब सुदृढीकरण / Web Reinforcement

Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end that is embedded in the confined core [as shown in (a) of Fig. 9.18]. In compelling circumstances, it may also be made up of two pieces of reinforcement; a U-stirrup with a 135° hook and a 10 diameter extension (but not < 75 mm) at each end, embedded in the confined core and a crosstie [as shown in (b) of Fig. 9.18]. A crosstie is a bar having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

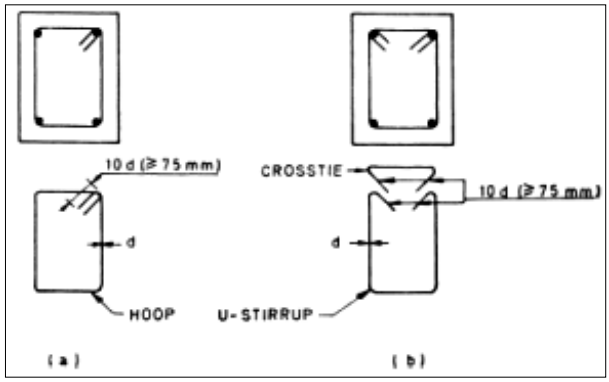


Fig. 9.18 Beam Web Reinforcement (IS 13920: 1993)

9.12 कॉलम जिन्हें आर सी इमारतों में भूकंप बलों का विरोध करने के लिए डाला जाता है / Columns that are required to resist Earthquake Forces in RC Buildings

Columns, the vertical members in RC buildings, contain two types of steel reinforcement, namely:

(a) long straight bars (called *longitudinal bars*) placed vertically along the length, and

(b) closed loops of smaller diameter steel bars (called transverse *ties*) placed horizontally at regular intervals along its full length.

Columns can sustain two types of damage, namely *axial-flexural (or combined compression-bending) failure* and *shear failure*. Shear damage is brittle and must be avoided in columns by providing transverse ties at close spacing.

Closely spaced horizontal closed ties (Fig. 9.19) help in three ways, namely

(i) they carry the horizontal shear forces induced by earthquakes, and thereby resist diagonal shear cracks,

(ii) they hold together the vertical bars and prevent them from excessively bending outwards(in technical terms, this bending phenomenon is called *buckling*), and

(iii) they contain the concrete in the column within the closed loops. The ends of the ties must be bent as 135° hooks. Such hook ends prevent opening of loops and consequently bulging of concrete and buckling of vertical bars.

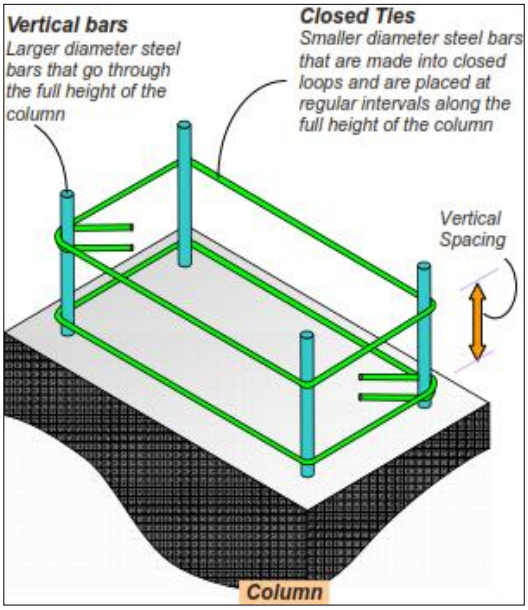


Fig. 9.19 Steel reinforcement in columns – closed ties at close spacing improve the performance of column under strong earthquake shaking

Construction drawings with clear details of closed ties are helpful in the effective implementation at construction site. In columns where the spacing between the corner bars exceeds 300mm, the Indian Standard prescribes *additional* links with *180*° hook ends for ties to be effective in holding the concrete in its place and to prevent the buckling of vertical bars. These links need to go around both vertical bars and horizontal closed ties (Fig. 9.20); special care is required to implement this properly at site.

Designing a column involves selection of *materials to be used* (*i.e.* grades of concrete and steel bars), choosing *shape and size of the cross-section*, and calculating *amount and distribution of steel reinforcement*. The first two aspects are part of the overall design strategy of the whole building. The IS 13920 : 1993 requires columns to be at least 300mm wide. A column width of up to 200 mm is allowed if unsupported length is less than 4m and beam length is less than 5m. Columns that are required to resist earthquake forces must be designed to prevent *shear* failure by a skillful selection of reinforcement.

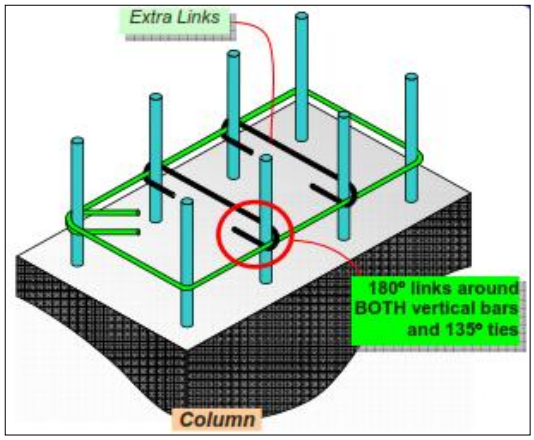


Fig. 9.20 Extra links are required to keep the concrete in place – 180° links are necessary to prevent the135° tie from bulging outwards

9.13 एक्सीयल लोडेड मेंबर्स के लिए सामान्य आवश्यकताएँ / General Requirements for Axial Loaded Members

- These requirements apply to frame members which have a factored axial stress in excess of 0.1 f_{ck} under the effect of earthquake forces.
- The minimum dimension of the member shall not be less than 200 mm. However, in frames which have beams with centre to centre span exceeding 5 m or columns of unsupported length exceeding 4 m, the shortest dimension of the column shall not be less than 300 mm.
- The ratio of the shortest cross sectional dimension to the perpendicular dimension shall preferably not be less than 0.4.

9.13.1 अनुदैर्ध्य सुदृढीकरण / Longitudinal Reinforcement

- Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall be spliced at one section.
- Any area of a column that extends more than 100 mm beyond the confined core due to architectural requirements shall be detailed in the following manner.

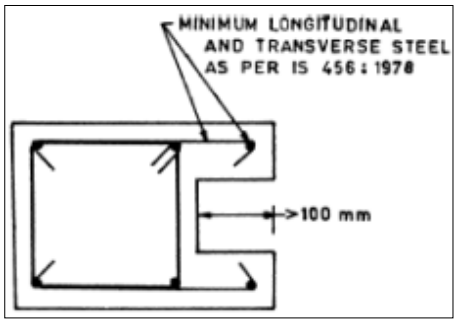


Fig. 9.21 Reinforcement requirement for Column with more than 100 mm projection beyond core(IS 13920: 1993)

a) In case the contribution of this area to strength has been considered, then it will have the minimum longitudinal and transverse reinforcement as per IS 13920: 1993.
b) However, if this area has been treated as non-structural, the minimum reinforcement requirements shall be governed by IS 456: 1978 provisions minimum longitudinal and transverse reinforcement, as per IS 456: 1978 (as shown in Fig. 9.21).

9.13.2 अनुप्रस्थ सुदृढीकरण / Transverse Reinforcement

- Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end, that is embedded in the confined core [as shown in (A) of Fig. 9.22].

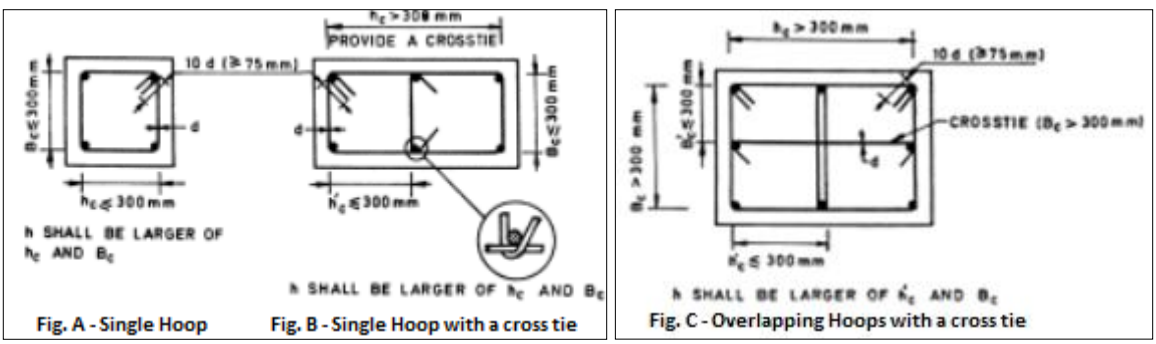


Fig. 9.22 Transverse Reinforcement in Column (IS 13920: 1993)

- The parallel legs of rectangular hoop shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided [as shown in (B) of Fig. 9.22]. Alternatively, a pair of overlapping hoops may be provided within the column [as shown in (C) of Fig. 9.22]. The hooks shall engage peripheral longitudinal bars.

- The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided, as per Para 9.15 below.

9.14 बीम-कॉलम जोड़ जो आर सी भवनों में भूकंप बलों का विरोध करते हैं / Beam-Column Joints that resist Earthquakes Forces in RC Buildings

In RC buildings, portions of *columns* that are common to *beams* at their intersections are called *beam column joints* (Fig. 9.23). The joints have *limited force carrying capacity*. When forces larger than these are applied during earthquakes, joints are severely damaged. Repairing damaged joints is difficult, and so damage must be avoided. Thus, beam-column joints must be designed to resist earthquake effects.

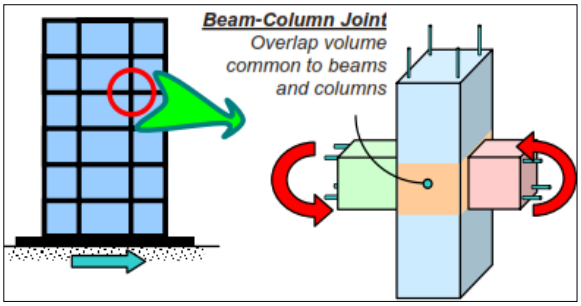


Fig. 9.23 Beam-Column Joints are critical parts of a building – they need to be designed

Under earthquake shaking, the beams adjoining a joint are subjected to moments in the same (clockwise or counter-clockwise) direction.

- Under these moments, the top bars in the beam-column joint are pulled in one direction and the bottom ones in the opposite direction. These forces are balanced by bond stress developed between concrete and steel in the joint region. (Fig. 9.24)

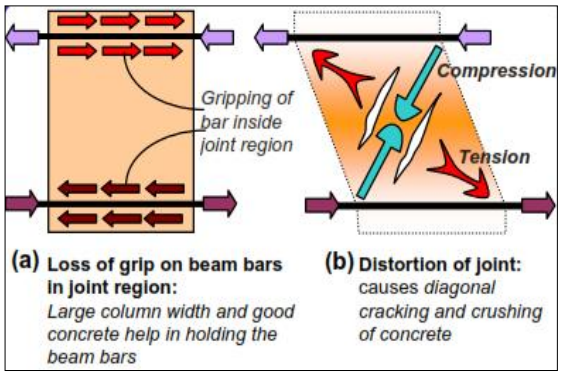


Fig.9.24 Pull-push forces on joints cause two problems – these result in irreparable damage in joints under strong seismic shaking

- If the column is not wide enough or if the strength of concrete in the joint is low, there is insufficient grip of concrete on the steel bars. In such circumstances, the bar slips inside the joint region, and beams loose their capacity to carry load. Further, under the action of the above pull-push forces at top and bottom ends, joints undergo geometric distortion; one diagonal length of the joint elongates and the other compresses.

- If the column cross-sectional size is insufficient, the concrete in the joint develops diagonal cracks.

9.14.1 बीम-कॉलम जोड़ मजबूत करने के लिए सामान्य आवश्यकताएँ / General Requirements for Reinforcing the Beam-Column Joint

Diagonal cracking and crushing of concrete in joint region should be prevented to ensure good earthquake performance of RC frame buildings. (Fig. 9.25)

- Using *large column sizes* is the most effective way of achieving this.
- In addition, *closely spaced closed-loop steel ties* are required around column bars to hold together concrete in joint region and to resist shear forces.
- *Intermediate column bars* also are effective in confining the joint concrete and resisting horizontal shear forces. Providing closed-loop ties in the joint requires some extra effort.
- IS: 13920–1993 recommends continuing the transverse loops around the column bars through the joint region.

In practice, this is achieved by preparing the cage of the reinforcement (both *longitudinal bars* and *stirrups*) of all beams at a floor level to be prepared on top of the beam formwork of that level and lowered into the cage. (Fig. 9.26)

However, this may not always be possible particularly when the beams are long and the entire reinforcement cage becomes heavy.

The gripping of beam bars in the joint region is improved *first* by using columns of reasonably large cross-sectional size.

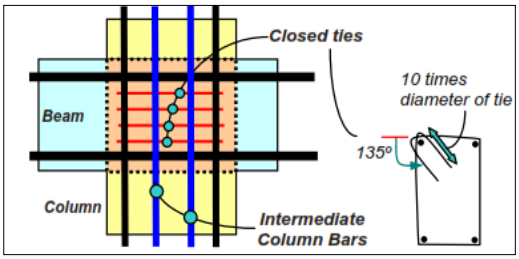


Fig. 9.25 Closed loop steel ties in beam-column joints – such ties with 135° hooks resist the ill effects of distortion of joints

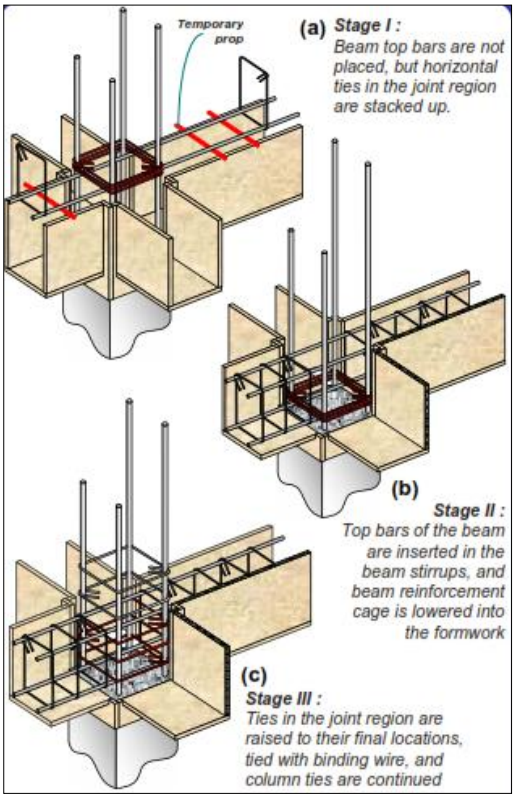


Fig. 9.26 Providing horizontal ties in the joints – three-stage procedure is required

The Indian Standard IS: 13920-1993 requires building columns in seismic zones III, IV and V to be at least *300mm* wide in each direction of the cross-section when they support beams that are longer than *5m* or when these columns are taller than *4m* between floors (or beams).

In exterior joints where beams terminate at columns, longitudinal beam bars need to be anchored into the column to ensure proper gripping of bar in joint. The length of anchorage for a bar of grade Fe415 (characteristic tensile strength of 415MPa) is about *50* times its diameter. This

length is measured from the face of the column to the end of the bar anchored in the column. (Fig. 9.27)

- In columns of small widths and when beam bars are of large diameter (Fig. 9.28(a)), a portion of beam top bar is embedded in the column that is cast up to the soffit of the beam, and a part of it overhangs. It is difficult to hold such an overhanging beam top bar in position while casting the column up to the soffit of the beam. Moreover, the vertical distance beyond the *90º* bend in beam bars is not very effective in providing anchorage.

- On the other hand, if column width is large, beam bars may not extend below soffit of the beam (Fig. 9.28 (b)). Thus, it is preferable to have columns with sufficient width.

- In interior joints, the beam bars (both top and bottom) need to go through the joint without any cut in the joint region. Also, these bars must be placed within the column bars and with no bends.

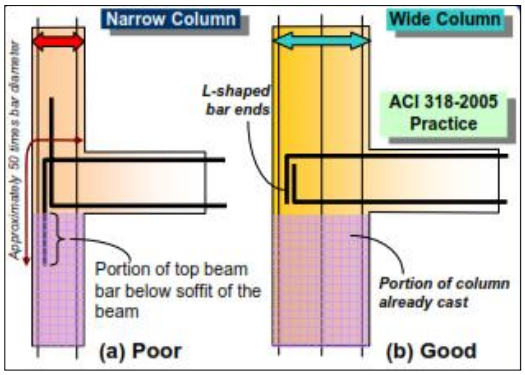


Fig. 9.27 Anchorage of beam bars in exterior joints – diagrams show elevation of joint region

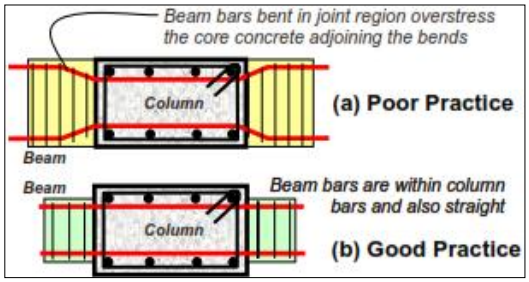


Fig. 9.28 Anchorage of beam bars in interior joints– diagrams (a) and (b) show cross-sectional views in plan of joint region

9.15 विशेष सीमित सुदृढीकरण / Special Confining Reinforcement

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

- Special confining reinforcement shall be provided over a length '*l*o' from each joint face, towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces (as shown in Fig. 9.29). The length '*l*o' shall not be less than

  (a) larger lateral dimension of the member at the section where yielding occurs,

  (b) 1/6 of clear span of the member, and

  (c) 450 mm

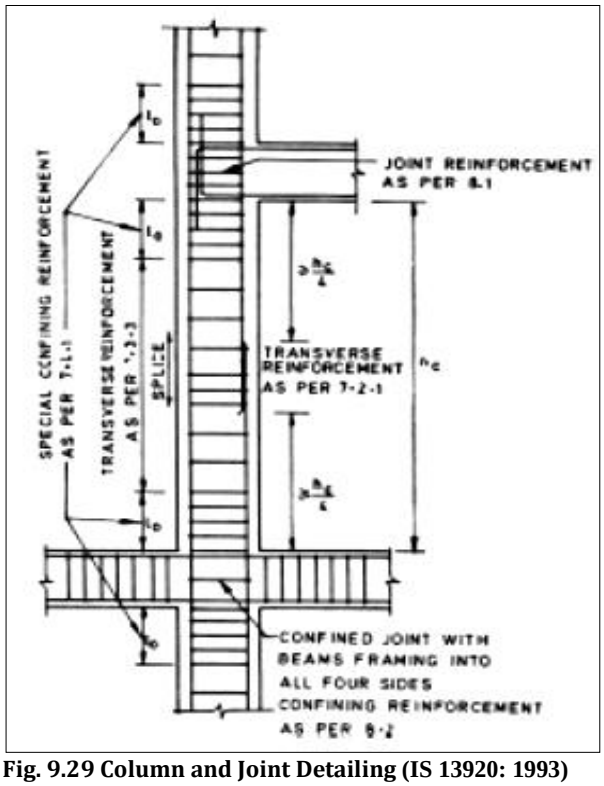


Fig. 9.29 Column and Joint Detailing (IS 13920: 1993)

- When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (as shown in Fig. 9.30).

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

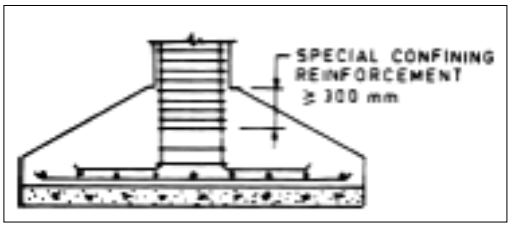


Fig. 9.30 Provision of Special confining reinforcement in Footings (IS 13920: 1993)

- Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (as shown in Fig. 9.31). This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

- Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result due to the presence of bracing, a mezzanine floor or a R.C.C. wall on either side of the column that extends only over a part of the column height (as shown in Fig. 9.31).

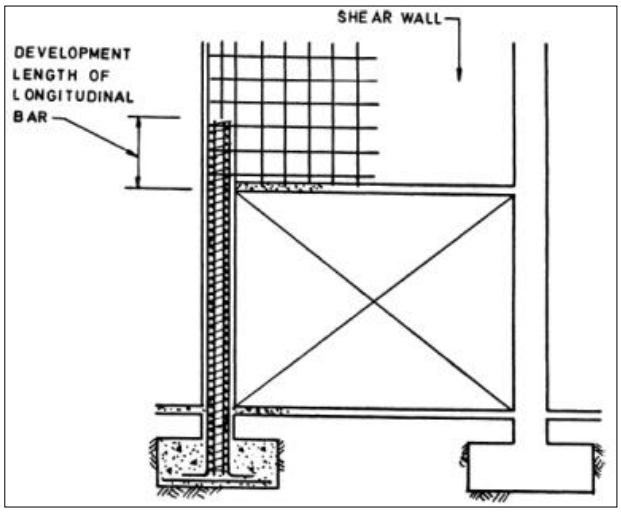


Fig. 9.31 Special Confining Reinforcement Requirement for Columns under Discontinued Walls (IS 13920: 1993)

9.16 विशेषतः भूकंपीय क्षेत्र में कतरनी दीवारों वाली इमारतों का निर्माण / Construction of Buildings with Shear Walls preferably in Seismic Regions

Reinforced concrete (RC) buildings often have *vertical plate-like* RC walls called *Shear Walls* in addition to slabs, beams and columns. These walls generally start at foundation level and are continuous throughout the building height. Their thickness can be as low as 150mm, or as high as 400mm in high rise buildings. Shear walls are usually provided along *both* length and width of buildings. Shear walls are like *vertically-oriented* wide *beams* that carry earthquake loads downwards to the foundation. (Fig. 9.32)

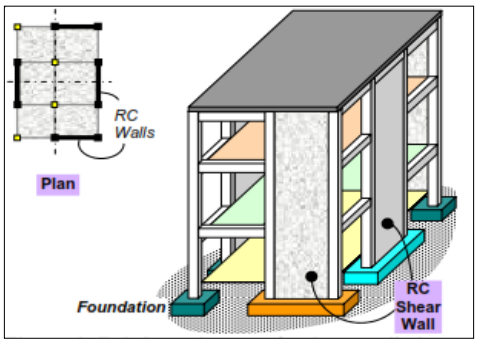


Fig. 9.32 Reinforced concrete shear walls in buildings – an excellent structural system for earthquake resistance

Properly designed and detailed buildings with shear walls have shown *very good* performance in past earthquakes. Shear walls in high seismic regions require special detailing. Shear walls are efficient, both in terms of construction cost and effectiveness in minimizing earthquake damage in structural and non-structural elements (like glass windows and building contents).

- Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces lateral sway of the building and thereby reduces damage to structure and its contents.
- Since shear walls carry *large* horizontal earthquake forces, the overturning effects on them are large. Thus, design of their foundations requires special attention.
- Shear walls should be provided along preferably *both* length and width. However, if they are provided along only one direction, a proper grid of beams and columns in the vertical plane (called a *moment-resistant frame*) must be provided along the other direction to resist strong earthquake effects.

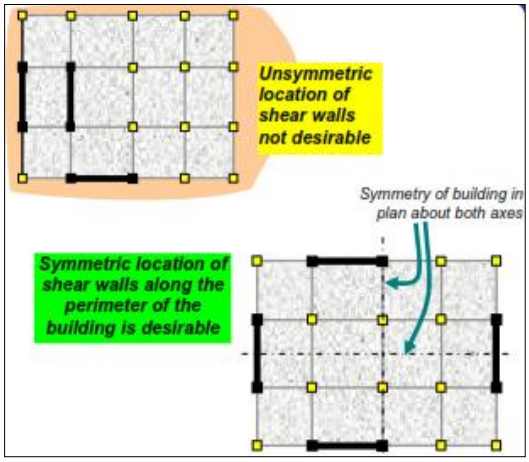


Fig. 9.33 Shear walls must be symmetric in plan layout – twist in buildings can be avoided

- Door or window openings can be provided in shear walls, but their size must be small to ensure least interruption to force flow through walls.
- Shear walls in buildings must be symmetrically located in plan to reduce ill-effects of twist in buildings. (Fig. 9.33)
- Shear walls are more effective when located along exterior perimeter of the building – such a layout increases resistance of the building to twisting.

9.16.1 तन्य डिजाइन और कतरनी दीवारों की ज्यामिति / Ductile Design and Geometry of Shear Walls

Shear walls are oblong in cross-section, *i.e.*, one dimension of the cross-section is much larger than the other. While rectangular cross-section is common, L- and U-shaped sections are also used. Overall geometric proportions of the wall, types and amount of reinforcement, and connection with remaining elements in the building help in improving the ductility of walls. The Indian Standard *Ductile Detailing Code* for RC members (IS:13920-1993) provides special design guidelines for ductile detailing of shear walls.

9.17 इम्प्रूव्ड़ डिजाइन रणनीतियां / Improved design strategies

9.17.1 हानिकारक भूकंप प्रभाव से भवनों का संरक्षण / Protection of Buildings from Damaging Earthquake Effects

Conventional seismic design attempts to make buildings that do not collapse under strong earthquake shaking, but may sustain damage to non-structural elements (like glass facades) and to some structural members in the building. There are two basic technologies –*Base Isolation*

Devices and *Seismic Dampers,* which are used to protect buildings from damaging earthquake effects.

9.17.2 आधार अलगाव / Base Isolation

The idea behind *base isolation* is to detach (*isolate*) the building from the ground in such a way that earthquake motions are not transmitted up through the building, or at least greatly reduced.

- As illustrated in Fig. 9.34, when the ground shakes, the rollers freely roll, but the building above does not move. Thus, no force is transferred to the building due to shaking of the ground; simply, *the building does not experience the earthquake*.

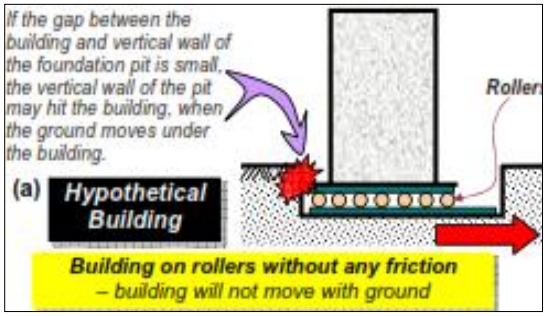


Fig. 9.34 Hypothetical Building

- As illustrated in Fig. 9.35, if the same building is rested on flexible pads that offer resistance against lateral movements, then *some* effect of the ground shaking will be transferred to the building above.

- As illustrated in Fig. 9.36, if the flexible pads are properly chosen, the forces induced by ground shaking can be a few times smaller than that experienced by the building built directly on ground, namely a *fixed base building*.

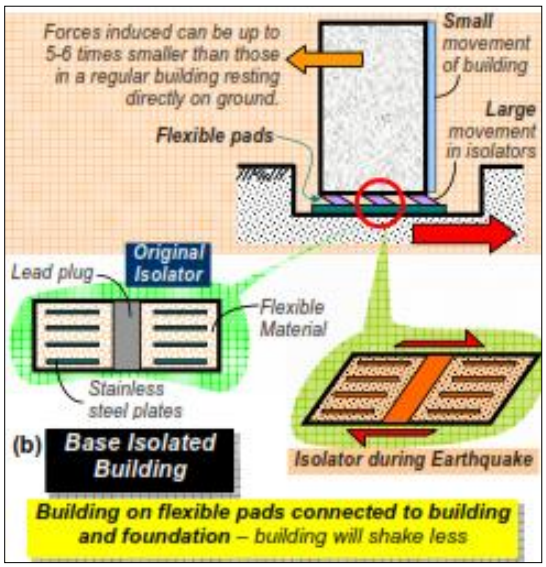


Fig. 9.35 Base Isolated Building

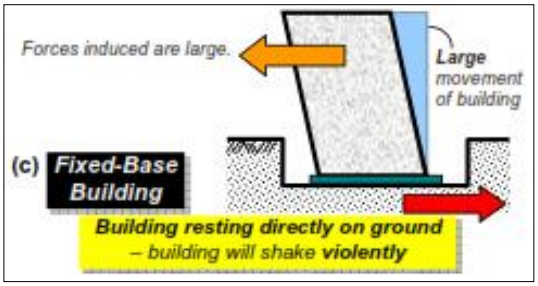


Fig. 9.36 Fixed-Base Building

9.17.3 भूकंपी स्पंज / Seismic Dampers

Seismic dampers are special devices introduced in the building to absorb the energy provided by the ground motion to the building. These dampers act like the hydraulic shock absorbers in cars – much of the sudden jerks are absorbed in the hydraulic fluids and only little is transmitted above to the chassis of the car.

When seismic energy is transmitted through them, dampers absorb part of it, and thus *damp* the motion of the building. Commonly used types of seismic dampers (Fig. 9.37) include

- *Viscous dampers* – Energy is absorbed by silicone-based fluid passing between piston-cylinder arrangement.

- *Friction dampers* – Energy is absorbed by surfaces with friction between them rubbing against each other.

- *Yielding dampers* – Energy is absorbed by metallic components that yield.

In India, friction dampers have been provided in an 18-storey RC frame structure in Gurgaon.

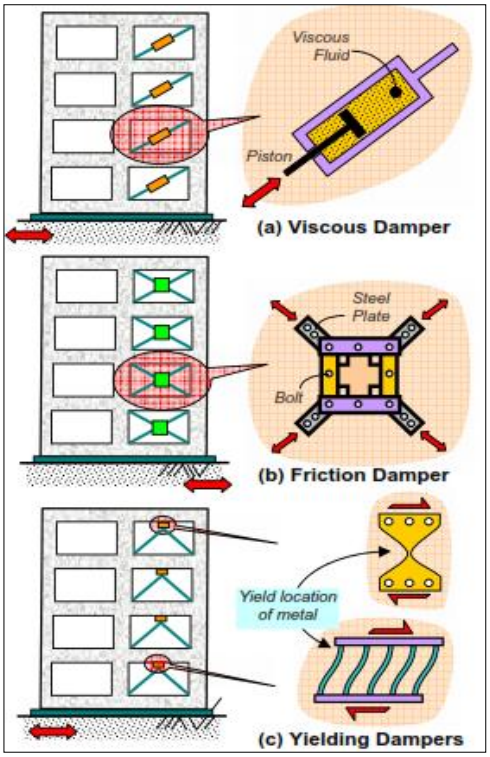
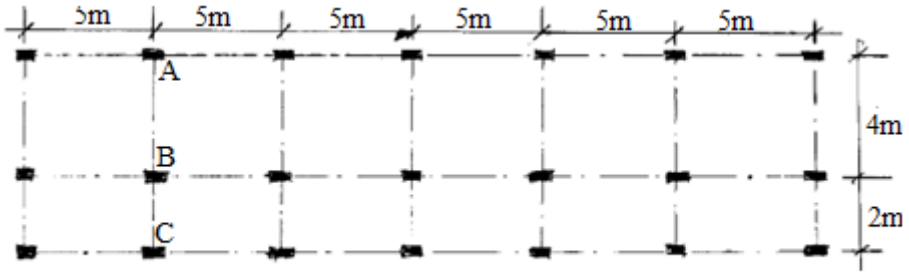


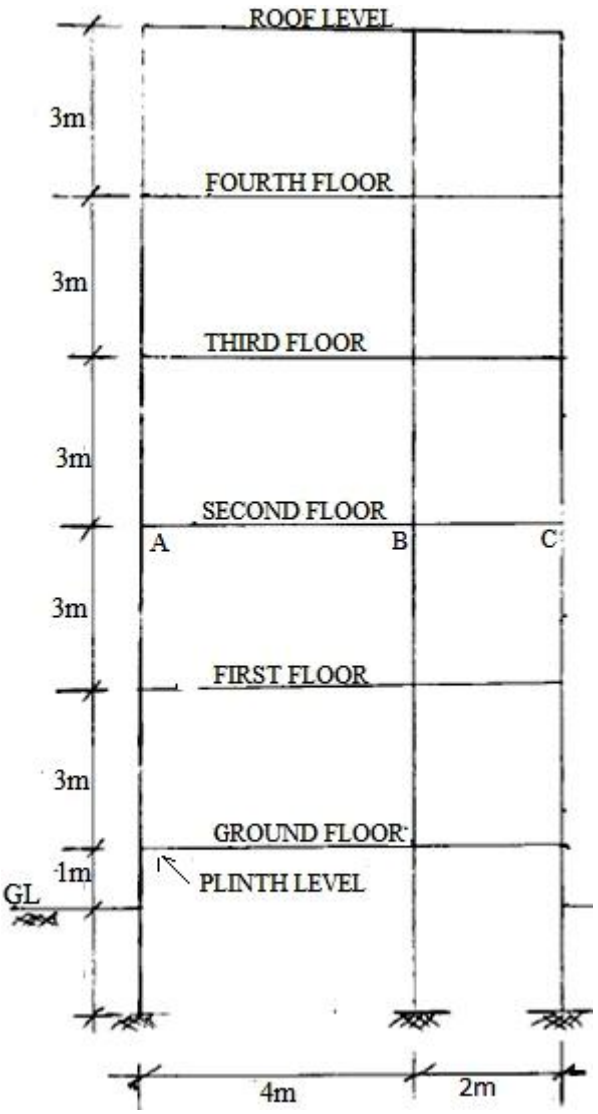
Fig. 9.37 Seismic Energy Dissipation Devices each device is suitable for a certain building

9.18 डिजाइन उदाहरण / Design Example – Beam Design of RC Frame with Ductile Detailing

Exercise – 2: Beam Design of RC Frame Building as per Provision of IS 13920 : 1993 and IS 1893 (Part 1) : 2002. Beam marked ABC is considered for Design.



PLAN



ELEVATION

Solution:

1. **General Data:** Grade of Concrete = M 25
   Grade of steel = Fe 415 Tor Steel

2. **Load Combinations:**

   As per Cl. 6.3 of IS 1893 (Part 1): 2002, following are load combinations for Earthquake Loading.

S. No.	Load Combination	DL	LL	EQ	Remark
1	1.5 DL + 1.5 LL	1.5	1.5	–	As per Table – 8 of IS 1893 (Part 1) : 2002
2	1.2 (DL + LL* + EQ_x)	1.5	0.25 or 0.50	+1.2	
3	1.2 (DL + LL* – EQ_x)	1.5	0.25 or 0.50	–1.2	
4	1.2 (DL + LL* + EQ_y)	1.5	0.25 or 0.50	+1.2	
5	1.2 (DL + LL* – EQ_y)	1.5	0.25 or 0.50	–1.2	
6	1.5 (DL + EQ_x)	1.5		+1.5	
7	1.5 (DL – EQ_x)	1.5		–1.5	
8	1.5 (DL + EQ_y)	1.5		+1.5	
9	1.5 (DL – EQ_y)	1.5		–1.5	
10	0.9 DL + 1.5 EQ_x	1.5		+1.5	
11	0.9 DL – 1.5 EQ_x	1.5		–1.5	
12	0.9 DL + 1.5 EQ_y	1.5		+1.5	
13	0.9 DL – 1.5 EQ_y	1.5		–1.5	

EQ_x implies E/Q Loading in X – direction & EQ_y implies E/Q Loading in Y – direction, where X & Y are orthogonal directions and Z is vertical direction. These load combinations are for E/Q Loading. In practice, Wind Load should also be considered in lieu of E/Q Load and critical of the two should be used in the design.

In this exercise, emphasis is to show calculations for ductile design & detailing of building elements subjected to Earthquake in the plan considered, Beams parallel to Y – direction are not significantly affected by Earthquake force in X – direction (except in case of highly unsymmetrical building) and vice versa. Beams parallel to Y – direction are designed for Earthquake loading in Y – direction only.

Torsion effect is not considered in this example.

3. Force Data:

For Beam AB, force resultants for various load cases (i.e. DL, LL & EQ Load) from Computer Analysis (or manually by any method of analysis) to illustrate the procedure of design are tabulated below.

Table – A Force resultants in beam AB for various load cases

Load Case	Left End		Centre		Right End	
	Shear (KN)	Moment (KN-m)	Shear (KN)	Moment (KN-m)	Shear (KN)	Moment (KN-m)
DL	– 51	– 37	4	32	59	– 56
LL	– 14	– 12	1	11	16	– 16
EQ_Y	79	209	79	11	79	– 119

Table – B Force resultants in beam AB for different load combinations

Load Combinations	Left End		Centre		Right End	
	Shear (KN)	**Moment (KN-m)**	**Shear (KN)**	**Moment (KN-m)**	**Shear (KN)**	**Moment (KN-m)**
1.5 DL + 1.5 LL	98	– 74	7	**64**	111	– 108
1.2 (DL + LL* + EQ_y)	31	205	101	52	172	– 303
1.2 (DL + LL* – EQ_y)	162	– 300	92	31	22	159
1.5 (DL + EQ_y)	44	261	**127**	61	**209**	**– 372**
1.5 (DL – EQ_y)	97	**– 371**	115	34	33	205
0.9 DL + 1.5 EQ_y	75	**283**	124	42	174	– 339
0.9 DL – 1.5 EQ_y	**167**	– 349	117	15	68	**238**

4. Various checks for Flexure Member:

(i) Check for Axial Stress:

As per Cl. 6.1.1 of IS 13920 : 1993, flexural axial stress on the member under E/Q loading shall not exceed 0.1 $f_{ck.}$

Factored Axial Force = 0.00 KN
Factored Axial Stress = 0.00 MPa < 0.10 $f_{ck.}$ OK

Hence, the member is to be designed as Flexure Member.

(ii) Check for Member size:

As per Cl. 6.1.3 of IS 13920 : 1993, width of the member shall not be less than 200 mm.

Width of the Beam, B = 250 mm > 200 mm OK
Depth of Beam, D = 550 mm

As per Cl. 6.1.2, member shall have a width to depth ratio of more than 0.3.

B/D = 250/550 = 0.4545 > 0.3 OK

As per Cl. 6.1.4, depth of member shall preferably be not more than 1/4 of the clear span i.e. (D/L) < 1/4 or (L/D) >4
Span = 4 m ; L/D = 4000/550 = 7.27 > 4 OK

Check for Limiting Longitudinal Reinforcement:

Nominal cover to meet Durability requirements as per Table – 16 of IS 456:2000 (Cl. 26.4.2) for Moderate Exposure = 30 mm .

Effective depth for Moderate Exposure conditions with 20 mm of bars in two layers = 550 – 30 – 20 – (20/2) = 490 mm

As per Cl. 6.2.1 (b) of IS 13920 : 1993, tension steel ratio on any face at any section shall not be less than $= (0.24 \sqrt{f_{ck}}) / f_y$
$= (0.24 \sqrt{25}) / 415 \quad = 0.289\ \% \approx 0.29\ \%$

Min. Reinforcement = (0.29/100) X 250 X 490 = 356 mm^2

Max. Reinforcement @ 2.5 % = (2.5/100) X 250 X 490 = 3063 mm^2

(iii) Design for Flexure:

Design for Hogging Moment at support A

At end A, from Table – B, M_u = 371 KN-m

Therefore, $M_u / bd^2 = 371 \times 10^6 / (250 \times 490 \times 490) = 6.18$

Referring to Table – 51 of SP – 16, for d'/d = 55/490 = 0.11,

We get As_t at top = 2.013, As_c = 0.866
Therefore, As_t at top = (2.013/100) x 250 x 490
= 2466 mm^2 > 356 mm^2 (Min. Reinforcement)
< 3063 mm^2 (Max. Reinforcement)

As_c at bottom = 0.866

As per Cl. 6.2.3 of IS 13290 : 1993, positive steel at a joint face must be at least equal to half the –ve steel at that face. Therefore, As_c at bottom must be at least 50 % of As_t, hence,

Revised As_c = 2.013/2 = 1.0065
As_c at bottom = (1.0065/100) x 250 x 490
= 1233 mm^2 > 426 mm^2 (Min. Reinforcement)
< 3063 mm^2 (Max. Reinforcement)

Design for Sagging Moment at support A

M_u = 283 KN-m

The beam will be designed as T-beam. The limiting capacity of the T-beam assuming $x_u < D_f$ and $x_u < x_{umax}$ may be calculated as follows:

$M_u = 0.87 f_y As_t d [1- (As_t f_y / b_f d f_{ck})]$ -------- (Eq. – 1)

Where, D_f = Depth of Flange
= 150 mm
x_u = Depth of Neutral Axis
x_{umax} = Limiting value of Neutral Axis
= 0.48 d
= 0.48 X 490
= 235.20 mm
b_w = 250 mm

b_f = Width of Flange
= $(L_0/6) + b_w + 6\ D_f$ or c/c of beam
= (0.7 X 4000/6) + 250 + 6 X 150
= 467 + 250 + 900 = 1617 mm or 4000 mm c/c
[Lower of two is to be adopted]

Substituting the values in Eq. – 1 and solving the quadratic equation,

$283 \times 10^6 = 0.87 \times 415\ As_t \times 490\ [1 - (As_t \times 415) / (1617 \times 490 \times 25)]$
$= 176914.5\ As_t\ [1 - 2.095 \times 10^{-5}\ As_t]$
$283 \times 10^6 = 176914.5\ As_t - 3.706\ As_t^2]$

$3.706\ As_t^2 - 176914.5\ As_t + 283 \times 10^6 = 0$

$As_t = [176914.5 \pm \sqrt{(176914.5^2 - 4 \times 3.706 \times 283 \times 10^6)}] / 2 \times 3.706$
$= [176914.5 \pm \sqrt{(3.129874 \times 10^{10} - 4195.192 \times 10^6)}] / 2 \times 3.706$
$= (176914.5 \pm 164631.55) / 7.412$

As_t at bottom = 1657.17 mm^2 > 356.00 mm^2
< 3063.00 mm^2 OK

It is necessary to check the design assumptions before finalizing the reinforcement.

$x_u = (0.87\ f_y\ As_t) / (0.36\ f_{ck}\ b_f)$
$= (0.87 \times 415 \times 1657) / (0.36 \times 25 \times 1617)$
= 41.10 mm < 150 mm OK
< d_f
< x_{umax} = 0.48 X 490 = 235 mm OK

% As_t = [1657/(250X490)] X 100 = 1.353 %

As per Cl. 6.2.4, "Steel provided at each of the top & bottom face of the member at any one section along its length shall be at least equal to $1/4^{th}$ of the maximum (-ve) moment steel provided at the face of either joint.

For Centre, M_u = 64 KN-m

$64 \times 10^6 = 0.87 \times 415\ As_t \times 490\ [1 - (As_t \times 415) / (1617 \times 490 \times 25)]$
$= 176914.5\ As_t\ [1 - 2.095 \times 10^{-5}\ As_t]$
$64 \times 10^6 = 176914.5\ As_t - 3.706\ As_t^2]$

$3.706\ As_t^2 - 176914.5\ As_t + 64 \times 10^6 = 0$

$As_t = [176914.5 \pm \sqrt{(176914.5^2 - 4 \times 3.706 \times 64 \times 10^6)}] / 2 \times 3.706$
= 365 mm^2

For Right Support, M_u = 238 KN-m

$238 \times 10^6 = 0.87 \times 415\ As_t \times 490\ [1 - (As_t \times 415) / (1617 \times 490 \times 25)]$
$= 176914.5\ As_t\ [1 - 2.095 \times 10^{-5}\ As_t]$
$238 \times 10^6 = 176914.5\ As_t - 3.706\ As_t^2]$

$3.706\ As_t^2 - 176914.5\ As_t + 238 \times 10^6 = 0$

$As_t = [176914.5 \pm \sqrt{(176914.5^2 - 4 \times 3.706 \times 238 \times 10^6)}] / 2 \times 3.706$
= 1386 mm^2

(iv) Reinforcement Requirement:

Top reinforcement is larger of As_t at top for hogging moment or As_c at top for sagging moment i.e. 2466 mm^2 or 968 mm^2. Hence, provide 2466 mm^2 at top.

Bottom reinforcement is larger of As_c at bottom for hogging moment or As_t at bottom for sagging moment i.e. 1233 mm^2 or 1936 mm^2. Hence, provide 1936 mm^2 at bottom.

Details of Reinforcement

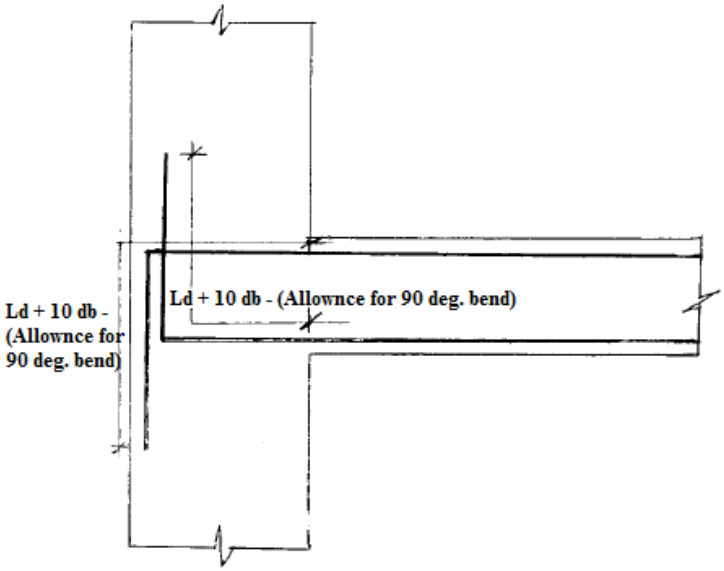
Top Reinforcement			
Beam AB	**Left End**	**Centre**	**Right End**
Hogging Moment	– 371	-	– 371
M_u / bd^2	6.18	-	6.18
As_t at top	2.013%	-	2.013
As_c at bottom	0.866 % < 2.013 / 2 = 1.0065%. Hence, revised As_c = 1.0065%	-	0.866 Revised to 1.0065% as per Cl. 6.2.3 of IS 13920:1993

Bottom Reinforcement			
Sagging Moment	283	64	238
As_t at bottom	As_t req. = 1657 mm^2 = 1.353% > 2.013/2 = 1.0065% OK Provide As_t at bottom = 1.353%	As_t req. = 365 mm^2 = 0.298 > 0.29% > 2.013/4 = 0.504% OK As per Cl. 6.2.4 of IS 13920:1993 Provide As_t at bottom = 0.504%	As_t req. = 1386 mm^2 = 1.17% > 0.29% > 2.013/2 = 1.0065% Provide As_t at bottom = 1.17%
As_c at top	As_c req. = 1657 / 2 = 829 mm^2 = 0.677% > 0.29% > 2.013 / 4 = 0.504% OK	As_c req. = 0.504/2 = 0.252% > 0.29% Provide Min.As_c = 0.29%	As_c req. = 1657 / 2 = 829 mm^2 = 0.677% > 0.29% > 2.013 / 4 = 0.504% OK

Summary of Reinforcement required			
Beam	Left End	Centre	Right End
	Top = 2.013% = 2466 mm^2 Bottom = 1.353% = 1658 mm^2	Top = 0.504% = 618 mm^2 Bottom = 0.504% = 618 mm^2	Top = 2.013% = 2466 mm^2 Bottom = 1.0065% = 1233 mm^2

Reinforcement provided			
	2 – 20Φ cont. + 4 – 25Φ extra As_t = 2592 mm^2 (2.116%) 2 – 20Φ cont. + 2 – 20Φ extra + 2 – 16 Φ As_t = 1658 mm^2 (1.353%)	2 – 20Φ cont. As_t = 628 mm^2 (0.512%) 2 – 20Φ cont. As_t = 628 mm^2 (0.512%)	2 – 20Φ cont. + 4 – 25Φ extra Top = 2592 mm^2 2 – 20Φ cont. + 2 – 20Φ extra + 2 – 16Φ As_t = 1658 mm^2 (1.353%)

Details of Reinforcement



L_d = Development Length in tension
d_b = Dia. of bar

For Fe 415 steel and M25 grade concrete as per Table – 65 of SP – 16

For 25Φ bars, 1007 + 10Φ - 8Φ = 1007+50 = 1057 mm
For 20Φ bars, 806 + 2Φ = 806+40 = 846 mm

(v) Design for Shear

Tensile steel provided at Left End = 2.116%

Permissible Design Stress of Concrete (As per Table – 19 of IS 456:2000) } τ_c = 0.835 MPa

Design Shear Strength of Concrete = τ_c b d
= (0.835 X 250 X 490) / 1000
= 102 KN

Similarly, Design Shear Strength of Concrete at centre for As_t = 0.512%

τ_c = 0.493 MPa

Shear Strength of Concrete at centre = τ_c b d
= (0.493 X 250 X 490) / 1000
= 60.40 KN

(vi) Shear force due to Plastic Hinge Formation at the ends of the beam

The additional shear due to formation of plastic hinges at both ends of the beams is evaluated as per Cl. 6.3.3 of IS 13920:1993 is given by

$V_{sway\ to\ right} = \pm 1.4\ [M_{ulim}^{As} + M_{ulim}^{Bh}] / L$
$V_{sway\ to\ left} = \pm 1.4\ [M_{ulim}^{Ah} + M_{ulim}^{Bs}] / L$

Where,
M_{ulim}^{As} = Sagging Ultimate Moment of Resistance of Beam Section at End A
M_{ulim}^{Ah} = Hogging Ultimate Moment of Resistance of Beam Section at End A
M_{ulim}^{Bh} = Sagging Ultimate Moment of Resistance of Beam Section at End B
M_{ulim}^{Bs} = Hogging Ultimate Moment of Resistance of Beam Section at End B

At Ends, beam is provided with steel – p_t = 2.116%, p_c = 1.058%

Referring Table 51 of SP – 16 for p_t = 2.116%, p_c = 1.058%

The lowest value of M_u^{Ah} / bd^2 is found.
$M_u^{Ah} / bd^2 = 6.45$

Hogging Moment Capacity at End A
$M_u^{Ah} = 6.45 \times 250 \times 490^2$
$= 3.8716 \times 10^8$ N-mm
= 387.16 KN-m

Similarly, for M_u^{As}, p_t = 1.058%, p_c = 2.116%

Contribution of Compressive steel is ignored while calculating the Sagging Moment Capacity at T-beam.
$M_u^{As} = 0.87 f_y As_t d [1- (As_t f_y / b_f d f_{ck})]$
= 0.87 X 415 X 1658 X 490 [1– (1658 X 415 / 1617 X 490 X 25)]
= 283.13 KN-m

Similarly, for Right End of beam

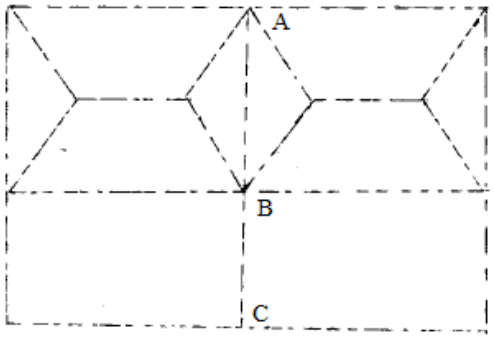
M_u^{Bh} = 387.16 KN-m & M_u^{Bs} = 283.13 KN-m

Shear due to Plastic Hinge is calculated as

$V_{sway\ to\ right} = \pm 1.4\ [M_u^{As} + M_u^{Bh}] / L$
$= \pm 1.4\ [283.13 + 387.16] / 4$
$= 234.60$ KN

$V_{sway\ to\ left} = \pm 1.4\ [M_u^{Ah} + M_u^{Bs}] / L$
$= \pm 1.4\ [387.16 + 283.13] / 4$
$= 234.60$ KN

Design Shear



Dead Load of Slab = 5.0 KN/m^2, Live Load = 4.0 KN/m^2

Load due to Slab in Beam AB	= 2 X [1/2 X 4 X 2] X 5	= 40 KN	(10 KN/m)
Self Wt. Of Beam	= 0.25 X 0.55 X 25 X 4	= 13.75 KN ≈ 14.00 KN	(3.44 KN/m)
Live Load	= 2 X [1/2 X 4 X 2] X 4	= 32 KN	(8 KN/m)
Shear Force due to DL	= 1/2 X [40 + 14]	= 27 KN	
Shear Force due to LL	= 1/2 X [32]	= 16 KN	

As per Cl. 6.3.3 of IS 13920:1993, the Design shear at End A i.e. V_{ua}, and Design Shear at End B i.e. V_{ub} are computed as

(i) For Sway Right

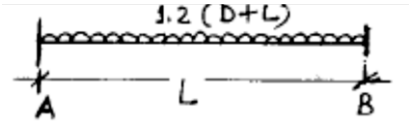
$V_{ua} = V_a^{D+L} - 1.4\ [M_{ulim}^{As} + M_{ulim}^{Bh}] / L_{AB}$
$V_{ub} = V_b^{D+L} + 1.4\ [M_{ulim}^{As} + M_{ulim}^{Bh}] / L_{AB}$

(ii) For Sway Left

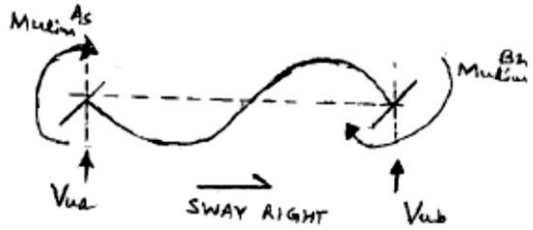
$V_{ua} = V_a^{D+L} + 1.4\ [M_{ulim}^{Ah} + M_{ulim}^{Bs}] / L_{AB}$
$V_{ub} = V_b^{D+L} - 1.4\ [M_{ulim}^{Ah} + M_{ulim}^{Bs}] / L_{AB}$

Where,

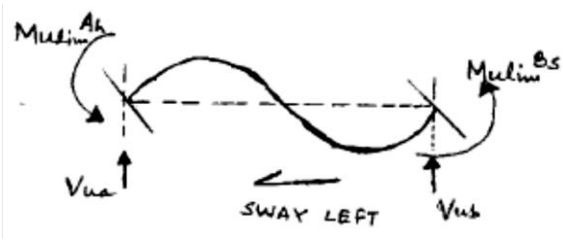
V_a^{D+L} & V_b^{D+L} = Shear at ends A & B respectively due to vertical load with Partial Safety Factor of 1.2 on Loads



$V_a^{D+L} = V_b^{D+L} = 1.2\ (D+L) / 2$



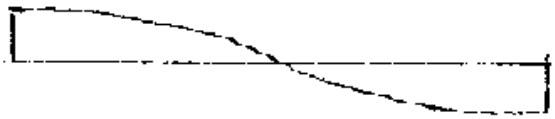
----------For equ. (i)



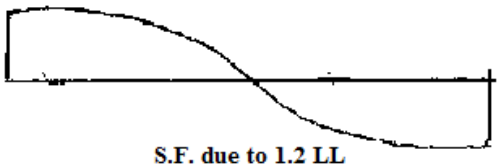
----------For equ. (ii)

$1.4 \times [M_u^{As} + M_u^{Bh}] / L = 234.60$ KN

$1.4 \times [M_u^{Ah} + M_u^{Bs}] / L = 234.60$ KN



S.F. due to 1.2 DL

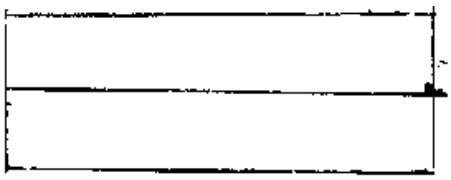


S.F. due to 1.2 LL

$V_a^D = V_b^D = 1.2 \times 27 = 32.4$

$V_a^L = V_b^L = 1.2 \times 16 = 19.2$

$\Big\} = 51.6$



Shear due to Sway Right



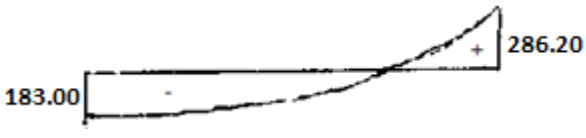
Shear due to Sway Left

V_{ua} = [1.2 (27+16)] – 234.60 = 51.6 – 234.60 = (-) 183.00 KN

V_{ub} = [1.2 (27+16)] + 234.60 = 51.6 + 234.60 = (+) 286.20 KN

V_{ub} = [1.2 (27+16)] + 234.60 = 51.6 + 234.60 = (+) 286.20 KN

V_{ua} = [1.2 (27+16)] – 234.60 = 51.6 – 234.60 = (-) 183.00 KN





As per Cl. 6.3.3 of IS 13920:1993, the Design Shear Force to be resisted shall be of maximum of

(i) Calculate factored S.F. as per analysis (Refer Table – B)
(ii) Shear Force due to formation of Plastic Hinges at both ends of the beam plus factored gravity load on the span.

Hence, Design shear Force V_u will be 286.20 KN (corresponding to formation of Plastic Hinge).

From Analysis as per Table – B, S.F. at mid-span of the beam is 127 KN. However, Shear due to formation of Plastic Hinge is 234.60 KN. Hence, design shear at centre of span is taken as 234.60 KN.

The required capacity of shear reinforcement at ends,

$V_{us} = V_u - V_c$
$= 286.20 – 102$
$= 184.20$ KN

And at centre, $V_{us} = 234.60 – 60.40$
$= 174.20$ KN

At supports,

$V_{us} / d = 286.20 / 49 = 5.84$ KN/cm

Therefore, requirement of stirrups is

12Φ – 2 legged strippus @ 135 c/c [$V_{us} / d = 6.06$]

However, provide 12Φ – 2 legged strippus @ 120 c/c as per provision of Cl. 6.3.5 of IS 13920:1993. [$V_{us} / d = 6.806$]

At centre,

$V_{us} / d = 234.60 / 49 = 4.78$ KN/cm
Provide 12Φ – 2 legged strippus @ 170 c/c [$V_{us} / d = 4.804$]

As per Cl. 6.3.5 of IS 13920:1993, the spacing of stirrups in the mid-span should not exceed $d/2 = 490/2 = 245$ mm

Minimum Shear Reinforcement as per Cl. 26.5.1.6 of IS 456:2000 is

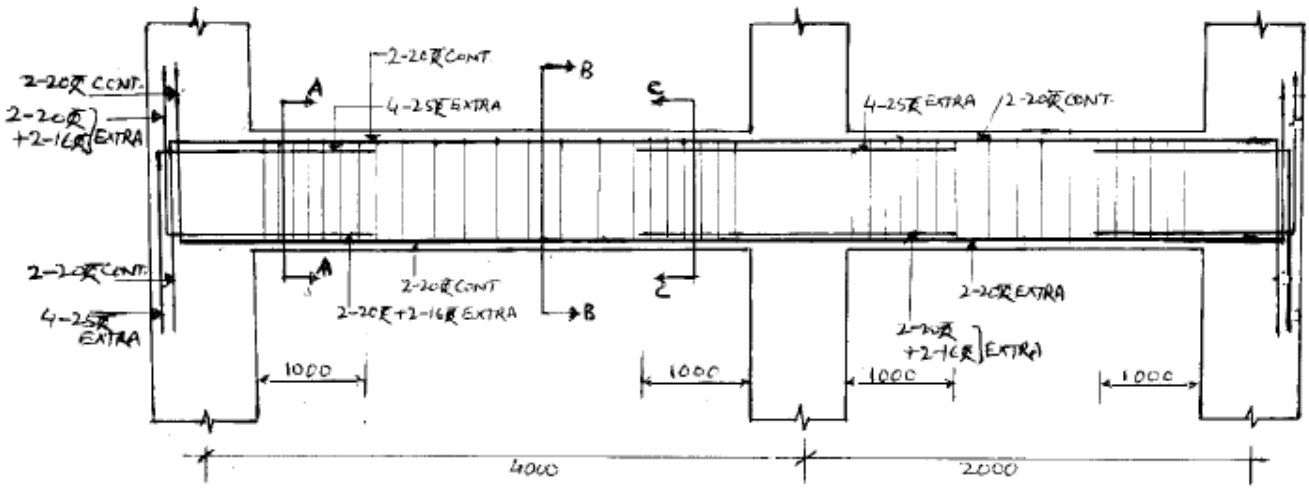
$S_v = A_{sv} X 0.8 f_y / 0.46$
$= (2 X 79 X 0.87 X 415) / (250 X 0.4)$
$= 570$ mm

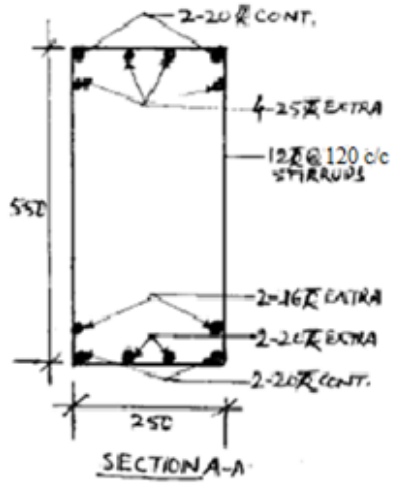
As per CL. 6.3.5 of IS 13920:1993, “Spacing of Links over a length of 2d at either end of beam shall not exceed

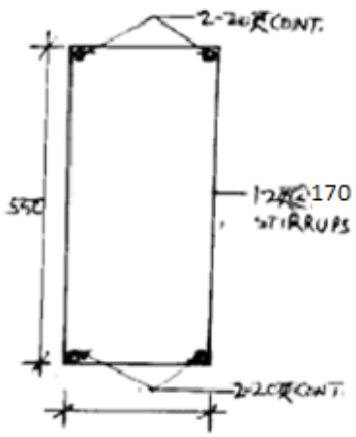
(i) $d/4 = 490/4 = 122.50$ mm
(ii) 8 times dia. of smallest longitudinal bar $= 8 X 16 = 128$ mm

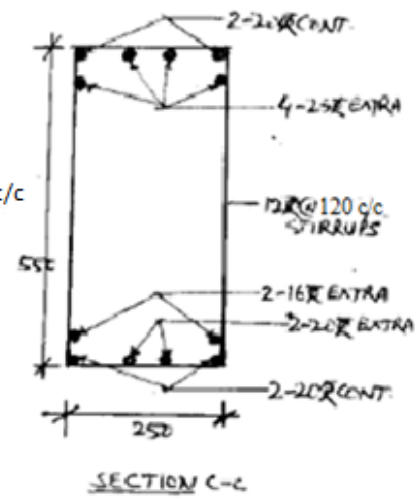
However, it need not be less than 100 mm.

The reinforcement detailing is shown as below:









***

अध्याय / Chapter – 10

अल्प सामर्थ्य चिनाई संरचनाओं के निर्माण / Construction of Low Strength Masonry Structures

Two types of construction are included herein, namely

a) Brick construction using weak mortar, and
b) Random rubble and half-dressed stone masonry construction using different mortars such as clay mud lime-sand and cement sand.

10.1 भूकंप के दौरान ईंट चिनाई की दीवारों का व्यवहार / Behaviour of Brick Masonry Walls during Earthquakes

Of the three components of a masonry building (*roof*, *wall* and *foundation* as illustrated in Fig.10.1), the walls are most vulnerable to damage caused by horizontal forces due to earthquake.

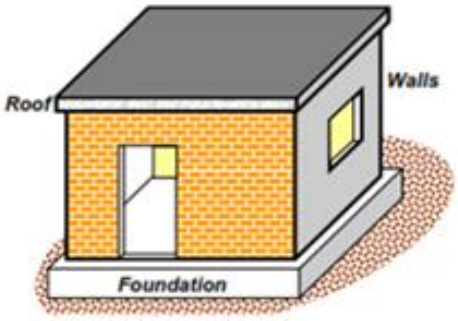


Fig. 10.1 Basic components of Masonry Building

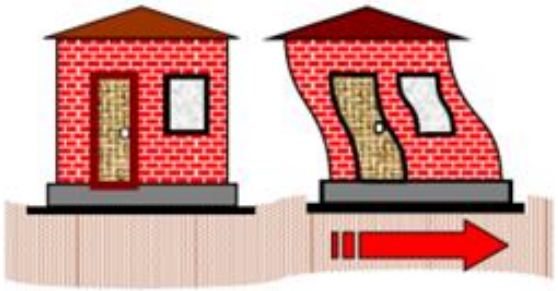


Fig. 10.2 Effect of Inertia in a building when shaken at its base

Ground vibrations during earthquakes cause inertia forces at locations of mass in the building (Fig. 10.2). These forces travel through the roof and walls to the foundation. The main emphasis is on ensuring that these forces reach the ground without causing major damage or collapse.

A wall topples down easily if pushed horizontally at the top in a direction perpendicular to its plane (termed *weak*

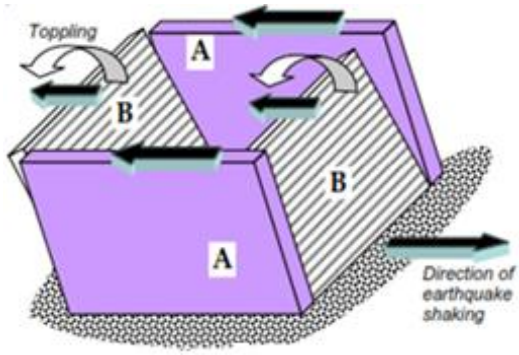


Fig. 10.3 For the direction of Earthquake shaking shown. wall B tends to fail

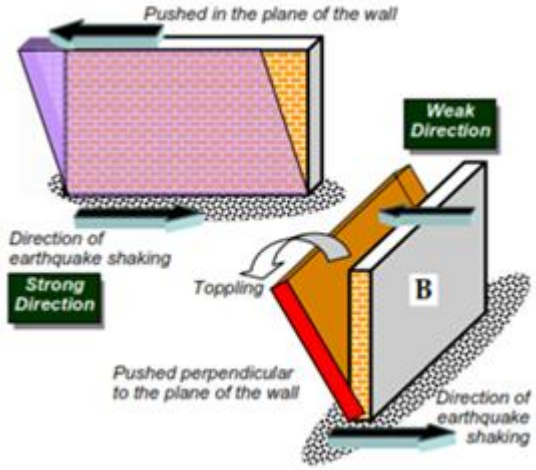


Fig. 10.4 Direction of force on a wall critically determines its earthquake performance

direction), but offers much greater resistance if pushed along its length (termed *strong direction*) (Fig. 10.3 & 10.4).

The ground shakes simultaneously in the vertical and two horizontal directions during earthquakes. However, the horizontal vibrations are the most damaging to normal masonry buildings. Horizontal inertia force developed at the roof transfers to the walls acting either in the weak or in the strong direction. If all the walls are not tied together like a box, the walls loaded in their weak direction tend to topple.

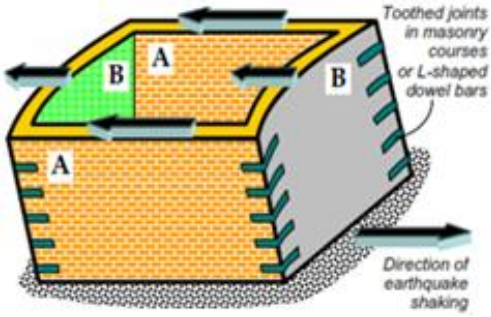


Fig. 10.5 Wall B properly connected to Wall A (Note: roof is not shown): Walls A (loaded in strong direction) support Walls B (loaded in weak direction)

To ensure good seismic performance, all walls must be joined properly to the adjacent walls. In this way, walls loaded in their weak direction can *take advantage* of the good lateral resistance offered by walls loaded in their strong direction (Fig. 10.5). Further, walls also need to be tied to the roof and foundation to preserve their overall integrity.

10.2 चिनाई वाली इमारतों में बॉक्स एक्शन कैसे सुनिश्चित करें / How to ensure Box Action in Masonry Buildings

A simple way of making these walls behave well during earthquake shaking is by making them act together as a box along with the roof at the top and with the foundation at the bottom. A number of construction aspects are required to ensure this box action.

- Firstly, connections between the walls should be good. This can be achieved by (a) ensuring good interlocking of the masonry courses at the junctions, and (b) employing horizontal bands at various levels, particularly at the lintel level.

- Secondly, the sizes of door and window openings need to be kept small. The smaller the opening, the larger is the resistance offered by the wall.

- Thirdly, the tendency of a wall to topple when pushed in the weak direction can be reduced by limiting its length-to-thickness and height to-thickness ratios. Design codes specify limits for these ratios. A wall that is too tall or too long in comparison to its thickness is particularly vulnerable to shaking in its weak direction (Fig. 10.6).

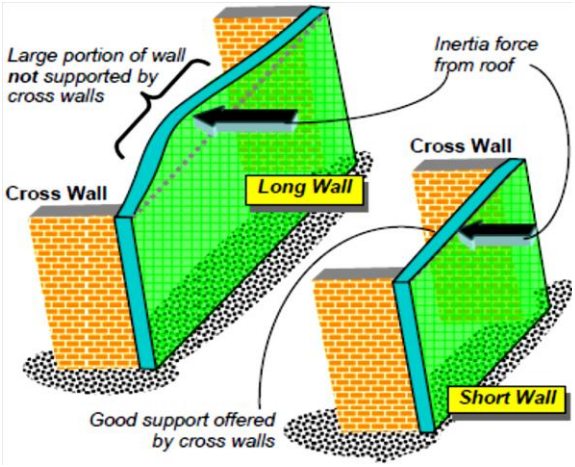


Fig. 10.6 Slender walls are vulnerable

Brick masonry buildings have large mass and hence attract large horizontal forces during earthquake shaking. They develop numerous cracks under both compressive and tensile forces caused by earthquake shaking. The focus of *earthquake resistant* masonry building construction is to ensure that these effects are sustained without major damage or collapse. Appropriate choice of structural configuration can help achieve this.

The structural configuration of masonry buildings includes aspects like (a) overall shape and size of the building, and (b) distribution of mass and (horizontal) lateral load resisting elements across the building.

Large, tall, long and un-symmetric buildings perform poorly during earthquakes. A strategy used in making them earthquake resistant is developing good *box action* between all the elements of the building, *i.e.*, between roof, walls and foundation (Fig. 10.7). For example, a horizontal band introduced at the lintel level ties the walls together and helps to make them behave as a single unit.

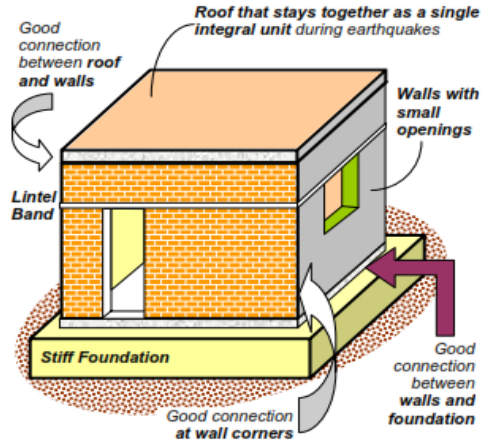


Fig. 10.7 Essential requirements to ensure box action in a masonry building

10.3 क्षैतिज बैंड की भूमिका / Role of Horizontal Bands

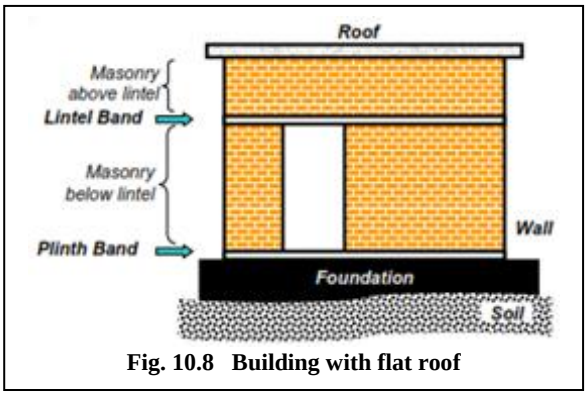


Fig. 10.8 Building with flat roof

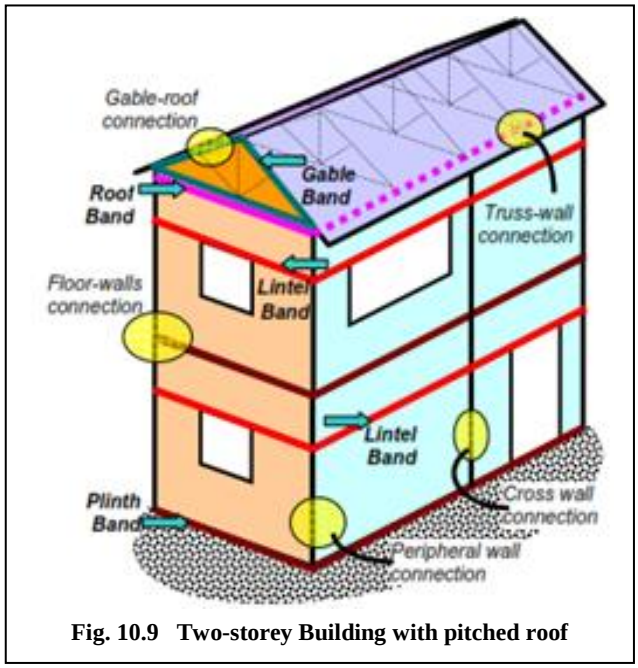


Fig. 10.9 Two-storey Building with pitched roof

Horizontal bands are the most important earthquake-resistant feature in masonry buildings. The bands are provided to hold a masonry building as a single unit by tying all the walls together, and are similar to a closed belt provided around cardboard boxes (Fig. 10.8 & 10.9).

- The lintel band undergoes bending and pulling actions during earthquake shaking (Fig.10.10).
- To resist these actions, the construction of lintel band requires special attention.

- Bands can be made of wood (including bamboo splits) or of reinforced concrete (RC); the RC bands are the best (Fig. 10.11).
- The straight lengths of the band must be properly connected at the wall corners.
- In wooden bands, proper nailing of straight lengths with spacers is important.
- In RC bands, adequate anchoring of steel links with steel bars is necessary.
- The lintel band is the most important of all, and needs to be provided in almost all buildings.

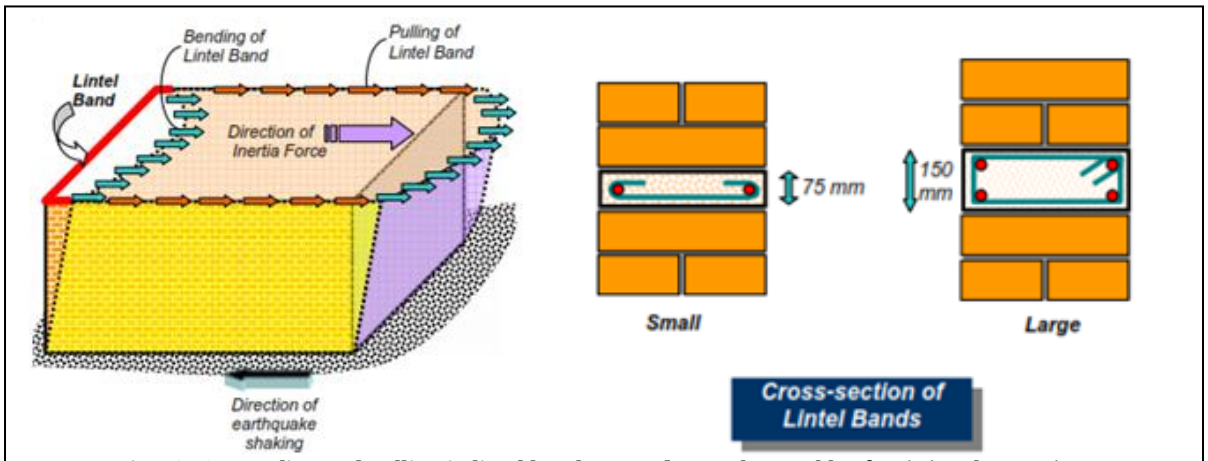


Fig. 10.10 Bending and pulling in lintel bands – *Bands must be capable of resisting these actions*

- The gable band is employed only in buildings with pitched or sloped roofs.
- In buildings with flat *reinforced concrete* or *reinforced brick* roofs, the roof band is not required, because the roof slab also plays the role of a band. However, in buildings with flat timber or CGI sheet roof, roof band needs to be provided. In buildings with pitched or sloped roof, the roof band is very important.
- Plinth bands are primarily used when there is concern about uneven settlement of foundation soil.

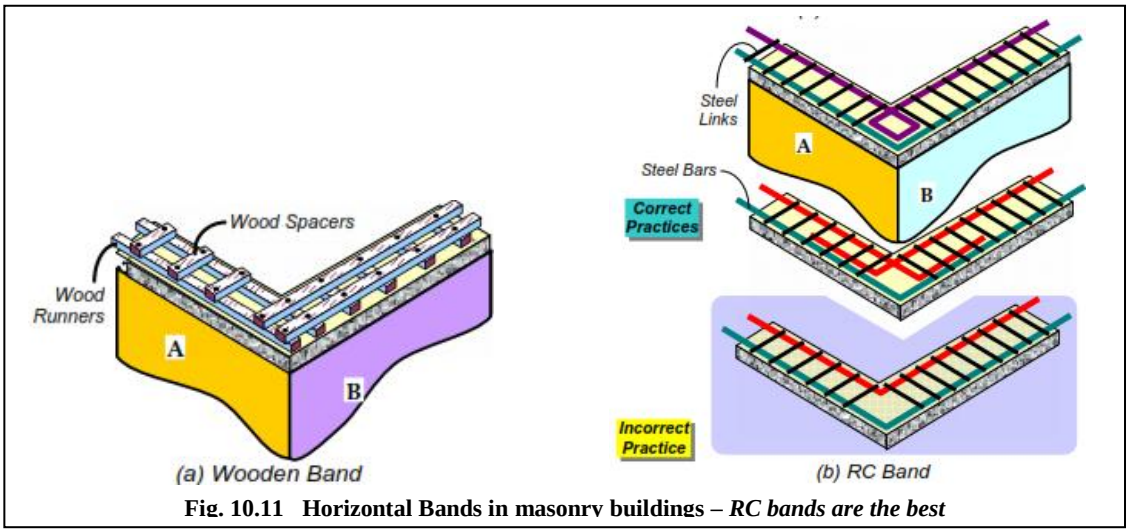


Fig. 10.11 Horizontal Bands in masonry buildings – *RC bands are the best*

Lintel band: Lintel band is a band provided at lintel level on all load bearing internal, external longitudinal and cross walls.

Roof band: Roof band is a band provided immediately below the roof or floors. Such a band need not be provided underneath reinforced concrete or brick-work slabs resting on bearing

walls, provided that the slabs are continuous over the intermediate wall up to the crumple sections, if any, and cover the width of end walls, fully or at least 3/4 of the wall thickness.

Gable band: Gable band is a band provided at the top of gable masonry below the purlins. This band shall be made continuous with the roof band at the eaves level.

Plinth band: Plinth band is a band provided at plinth level of walls on top of the foundation wall. This is to be provided where strip footings of masonry (other than reinforced concrete or reinforced masonry) are used and the soil is either soft or uneven in its properties, as frequently happens in hill tracts. This band will serve as damp proof course as well.

10.4 अधोलंब सुदृढीकरण / 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 10.1. For walls thicker than 340 mm the area of the bars shall be proportionately increased.

No vertical steel need be provided in category A building. The vertical reinforcement shall be properly embedded in the plinth masonry of foundations and roof slab or roof band so as to develop its tensile strength in bond. It shall be passing through the lintel bands and floor slabs or floor level bands in all storeys.

Table – 10.1 Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units (IS 4326: 1993)

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	Nil	Nil	10	12
	Bottom	Nil	Nil	12	16
Three	Top	Nil	10	10	12
	Middle	Nil	10	12	16
	Bottom	Nil	12	12	16
Four	Top	10	10	10	Four storeyed building not permitted
	Third	10	10	12	
	Second	10	12	16	
	Bottom	12	12	20	

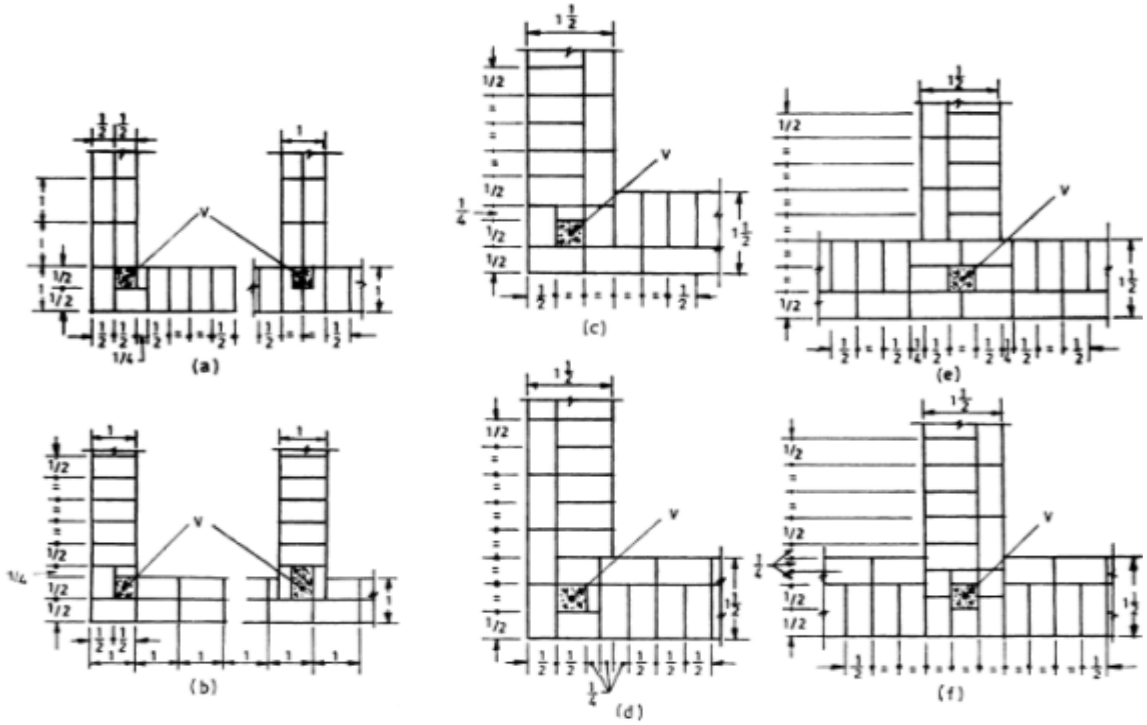
NOTES

1 **The diameters given above are for H.S.D. bars. For mild-steel plain bars, use equivalent diameters as given under Table – 10.6 Note 2.**

2 **The vertical bars will be covered with concrete M15 or mortar 1: 3 grade in suitably created pockets around the bars. This will ensure their safety from corrosion and good bond with masonry.**

3 **In case of floors/roofs with small precast components, also refer 9.2.3 of IS 4326: 1993 for floor/roof band details.**

- Bars in different storeys may be welded (IS 2751: 1979 and IS 9417: 1989, as relevant) or suitably lapped.
- Vertical reinforcement at jambs of window and door openings shall be provided as per Table – 10.1. It may start from foundation of floor and terminate in lintel band (Fig. 10.17).
- Typical details of providing vertical steel in brickwork masonry with rectangular solid units at corners and T-junctions are shown in Fig. 10.12.



1 - One brick length, ½ - half brick length, V - Vertical steel bar with mortar/ concrete filling in pocket
a) and b) - Alternate courses in obe brick wall
c) and d) - Alternate courses at corner junction of 1½ - brick wall
e) and f) - Alternate courses at T- junction of 1½ - brick wall

Fig. 10.12 Typical Details of Providing Vertical Steel Bars in Brick Masonry (IS 4326: 1993)

10.5 दीवारों में सूराखों का संरक्षण / Protection of Openings in Walls

Horizontal bands including *plinth band*, *lintel band* and *roof band* are provided in masonry buildings to improve their earthquake performance. Even if horizontal bands are provided, masonry buildings are weakened by the openings in their walls.

Embedding vertical reinforcement bars in the edges of the wall piers and anchoring them in the foundation at the bottom and in the roof band at the top, forces the slender masonry piers to undergo *bending* instead of *rocking*. In wider wall piers, the vertical bars enhance their capability to resist horizontal earthquake forces and delay the X-cracking. Adequate cross-sectional area of these vertical bars prevents the bar from yielding in tension. Further, the vertical bars also help protect the wall from sliding as well as from collapsing in the weak direction.

However, the most common damage, observed after an earthquake, is diagonal X-cracking of wall piers, and also inclined cracks at the corners of door and window openings.

When a wall with an opening deforms during earthquake shaking, the shape of the opening distorts and becomes more like a *rhombus* - two opposite corners move away and the other two come closer. Under this type of deformation, the corners that come closer develop cracks. The cracks are bigger when the opening sizes are larger. Steel bars provided in the wall masonry all

around the openings restrict these cracks at the corners. In summary, lintel and sill bands above and below openings, and vertical reinforcement adjacent to vertical edges, provide protection against this type of damage (Fig. 10.13).

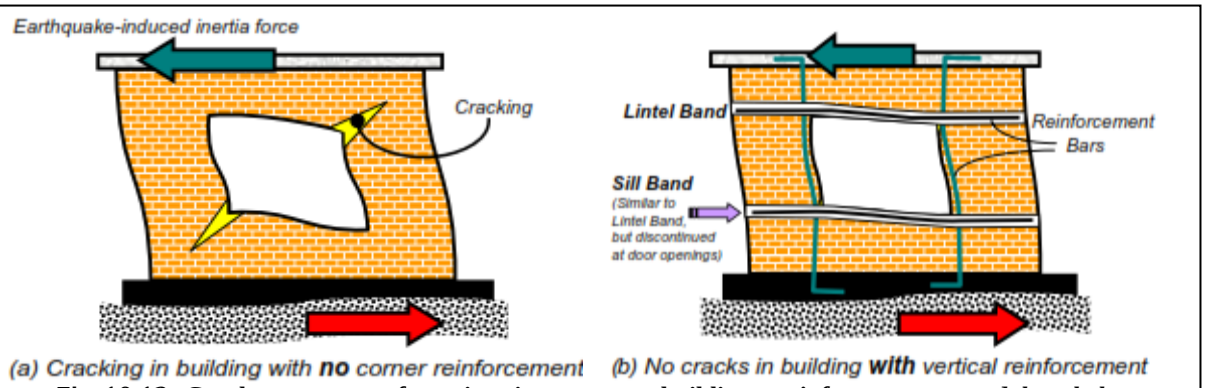


Fig. 10.13 Cracks at corners of openings in a masonry building – *reinforcement around them helps*

10.6 भूकंप प्रतिरोधी ईंट चिनाई भवन के निर्माण हेतु सामान्य सिद्धांत / General Principles for Construction of Earthquake Resistant Brick Masonry Building

- Low Strength Masonry constructions should not be permitted for important buildings.
- It will be useful to provide damp-proof course at plinth level to stop the rise of pore water into the superstructure.
- Precautions should be taken to keep the rain water away from soaking into the wall 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.
- 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.
- Ignoring tensile strength, free standing walls should be checked against overturning under the action of design seismic coefficient, ah, allowing for a factor of safety of 1.5.

10.6.1 भवनों की श्रेणियाँ / Categories of Buildings

For the purpose of specifying the earthquake resistant features in masonry and wooden buildings, the buildings have been categorized in five categories A to E based on the seismic zone and the importance of building *I*,

Where,

I = importance factor applicable to the building [Ref. Clause 6.4.2 and Table - 6 of IS 1893 (Part 1): 2002].

The building categories are given in Table – 10.2.

Table -10.2 Building Categories for Earthquake Resisting Features (IS 4326: 1993)

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.				

10.6.2 कमजोर गारे में ईंट चिनाई कार्य / Brickwork in Weak Mortars

- The fired bricks should have a compressive strength not less than 3.5 MPa. Strength of bricks and wall thickness should he selected for the total building height.

- The mortar should be lime-sand (1:3) or clay mud of good quality. Where horizontal steel is used between courses, cement-sand mortar (1:3) should be used with thickness so as to cover the steel with 6 mm mortar above and below it. Where vertical steel is used, the surrounding brickwork of 1 X 1 or l½ X 1½ brick size depending on wall thickness should preferably be built using 1:6 cement-sand mortar.

- The minimum wall thickness shall be one brick in one storey construction and one brick in top storey and 1½brick in bottom storeys of up to three storey constructions. It should also not be less than l/16 of the length of wall between two consecutive perpendicular walls.

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

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

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

10.6.3 आयताकार चिनाई इकाइयों वाला चिनाई निर्माण / Masonry Construction with Rectangular Masonry Units

General requirements for construction of masonry walls using rectangular masonry units are:

10.6.3.1 *चिनाई इकाइयां / Masonry Units*

- Well burnt bricks conforming to IS 1077: 1992 or solid concrete blocks conforming to IS 2185 (Part 1): 1979 and having a crushing strength not less than 3.5 MPa shall be used. The strength of masonry unit required shall depend on the number of storeys and thickness of walls.
- Squared stone masonry, stone block masonry or hollow concrete block masonry, as specified in IS 1597 (Part 2): 1992 of adequate strength, may also be used.

Table – 10.3 Recommended Mortar Mixes (IS 4326: 1993)

*Category of Construction	Proportion of Cement-Lime-Sand†
A	M_2 (Cement-sand 1 : 6) or M_3 (Lime-cinder‡ 1 : 3) or richer
B, C	M_2 (Cement-lime-sand 1 : 2 : 9 or Cement-Sand 1 : 6) or richer
D, E	H_2 (Cement-sand 1 : 4) or M_1 (Cement-lime-Sand 1 : 1 : 6) or richer

NOTE — Though the equivalent mortar with lime will have less strength at 28 days, their strength after one year will be comparable to that of cement mortar.

*Category of construction is defined in Table 2.

†Mortar grades and specification for types of limes etc, are given in IS 1905 : 1987.

‡In this case some other pozzolanic material like Surkhi (burnt brick fine powder) may be used in place of cinder.

10.6.3.2 *गारा / Mortar*

- Mortars, such as those given in Table – 10.3 or of equivalent specification, shall preferably be used for masonry

construction for various categories of buildings.

- Where steel reinforcing bars are provided in masonry the bars shall be embedded with adequate cover in cement sand mortar not leaner than 1:3 (minimum clear cover 10 mm) or in cement concrete of grade M15 (minimum clear cover 15 mm or bar diameter whichever more), so as to achieve good bond and corrosion resistance.

10.6.4 दीवारें / Walls

- Masonry bearing walls built in mortar, as specified in 10.6.3.2 above, unless rationally designed as reinforced masonry shall not be built of greater height than 15 m subject to a maximum of four storeys when measured from the mean ground level to the roof slab or ridge level.
- The bearing walls in both directions shall be straight and symmetrical in plan as far as possible.
- The wall panels formed between cross walls and floors or roof shall be checked for their strength in bending as a plate or as a vertical strip subjected to the earthquake force acting on its own mass.

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

10.6.5 चिनाई बॉण्ड / Masonry Bond

For achieving full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course. To obtain full bond between perpendicular walls, it is necessary to make a slopping (stepped) joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise, the toothed joint (as shown in Fig. 10.14) should be made in both the walls alternatively in lifts of about 450 mm.

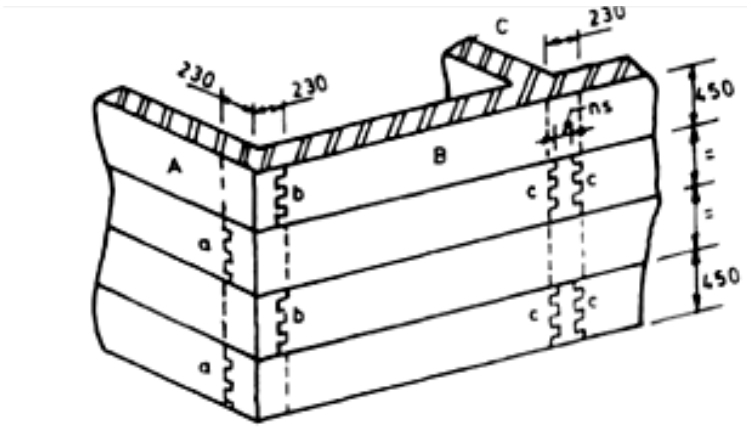


Fig. 10.14 Alternating Toothed Joints in Walls at Corner and T-Junction (IS 4326: 1993)

- Panel or filler walls in framed buildings shall be properly bonded to surrounding framing members by means of suitable mortar (as given in 10.6.3.2 above) or connected through dowels.

10.7 ओपनिंग का प्रभाव / Influence of Openings

Openings are functional necessities in buildings. During earthquake shaking, inertia forces act in the strong direction of some walls and in the weak direction of others. Walls shaken in the weak direction seek support from the other walls, *i.e.*, walls B1 and B2 seek support from walls A1 and A2 for shaking in the direction. To be more specific, wall B1 pulls walls A1 and A2, while wall B2 pushes against them.

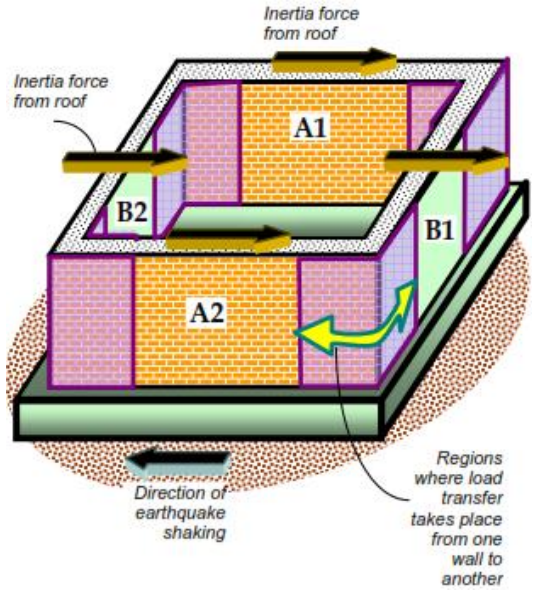
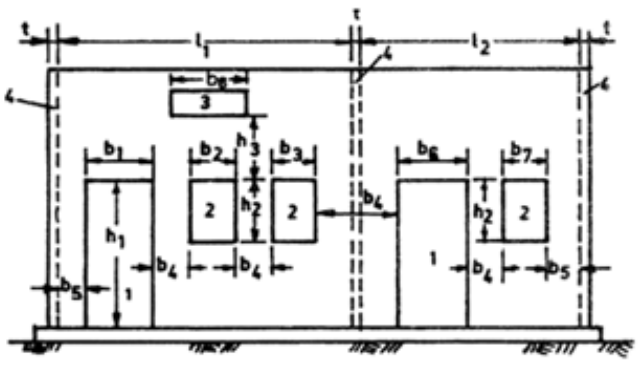


Fig. 10.15 Regions of force transfer from weak walls to strong walls in a masonry building – *Wall B1 pulls walls A1 and A2, while wall B2pushes walls A1 and A2*

Thus, walls transfer loads to each other at their junctions (and through the lintel bands and roof). Hence, the masonry courses from the walls meeting at corners must have good interlocking (Fig. 10.15). For this reason, openings near the wall corners are detrimental to good seismic performance. Openings too close to wall corners hamper the flow of forces from one wall to another. Further, large openings weaken walls from carrying the inertia forces in their own plane. Thus, it is best to keep all openings as small as possible and as far away from the corners as possible.

10.8 धारक दीवारों में ओपनिंग प्रदान करने की सामान्य आवश्यकताएँ / General Requirements of Providing Openings in Bearing Walls

- Door and window openings in walls reduce their lateral load resistance and hence, should preferably be small and more centrally located. The guidelines on the size and position of opening are given in Table – 10.4 and in Fig. 10.16.



1. Door 2. Ventilator 3. Window 4. Cross Wall

Fig. 10.16 Dimensions of Openings and Piers for Recommendations in Table 3 (IS 4326: 1993)

Table – 10.4 Size and Position of Openings in Bearing Walls				
S. No.	**Position 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	450 mm
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*	340 mm	450 mm	560 mm
4	Vertical distance between two openings one above the other h_3, *Min*	600 mm	600 mm	600 mm
5	Width of opening of ventilator b_8, *Max*	900 mm	900 mm	900 mm

- 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.

- Where openings do not comply with the guidelines as given in Table – 10.4, they should be strengthened by providing reinforced concrete or reinforcing the brickwork, as shown in Fig. 10.17 with high strength deformed (H.S.D.) bars of 8 mm dia but the quantity of steel shall be increased at the jambs.

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

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

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

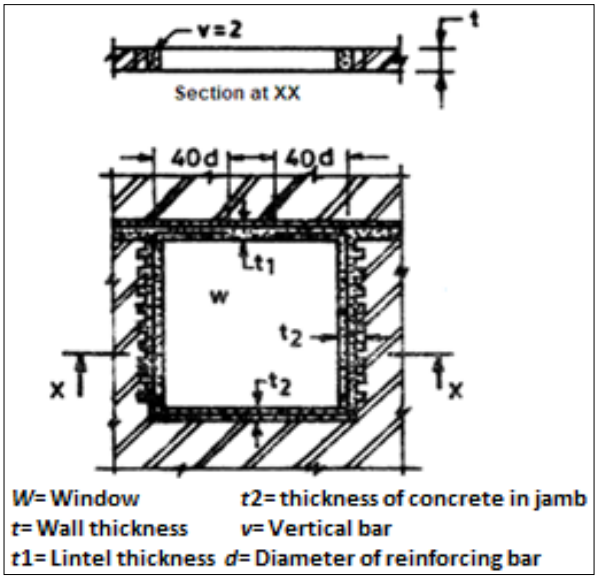


Fig. 10.17 Strengthening Masonry around Opening (IS 4326: 1993)

10.9 भूकंपी सुदृढ़ीकरण व्यवस्था / Seismic Strengthening Arrangements

All masonry buildings shall be strengthened as specified for various categories of buildings, as listed in Table – 10.5.

Table – 10.5 Strengthening Arrangements Recommended for Masonry Buildings (Rectangular Masonry Units)(IS 4326: 1993)		
Building Category	**Number of Storeyes**	**Strengthening to be Provided in all Storeys**
A	i) 1 to 3 ii) 4	a a, b, c
B	i) 1 to 3 ii) 4	a, b, c, f, g a, b, c, d, f, g
C	i) 1 and 2 ii) 3 and 4	a, b, c, f, g a to g
D	i) 1 and 2 ii) 3 and 4	a to g a to h
E	1 to 3*	a to h

Where,
a — Masonry mortar
b — Lintel band
c — Roof band and gable band where necessary
d — Vertical steel at corners and junctions of walls
e — Vertical steel at jambs of openings
f — Bracing in plan at tie level of roofs
g — Plinth band where necessary, and
h — Dowel bars
*4th storey not allowed in category E.

NOTE — *In case of four storey buildings of category B, the requirements of vertical steel may be checked through a seismic analysis using a design seismic coefficient equal to four times the one given in (a)* ***3.4.2.3*** *of IS 1893: 1984. (This is because the brittle behaviour of masonry in the absence of vertical steel results in much higher effective seismic force than that envisaged in the seismic coefficient provided in the code). If this analysis shows that vertical steel is not required the designer may take the decision accordingly.*

The overall strengthening arrangements to be adopted for category D and E buildings which consist of horizontal bands of reinforcement at critical levels, vertical reinforcing bars at corners, junctions of walls and jambs of opening are shown in Fig. 10.18 & 10.19.

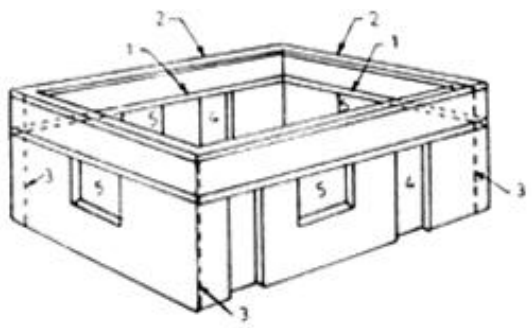


Fig. 10.18 Overall Arrangement of Reinforcing Masonry Buildings (IS 4326: 1993)

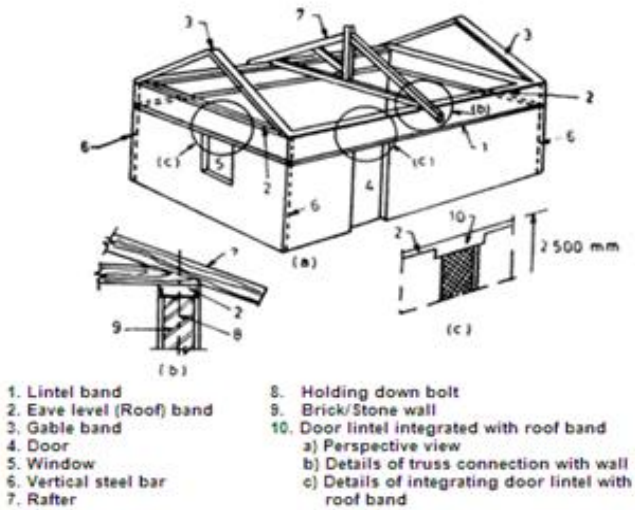


Fig. 10.19 Overall Arrangement of Reinforcing Masonry Building having Pitched Roof (IS 4326: 1993)

10.9.1 पट्टे का अनुभाग एवं सुदृढीकरण / Section and Reinforcement of Band

The band shall be made of reinforced concrete of grade not leaner than M15 or reinforced brickwork in cement mortar not leaner than 1:3. The bands shall be of the full width of the wall, not less than 75 mm in depth and reinforced with steel, as indicated in Table – 10.6.

Table – 10.6 Recommended Longitudinal Steel in Reinforced Concrete Bands (IS 4326: 1993)

Span	Building Category B		Building Category C		Building Category D		Building Category E	
	No. of Bars	Dia.	No. of Bars	Dia.	No. of Bars	Dia.	No. of Bars	Dia.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
m		mm		mm		mm		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 Span of wall will be the distance between centre lines of its cross walls or buttresses. For spans greater than 8 m it will be desirable to insert pillasters or buttresses to reduce the span or special calculations shall be made to determine the strength of wall and section of band.

2 The number and diameter of bars given above pertain to high strength deformed bars. If plain mild-steel bars are used keeping the same number, the following diameters may be used:

High Strength Def. Bar dia. 8 10 12 16 20

Mild Steel Plain bar dia. 10 12 16 20 25

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

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

5 Concrete mix shall be of grade M15 of IS 456: 1978 or 1: 2: 4 by volume.

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

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

As illustrated in Fig. 10.20 –

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

- In category D and E buildings, to further iterate the box action of walls steel dowel bars may be used at corners and T-junctions of walls at the sill level of windows to length of 900 mm from the inside corner in each wall. Such dowel may be in the form of

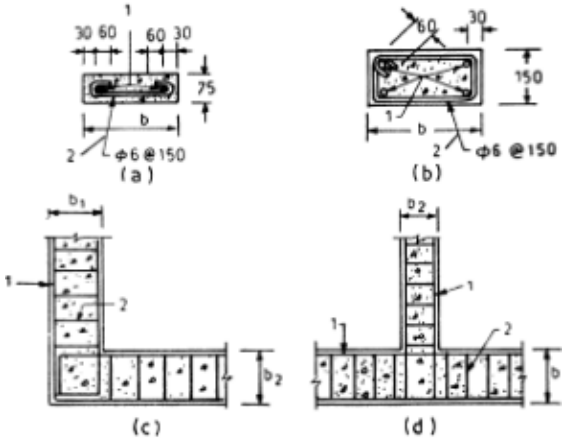


Fig. 10.20 Reinforcement and Bending Detail in RC Band ((IS 4326: 1993)

U stirrups 8 mm dia. Where used, such bars must be laid in 1:3 cement-sand-mortar with a minimum cover of 10 mm on all sides to minimize corrosion.

10.10 भूकंप के दौरान स्टोन चिनाई की दीवारों का व्यवहार / Behaviour of Stone Masonry Walls during Earthquakes

Stone has been used in building construction in India since ancient times since it is durable and locally available. The buildings made of thick stone masonry walls (thickness ranges from 600 to 1200 mm) are one of the most deficient building systems from earthquake-resistance point of view.

The main deficiencies include excessive wall thickness, absence of any connection between the two *wythes* of the wall, and use of *round* stones (instead of *shaped* ones) (Fig. 10.21 & 10.22).

Note: *A wythe is a continuous vertical section of masonry one unit in thickness. A wythe may be independent of, or interlocked with, the adjoining wythe (s). A single wythe of brick that is not structural in nature is referred to as a veneer.* (https://en.wikipedia.org/wiki/Wythe)



Fig. 10.21 Separation of a thick wall into two layers



Fig. 10.22 Separation of unconnected adjacent walls at junction

The main patterns of earthquake damage include:

(a) bulging/ separation of walls in the horizontal direction into two distinct *wythes*,
(b) separation of walls at corners and T-junctions,
(c) separation of poorly constructed roof from walls, and eventual collapse of roof, and
(d) disintegration of walls and eventual collapse of the whole dwelling

In the 1993 Killari (Maharashtra) earthquake alone, over 8,000 people died, most of them buried under the rubble of traditional stone masonry dwellings. Likewise, a majority of the over 13,800 deaths during 2001 Bhuj (Gujarat) earthquake is attributed to the collapse of this type of construction.

10.11 भूकंप प्रतिरोधी स्टोन चिनाई भवन के निर्माण हेतु सामान्य सिद्धांत / General principle for construction of Earthquake Resistant stone masonry building

10.11.1 भूकंप प्रतिरोधी लक्षण / Earthquake Resistant Features

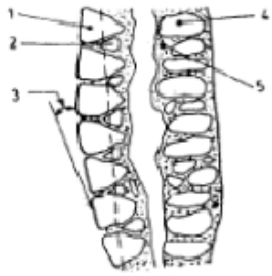
1. Low strength stone masonry buildings are weak against earthquakes, and should be avoided in high seismic zones. Inclusion of special earthquake-resistant features may enhance the earthquake resistance of these buildings and reduce the loss of life. These features include:

(a) Ensure proper wall construction
(b) Ensure proper bond in masonry courses
(c) Provide horizontal reinforcing elements
(d) Control on overall dimensions and heights

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

3. The wall thickness should not be larger than 450 mm. Preferably it should be about 350 mm, and the stones on the inner and outer *wythes* should be interlocked with each other.

NOTE - If the two wythes are not interlocked, they tend to delaminate during ground shaking bulge apart (as shown in Fig. 10.23) and buckle separately under vertical load leading to complete collapse of the wall and the building.

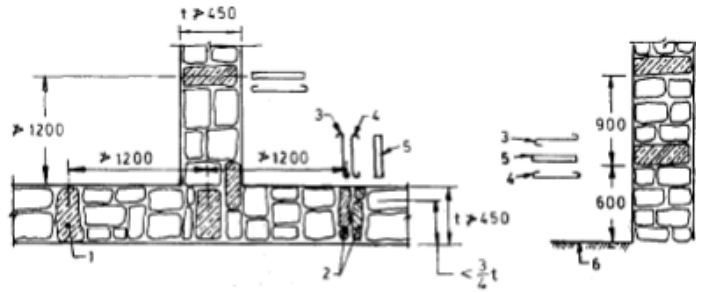


1. Half-dressed conical stone, 2. Small alignment stone
3. Rotation of Wythe, 4. Random rubble,
5. Mud or weak lime mortar.

Fig. 10.23 Wall delaminated with buckled withes (IS 13828: 1993)

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

5. **'Through' stones** at 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 (as shown in Fig. 10.24).



(a) Sectional plan of wall (b) Cross-section of wal
1. Through stone, 2. Pair of overlapping stone, 3. S-Shape tie, 4. Hooked tie,
5. Wood plank, 6. Floor level.
All dimensions in millimetres.

Fig. 10.24 Through Stone and Bond Elements (IS 13828: 1993)

6. 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 (as shown in Fig. 10.24). Alternatively, wood-bars of 38 mm X 38 mm cross section or concrete bars of 50 mm X50 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.

7. Use of 'bonding' elements of adequate length should also be made at corners and junctions of walls to break the vertical joints and provide bonding between perpendicular walls.

8. 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:

a) For categories A and 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.

b) 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, respectively.

9. 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. Top width should be equal to the thickness of main wall, t, and the base width equal to one sixth of wall height.

10. The stone masonry dwellings must have horizontal bands (*plinth*, *lintel*, *roof* and *gable bands*). These bands can be constructed out of wood or reinforced concrete, and chosen based on economy. It is important to provide at least one band (either *lintel* band or *roof* band) in stone masonry construction.

Note: *Although, this type of stone masonry construction practice is deficient with regards to earthquake resistance, its extensive use is likely to continue due to tradition and low cost. But, to protect human lives and property in future earthquakes, it is necessary to follow proper stone masonry construction in seismic zones III and higher. Also, the use of seismic bands is highly recommended.*

***

अध्याय / Chapter- 11

भूकंपीय मूल्यांकन और रेट्रोफ़िटिंग / SEISMIC EVALUATION AND RETROFITTING

There are considerable number of buildings that do not meet the requirements of current design standards because of inadequate design / or construction errors and need structural upgrading specially to meet the seismic requirements.

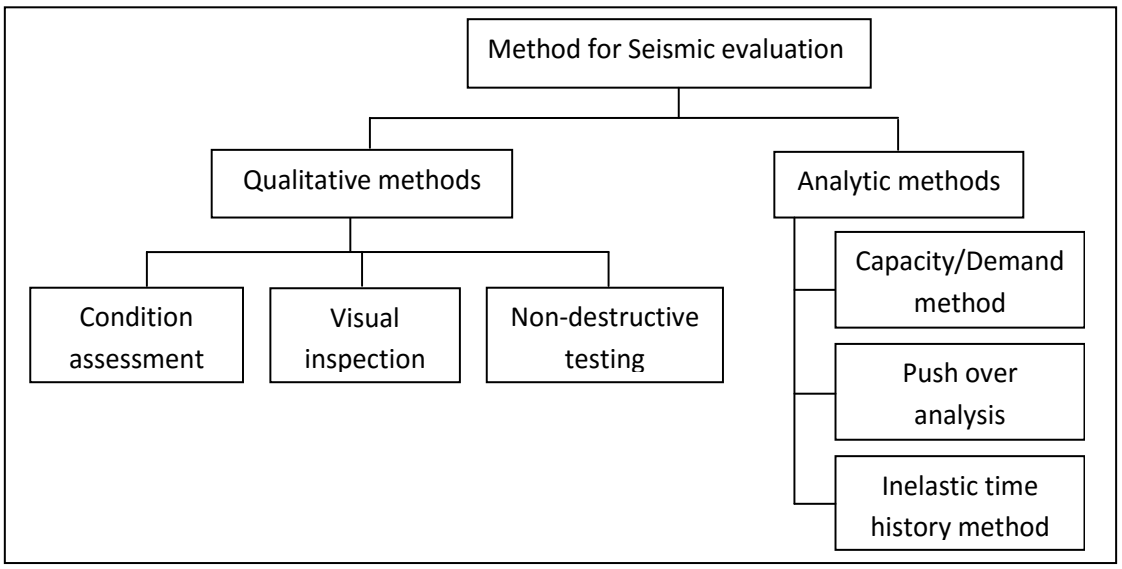
Retrofitting is the best solution to strengthen such buildings without replacing them.

11.1 भूकंपीय मूल्यांकन / SEISMIC EVALUATION

Seismic evaluation is to assess the seismic response of buildings, which may be seismically deficient or earthquake damaged for their future use. The evaluation is also helpful in choosing appropriate retrofitting techniques.

The methods available for seismic evaluation of existing buildings can be broadly divided into two categories:

1. Qualitative methods
2. Analytical methods



11.1.1 गुणात्मक तरीके / QUALITATIVE METHODS

The **qualitative methods** are based on the available background information of the structures, past performance of similar structures under severe earthquakes, visual inspection report, some non-destructive test results etc.

The evaluation of any building is a difficult task, which requires a wide knowledge about the structures, cause and nature of damage in structures and its components, material strength etc.

The proposed methodology is divided into three components:

1. **Condition assessment**

It is based on:

- data collection or information gathering of structures from architectural and structural drawings
- performance characteristics of similar type of buildings in past earthquakes
- rapid evaluation of strength, drift, materials, structural components and structural details

2. **Visual inspection/Field evaluation:** It is based on observed distress and damage in structures. Visual inspection is more useful for damaged structures however it may also be conducted for undamaged structures.

3. **Non-destructive evaluation:** It is generally carried out for quick estimation of materials strength, determination of the extent of determination and to establish causes remain out of reach from visual inspection and determination of reinforcement and its location. NDT may also be used for preparation of drawing in case of non-availability.

11.1.1.1 Condition Assessment for Evaluation

The aim of condition assessment of the structure is the collection of information about the structure and its past performance characteristics to similar type of structure during past earthquakes and the qualitative evaluation of structure for decision-making purpose. More information can be included, if necessary as per requirement.

(i) <u>Data collection / information gathering</u>

Collection of the data is an important portion for the seismic evaluation of any existing building. The information required for the evaluated building can be divided as follows:

<u>Building Data</u>

- Architectural, structural and construction drawings
- Vulnerability parameters: number of stories, year of construction, and total floor area
- Specification, soil reports, and design calculations
- Seismicity of the site

<u>Construction Data</u>

- Identifications of gravity load resisting system
- Identifications of lateral load resisting system
- Maintenance, addition, alteration, or modifications in structures
- Field surveys of the structure’s existing condition

Structural Data

- Materials
- Structural concept: vertical and horizontal irregularities, torsional eccentricity, pounding, short column and others
- Detailing concept: ductile detailing, special confinement reinforcement
- Foundations
- Non-structural elements

(ii) Past Performance data

Past performance of similar type of structure during the earthquake provides considerable amount of information for the building, which is under evaluation process. Following are the areas of concerns, which are responsible for poor performance of buildings during earthquake.

Material concerns

- Low grade on concrete
- Deterioration in concrete and reinforcement
- High cement-sand ratio
- Corrosion in reinforcement
- Use of recycled steel as reinforcement
- Spalling of concrete by the corrosion of embedded reinforcing bars
- Corrosion related to insufficient concrete cover
- Poor concrete placement and porous concrete

Structural concerns

- The relatively low stiffness of the frames: excessive inter-storey drifts, damage to non-structural items
- Pounding: column distress, possibly local collapse
- Unsymmetrical buildings (U, T, L, V) in plan: torsional effects and concentration of damage at the junctures (i.e. re-entrant corners)
- Unsymmetrical buildings in elevation: abrupt change in lateral resistance
- Vertical strength discontinuities: concentrate damage in the “soft” stories
- Short column

Detailing concerns

- Large tie spacing in columns lack of confinement of concrete core: shear failures
- Insufficient column lengths: concrete to spall
- Locations of inadequate splices: brittle shear failure
- Insufficient column strength for full moment hinge capacity: brittle shear failure
- Lack of continuous beam reinforcement: hinge formation during load reversals
- Inadequate reinforcing of beam column joints or location of beam bar splices at columns: joint failures
- Improper bent-up of longitudinal reinforcing in beams as shear reinforcement: shear failure during load reversal

- Foundation dowels that are insufficient to develop the capacity of the column steel above: local column distress

(iii) Seismic Evaluation Data

Seismic evaluation of data will provide a general idea about the building performance during an earthquake. The criteria of evaluation of building will depend on materials, strength and ductility of structural components and detailing of reinforcement.

Material Evaluation

- Buildings height > 3 stories, minimum grade concrete M 20, desirable M 30 to M 40 particularly in columns of lower stories
- Maximum grade of steel should be Fe 415 due to adequate ductility
- No significant deterioration in reinforcement
- No evidence of corrosion or spalling of concrete

Structural components

- Evaluation of columns shear strength and drift: check for permissible limits
- Evaluation of plan irregularities: check for torsional forces and concentration of forces
- Evaluation of vertical irregularities: check for soft storey, mass or geometric discontinuities
- Evaluation of beam-column joints: check for strong column-weak beams
- Evaluation of pounding: check for drift control or building separation
- Evaluation of interaction between frame and infill: check for force distribution in frames and overstressing of frames

(i) Flexural members
- Limitation of sectional dimensions
- Limitation on minimum and maximum flexural reinforcement: at least two continuous reinforced bars at top and bottom of the members
- Restriction of lap splices
- Development length requirements: for longitudinal bars
- Shear reinforcement requirements: stirrup and tie hooks, tie spacing, bar splices

(ii) Columns
- Limitation of sectional dimensions
- Longitudinal reinforcement requirement
- Transverse reinforcement requirements: stirrup and tie hooks, column tie spacing, column bar splices
- Special confining requirements

(iii) Foundation
- Column steel doweled into the foundation

Non-structural components

- Cornices, parapet, and appendages are anchored
- Exterior cladding and veneer are well anchored

11.1.1.2 Field Evaluation/ Visual Inspection Method

The procedure for visual inspection method is as below:

Equipments

- Optical magnification: allows a detailed view of local areas of distress
- Stereomicroscope: that allow a three dimensional view of the surface. Investigator can estimate the elevation difference in surface features by calibrating the focus adjustment screw
- Fibrescope and borescopes: allow inspection of regions that are inaccessible to the naked eye
- Tape: to measure the dimension of structure, length of cracks
- Flashlight: to aid in lighting the area to be inspected, particularly in post-earthquake evaluation, power failure
- Crack comparator: to measure the width of cracks at representative locations, two types: plastic cards and magnifying lens comparators
- Pencil: to draw the sketch of cracks
- Sketchpad: to prepare a representation of wall elevation, indicating the location of cracks, spalling, or other damage, records of significant features such as non-structural elements
- Camera: for photographs or video tape of the observed cracking

Action

- Perform a walk through visual inspection to become familiar with the structure
- Gather background documents and information on the design, construction, maintenance, and operation of structure
- Plan the complete investigation
- Perform a detailed visual inspection and observe type of damage: cracks spalls and delaminations, permanent lateral displacement, and buckling or fracture of reinforcement, estimating of drift
- Observe damage documented on sketches: interpreted to assess the behaviour during earthquake
- Perform any necessary sampling: basis for further testing

Data Collection

- To identify the location of vertical structural elements: columns and walls
- To sketch the elevation with sufficient details: dimensions, openings, observed damage such as cracks, spalling, and exposed reinforcing bars, width of cracks
- To take photographs of cracks: use marker, paint or chalk to highlight the fine cracks or location of cracks in photographs
- Observation of the non-structural elements: inter-storey displacement

Limitations

- Applicable for surface damage that can be visualised
- No identification of inner damage: health monitoring of building, chang of frequency and mode shapes

11.1.1.3 Non-destructive testing (NDT)

Visual inspection has the obvious limitation that only visible surface can be inspected. Internal defects go unnoticed and no quantitative information is obtained about the properties of the concrete. For these reasons, a visual inspection is usually supplemented by NDT methods. Other detailed testing is then conducted to determine the extent and to establish causes.

NDT tests for condition assessment of structures

Some methods of field and laboratory testing that may assess the minimum concrete strength and condition and location of the reinforcement in order to characterize the strength, safety, and integrity are:

(i) Rebound hammer/ Swiss hammer

The rebound hammer is the most widely used non-destructive device for quick surveys to assess the quality of concrete. In 1948, Ernest Schmidt, a Swiss engineer, developed a device for testing concrete based upon the rebound principal strength of in-place concrete; comparison of concrete strength in different locations and provides relative difference in strength only.

Limitations:

- Not give a precise value of compressive strength, provide estimate strength for comparison
- Sensitive to the quality of concrete; carbonation increases the rebound number
- More reproducible results from formed surface rather than finished surface; smooth hard-towelled surface giving higher values than a rough-textured surface
- Surface moisture and roughness also affect the reading; a dry surface results in a higher rebound number
- Not take more than one reading at the same spot

(ii) Penetration resistance method – Windsor probe test

Penetration resistance methods are used to determine the quality and compressive strength of in-situ concrete. It is based on the determination of the depth of penetration of probes (steel rods or pins) into concrete by means of power-actuated driver. This provides a measure of the hardness or penetration resistance of the material that can be related to its strength.

Limitations:

- Both probe penetration and rebound hammer test provide means of estimating the relative quality of concrete not absolute value of strength of concrete.
- Probe penetration results are more meaningful than the results of rebound hammer.
- Because of greater penetration in concrete, the prove test results are influenced to a lesser degree by surface moisture, texture, and carbonation effect.
- Probe test may be the cause of minor cracking in concrete.

(iii) Rebar locator/convert meter

It is used to determine quantity, location, size and condition of reinforcing steel in concrete. It is also used for verifying the drawing and preparing as-built data, if no previous information is available. These devices are based on interaction between the reinforcing bars and low frequency electromagnetic fields. Commercial convert meter can be divided into two classes: those based on the principal of magnetic reluctance and those based on eddy currents.

Limitations:

- Difficult to interpret at heavy congestion of reinforcement or when depth of reinforcement is too great.
- Embedded metals sometimes affect the reading.
- Used to detect the reinforcing bars closest to the face.

(iv) Ultrasonic pulse velocity

It is used for determination the elastic constants (modulus of elasticity and Poisson's ratio) and the density. By conducting tests at various points on a structure, lower quality concrete can be identified by its lower pulse velocity. Pulse-velocity measurements can detect the presence of voids of discontinuities within a wall; however, these measurements can not determine the depth of voids.

Limitations:

- Moisture content: an increase in moisture content increases the pulse velocity
- Presence of reinforcement oriented parallel to the pulse propagation direction: the pulse may propagate through the bars and result is an apparent pulse velocity that is higher than that propagating through concrete
- Presence of cracks and voids: increases the length of the travel path and result in a longer travel time

(v) Impact echo

Impact echo is a method for detecting discontinuities within the thickness of a wall. An impact-echo test system is composed of three components: an impact source, a receiving transducer, and a waveform analyzer or a portable computer with a data acquisition.

Limitations:

- Accuracy of results highly dependent on the skill of the engineer and interpreting the results
- The size, type, sensitivity, and natural frequency of the transducer, ability of FFT analyzer also affect the results
- Mainly used for concrete structures

(vi) Spectral analysis of surface waves (SASW)

To assess the thickness and elastic stiffness of material, size and location of discontinuities within the wall such as voids, large cracks, and delimitations.

Limitations:

- Interpretation of results is very complex
- Mainly used on slab and other horizontal surface, to determine the stiffness profiles of soil sites and of flexible and rigid pavement systems, measuring the changes in elastic properties of concrete slab

(vii) Penetrating radar

It is used to detect the location of reinforcing bars, cracks, voids or other material discontinuities, verify thickness of concrete.

Limitations:

- Mainly used for detecting subsurface condition of slab-on-grade
- Not useful for detecting the small difference in materials
- Not useful for detecting the size of bars, closely spaced bars make difficult to detect features below the layer of reinforcing steel

11.1.2 विश्लेषणात्मक तरीके / ANALYTICAL METHODS

Analytical methods are based on considering capacity and ductility of the buildings, which are based on detailed dynamic analysis of buildings. The methods in this category are capacity/demand method, pushover analysis, inelastic time history analysis etc. Brief discussions on the method of evaluation are as follows:

11.1.2.1 Capacity/Demand (C/D) method

- The forces and displacements resulting from an elastic analysis for design earthquake are called demand.
- These are compared with the capacity of different members to resist these forces and displacements.
- A (C/D) ratio less than one indicate member failure and thus needs retrofitting.
- When the ductility is considered in the section, the demand capacity ratio can be equated to section ductility demand of 2 or 3.
- The main difficulty encountered in using this method is that there is no relationship between member and structure ductility factor because of non-linear behaviour.

11.1.2.2 Push Over Analysis

- The push over analysis of a structure is a static non-linear analysis under permanent vertical loads and gradually increasing lateral loads.
- The equivalent static lateral loads approximately represent earthquake-induced forces.
- A plot of total base shear verses top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness.
- The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity.
- On a building frame, load/displacement is applied incrementally, the formation of plastic hinges, stiffness degradation, and plastic rotation is monitored, and lateral inelastic force versus displacement response for the complete structure is analytically computed.
- This type of analysis enables weakness in the structure to be identified. The decision to retrofit can be taken on the basis of such studies.

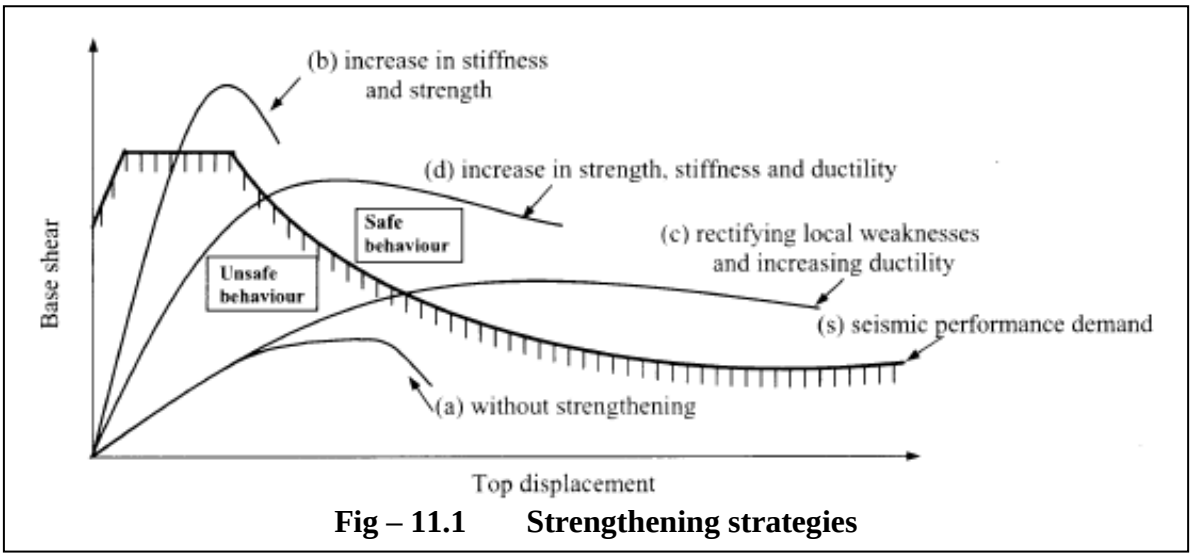


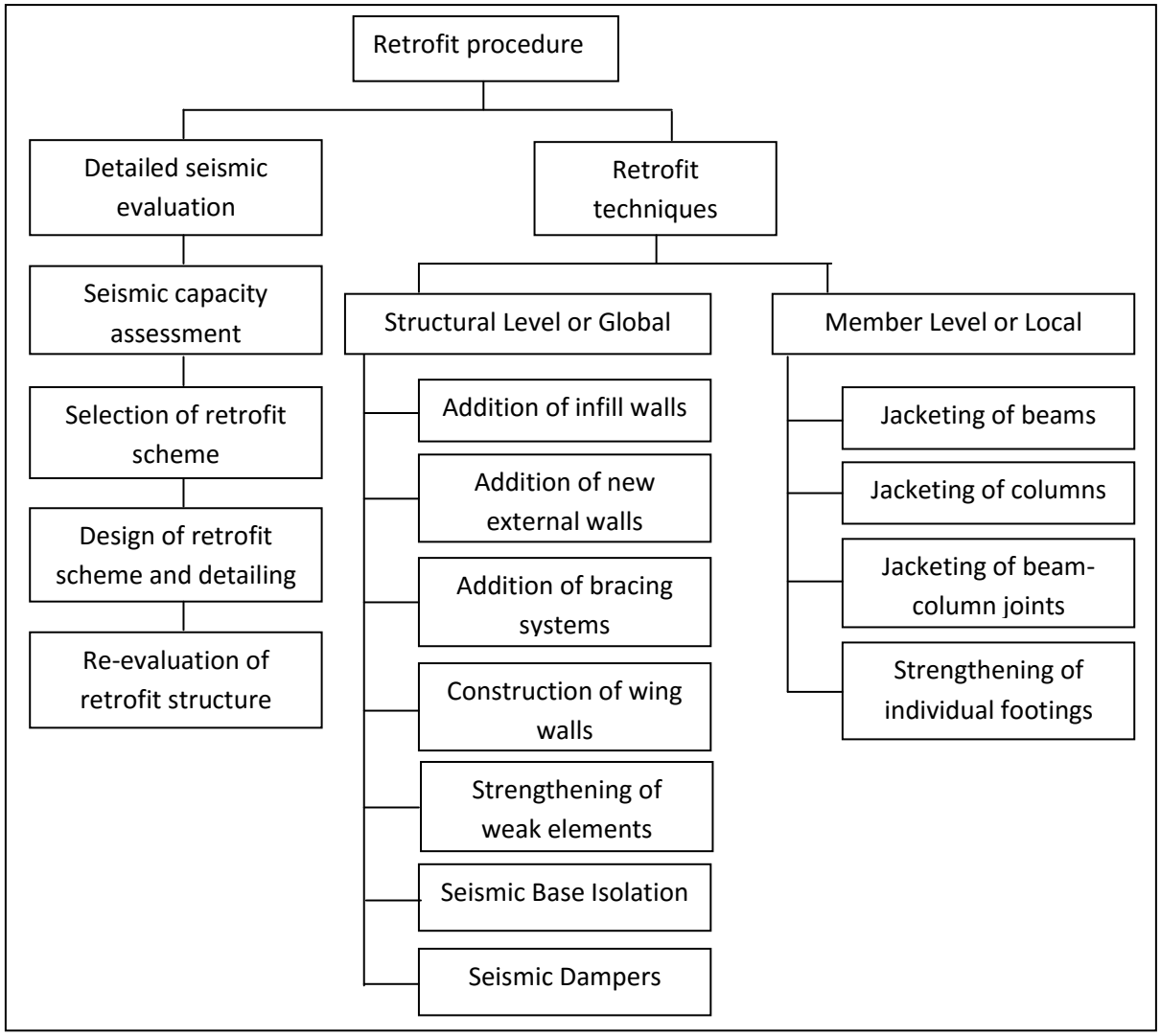
Fig – 11.1 Strengthening strategies

11.1.2.3 Inelastic time-history analysis

- A seismically deficient building will be subjected to inelastic action during design earthquake motion.
- The inelastic time history analysis of the building under strong ground motion brings out the regions of weakness and ductility demand in the structure.
- This is the most rational method available for assessing building performance.
- There are computer programs available to perform this type of analysis.
- However, there are complexities with regard to biaxial inelastic response of columns, modelling of joints behaviour, interaction of flexural and shear strength and modelling of degrading characteristics of member.
- The methodology is used to ascertain deficiency and post-elastic response under strong ground shaking.

11.2 भवनों की रेट्रोफ़िटिंग / Retrofitting of Building

Retrofitting is to upgrade the strength and structural capacity of an existing structure to enable it to safely withstand the effect of strong earthquakes in future.



11.2.1 स्ट्रक्चरल लेवल या ग्लोबल रेट्रोफ़िट तरीके / Structural Level or Global Retrofit Methods

Two approaches are used for structural-level retrofitting:

(i) Conventional Methods
(ii) Non-conventional methods

11.2.1.1 Conventional Methods

Conventional Methods are based on increasing the seismic resistance of existing structure. The main categories of these methods are as follow:

a) Addition of infilled walls
b) Addition of new external walls
c) Addition of bracing system
d) Construction of wing walls
e) Strengthening of weak elements

11.2.1.1.1 Addition of infilled walls

The construction of infill walls within the frames of the load bearing structures, as shown in the example of Fig – 11.2, aims to drastically increase the strength and the stiffness of the structure. This method can also be applied in order to correct design errors in the structure and, more specifically, when a large asymmetric distribution of strength or stiffness in elevation or an eccentricity of stiffness in plan have been recognised.

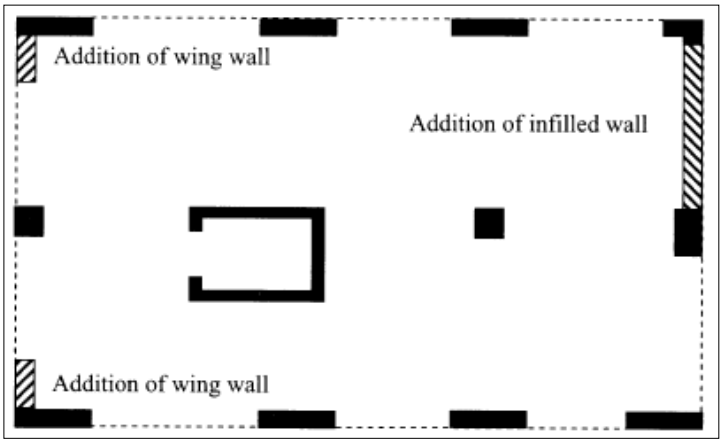


Fig - 11.2 Addition of infilled wall and wing walls

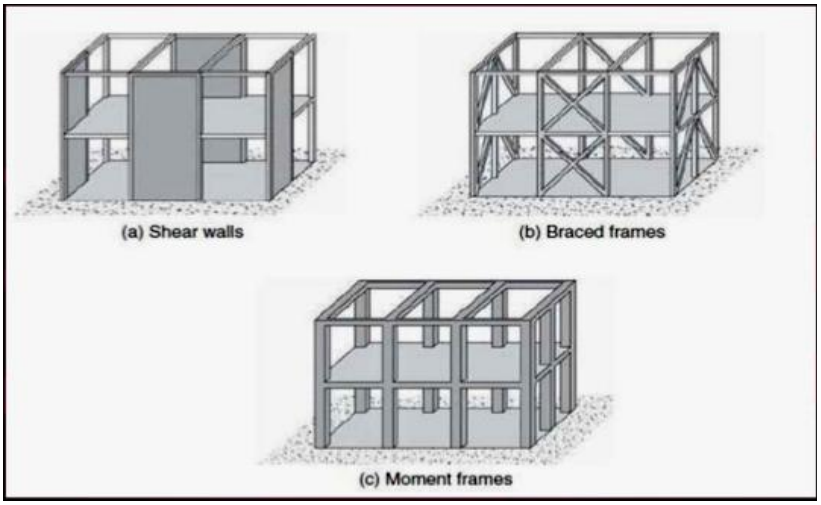


Fig - 11.3 Frames and shear wall

As shown in Fig – 11.4, there are two alternatives methods of adding infill walls. Either the infill wall is simply placed between two existing columns or it is extended around the columns to form a jacket. The second method is specifically recommended in order to increase the strength in this region. In the situation where the existing columns are very weak, a steel cage should be placed around the columns before constructing new walls and column jackets. In all cases, the base of any new wall should always be connected to the existing foundation.

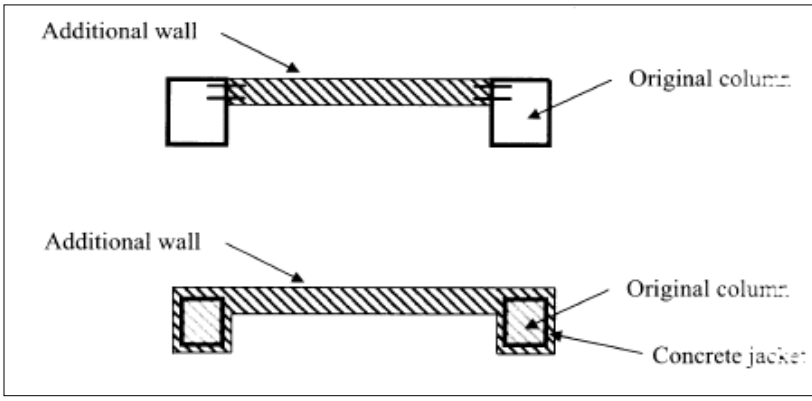


Fig – 11.4 Two alternative methods of adding infill walls

11.2.1.1.2 Addition of new external walls

In some cases, strengthening by adding concrete walls can be performed externally. This can often be carried out for functional reasons as, for example, in cases when the building must be kept in operation during the intervention works. New cast-in-place concrete walls, constructed outside the building, can be designed to resist part or all the total seismic forces induced in the building. The new walls are preferably positioned adjacent to vertical elements (columns or walls) of the building and are connected to the structure by placing special compression, tensile or shear connectors at every floor level of the building. As shown in Figure 11.5, new walls usually have a L-shaped cross-section and are constructed to be in contact with the external corners of the building.

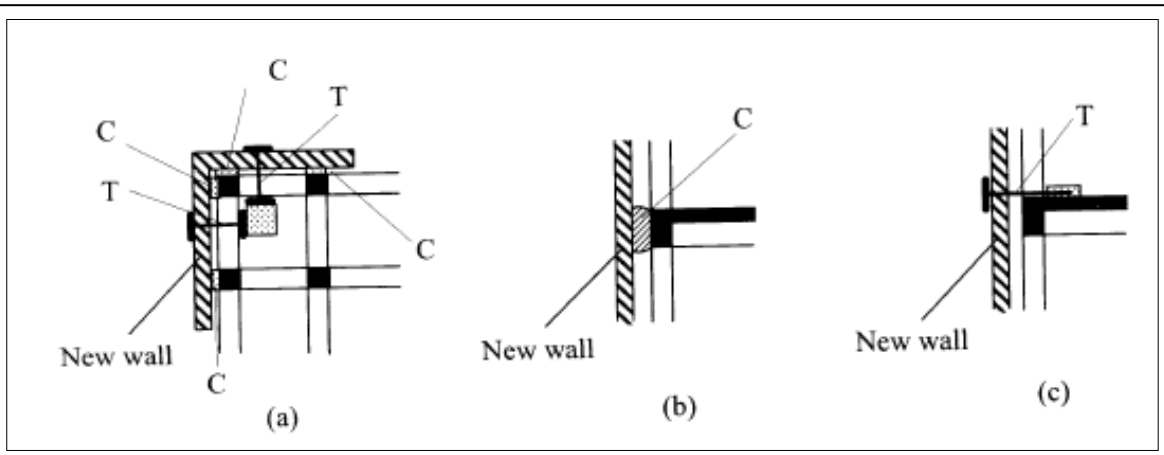


Fig – 11.5 Schematic arrangement of connections between the existing building and a new wall (a) plan, (b) section of compression connector and (c) section of tension connector

It is important to ensure that connectors behave elastically under seismic design action effects. For this reason, when designing the connectors, a resistance safety factor equal to 1.4 is recommended. The use of compression and tensile connectors, instead of shear connectors, is strongly recommended as much higher forces can be transferred. It is essential that the anchorage areas for the connectors on the existing building and on the new walls have enough strength to guarantee the transfer of forces between new walls and the existing structures.

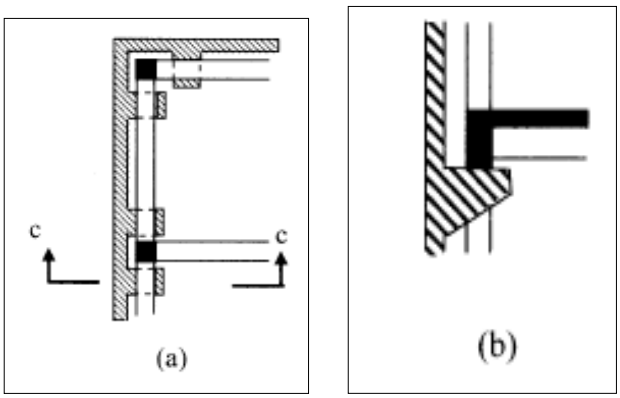


Fig – 11.6 Construction of cantilever beams to transfer axial forces to new walls (a) plan, (b) section c-c

A very important issue of the above method concerns the foundation of new walls. Foundation conditions should be improved if large axial forces can be induced in new walls during seismic excitation. In addition, the construction of short cantilever beams protruding from the wall, underneath the adjacent beams at every floor level of the building, as shown in Fig – 11.6, appears to be a good solution.

11.2.1.1.3 Addition of bracing systems

The construction of bracing within the frames of the load bearing structure aims for a high increase in the stiffness and a considerable increase in the strength and ductility of the structure. Bracing is normally constructed from steel elements, rather than reinforced concrete, as the elastic deformation of steel aids the absorption of seismic energy.

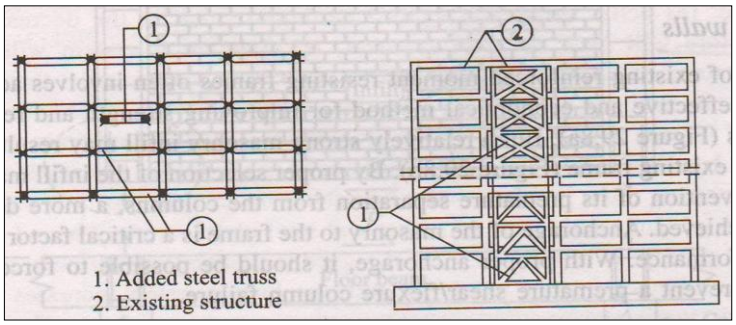


Fig – 11.7 Reinforced Concrete Building retrofitted with steel bracing

Bracing systems can be used in a similar way as that for steel constructions and can be applied easily in single-storey industrial buildings with a soft storey ground floor level where no or few brick masonry walls exist between columns.

Various truss configurations have been applied in practice, examples of which are: K-shaped, diamond shaped or cross diagonal. The latter is the most common and is often the most effective solution.

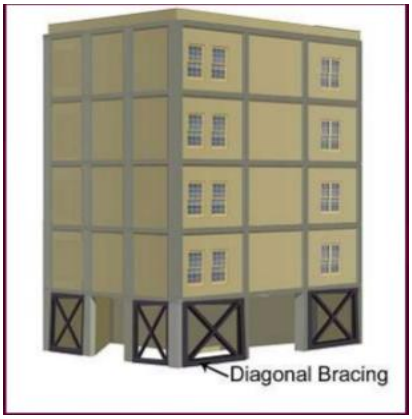


Fig – 11.8 Steel bracing soft storey

Use of steel bracing has a potential advantage over other schemes for the following reasons:

- Higher strength and stiffness can be proved
- Opening for natural light can be made easily
- Amount of work is less since foundation cost may be minimised
- Bracing system adds much less weight to the existing structure
- Most of the retrofitting work can be performed with prefabricated elements and disturbance to the occupants may be minimised



Fig – 11.9 Steel bracing

11.2.1.1.4 Construction of wing wall

The construction of reinforced concrete wing walls in continuous connection with the existing columns of a structure, as shown above in example of Fig – 11.2, is a very popular technique.

As presented in Fig – 11.10, there are two alternative methods of connecting the wing wall to the existing load bearing structure.

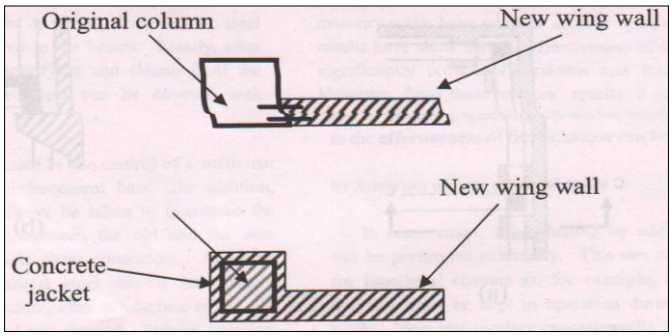


Fig – 11.10 Construction of reinforced concrete wing wall

- In the first method, the wall is connected to the column and the beams at the top and the base of any floor level. Steel dowels or special anchors are used for the connection and the reinforcement of the new wall is welded to the existing reinforcement.
- In the second method, the new wing wall is extended around the column to form a jacket. Obviously, in this case, stresses at the interface between the new concrete and the existing column are considerably lower when compared to the first method.

Moreover, uncertainties regarding the capacity of the connection between the wall and the column do not affect the seismic performance of the strengthened element. Therefore, the second alternative method is strongly recommended.

11.2.1.1.5 Strengthening weak elements

The selective strengthening of weak elements of the structure aims to avoid a premature failure of the critical elements of a building and to increase the ductility of the structure.

Usually, this method is applied to vertical elements and is accompanied by the construction of fibre reinforced polymer (FRP) jackets or, as shown in Fig- 11.11, steel cages around the vertical elements.

If a strength increase is also required, this method can include the construction of column jackets of shotcrete or reinforced concrete.

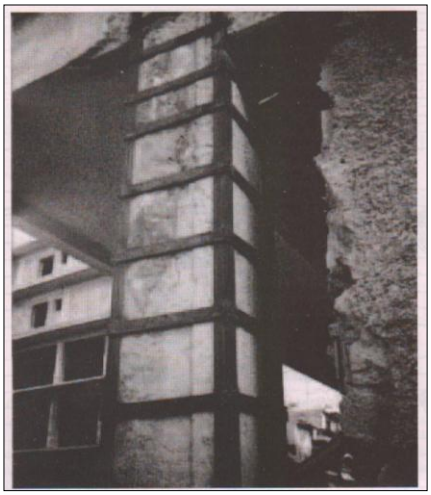


Fig – 11.11 Construction of a steel cage around a vertical element

11.2.1.2 Non-conventional methods

These are based on reduction of seismic demands. Seismic demands are the force and displacement resulting from an elastic analysis for earthquake design. Incorporation of energy absorbing systems to reduce seismic demands are as follows:

(i) Seismic Base Isolation
(ii) Seismic Dampers

11.2.1.2.1 Seismic Base Isolation

- Isolation of superstructure from the foundation is known as base isolation
- It is the most powerful tool for passive structural vibration control technique



Fig – 11.12 Base isolated structures

<u>Types of base isolations</u>

<u>Elastomeric Bearings</u>

- This is the most widely used Base Isolator
- The elastomer is made of either Natural Rubber or Neoprene
- The structure is decoupled from the horizontal components of the earthquake ground motion

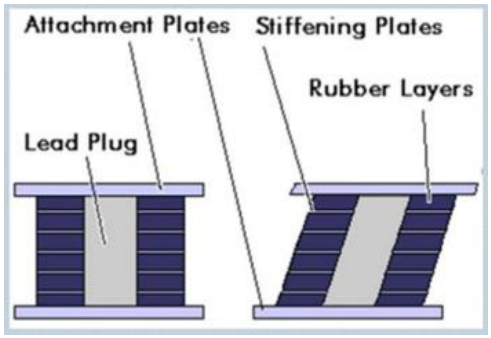


Fig – 11.13 Elastomeric Isolators

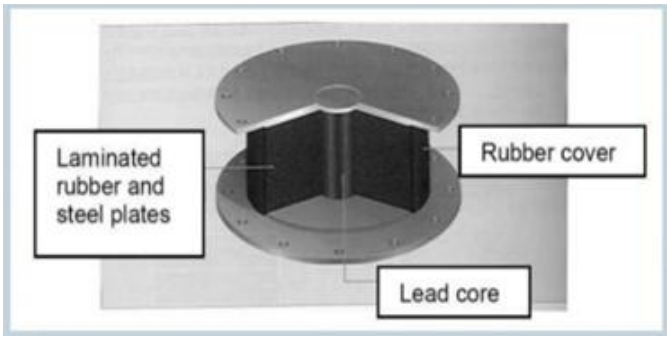


Fig – 11.14 Steel Reinforced Elastomeric Isolators

Sliding System

a) Sliding Base Isolation Systems

- It is the second basic type of isolators
- This works by limiting the base shear across the isolator interface



Fig – 11.15 Metallic Roller Bearing

b) Spherical Sliding Base Isolators

- The structure is supported by bearing pads that have curved surface and low friction
- During an earthquake, the building is free to slide on the bearings

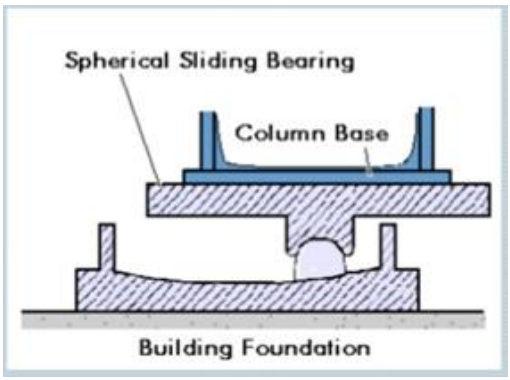


Fig – 11.16 Spherical Sliding Base Isolators

c) Friction Pendulum Bearing

- These are specially designed base isolators which works on the principle of simple pendulum
- It increases the natural time period of oscillation by causing the structure to

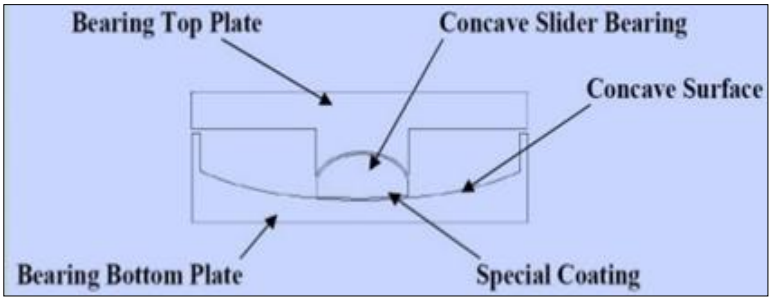


Fig – 11.17 Cross-section of Friction Pendulum Bearing

slide along the concave inner surface through the frictional interface
- It also possesses a re-centering capability
- Typically, bearings measure 1.0 m (3 feet) in dia., 200 mm (8 inches) in height and weight being 2000 pounds

d) Advantages of base isolation

- Isolate building from ground motion
- Building can remain serviceable throughout construction
    - ❖ Lesser seismic loads, hence lesser damage to the structure
    - ❖ Minimal repair of superstructure
- Does not involve major intrusion upon existing superstructure

e) Disadvantages of base isolation

- Expensive
- Cannot be applied partially to structures unlike other retrofitting
- Challenging to implement in an efficient manner
- Allowance for building displacements
- Inefficient for high rise buildings
- Not suitable for buildings rested on soft soil

11.2.1.2.2 Seismic Dampers

- Seismic dampers are used in place of structural elements, like diagonal braces, for controlling seismic damage in structures
- It partly absorbs the seismic energy and reduces the motion of buildings
- Types:
    - ❖ **Viscous Dampers**: Energy is absorbed by silicon-based fluid passing between piston-cylinder arrangement

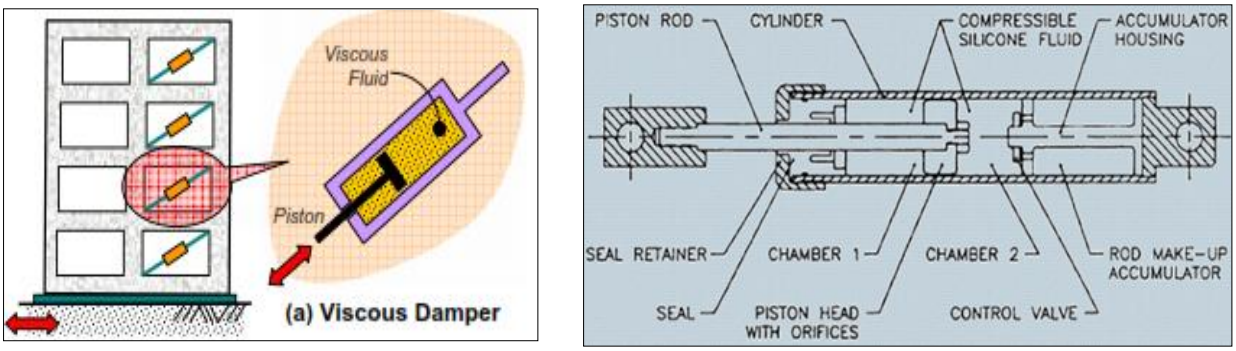


Fig -11.18 Cross-section of a Viscous Fluid Damper

❖ **Friction Dampers**: Energy is absorbed by surfaces with friction between rubbing against each other

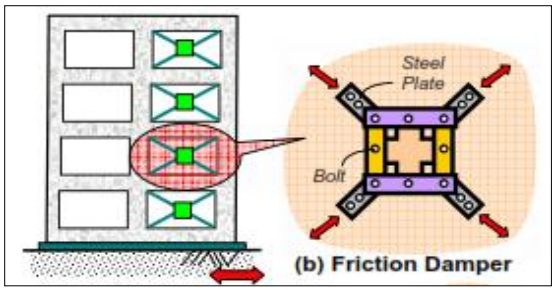


Fig – 11.19 Friction Dampers

❖ **Yielding Dampers**: Energy is absorbed by metallic components that yield

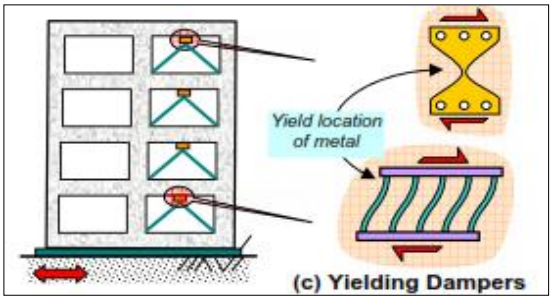


Fig – 11.20 Yielding Dampers

11.2.2 सदस्य स्तर या स्थानीय रिट्रोफाइट तरीके / Member Level or Local Retrofit Methods

The member level retrofit or local retrofit approach is to upgrade the strength of the members, which are seismically deficient. This approach is more cost effective as compared to the structural level retrofit.

Jacketing

The most common method of enhancing the individual member strength is jacketing. It includes the addition of concrete, steel, or fibre reinforced polymer (FRP) jackets for use in confining reinforced concrete columns, beams, joints and foundation.

Types of jacketing

(1) Concrete jacketing (2) Steel jacketing (3) Strap jacketing

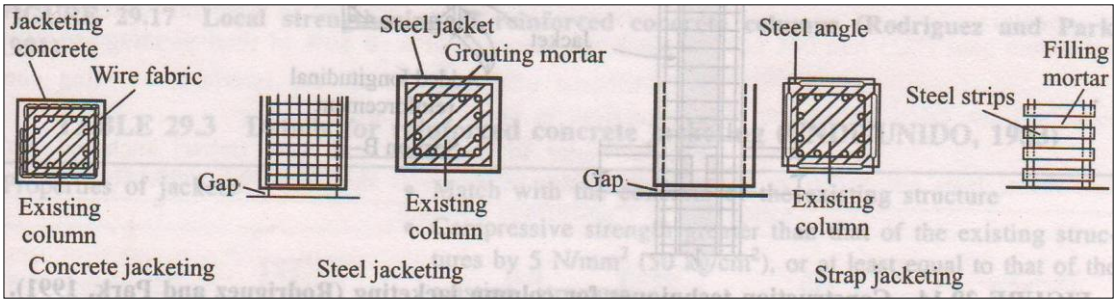


Fig – 11.21 Type of Jacketing

11.2.2.1 Member level Jacketing

(i) Jacketing of Columns

Different methods of column jacketing are as shown in Figures below:

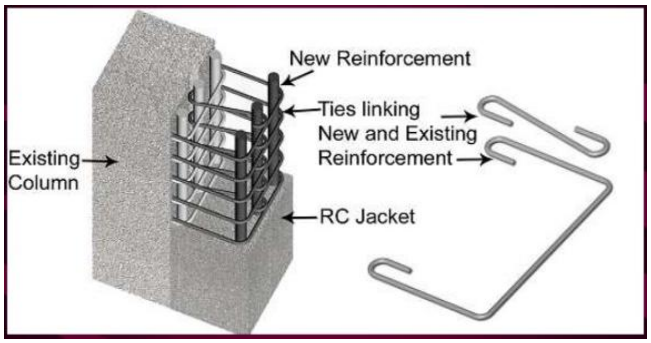


Fig – 11.22 (a) Reinforced Concrete Jacketing



Fig – 11.22 (b) Column with CFRP (Carbon Fibre Reinforced Polymer) Wrap



Fig – 11.22 (c) Column with Steel Jacketing



Fig – 11.22 (d) Column with Steel Caging

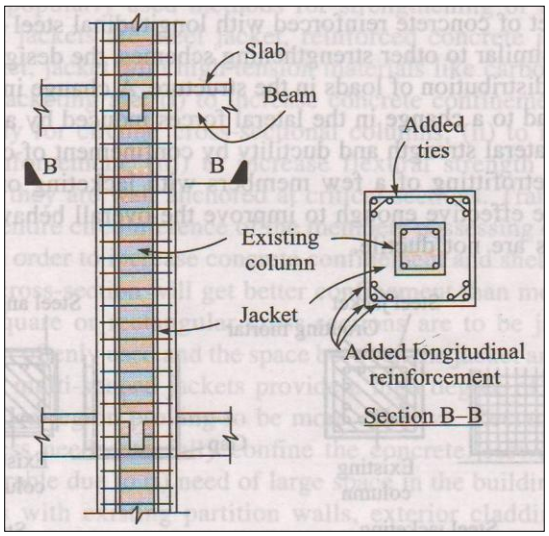


Fig – 11.22 (e) Construction techniques for column jacketing

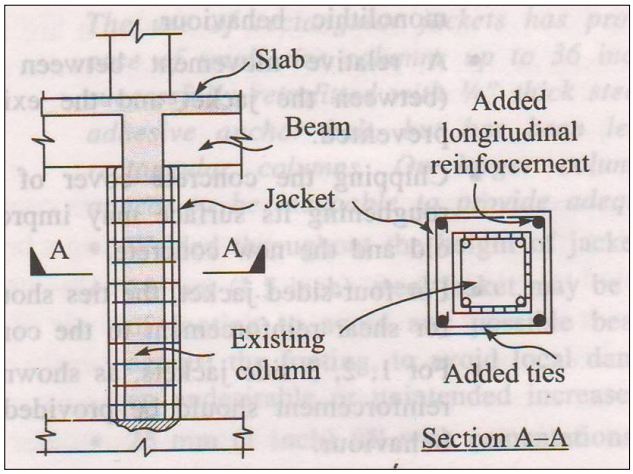


Fig – 11.22 (f) Local strengthening of RC Columns

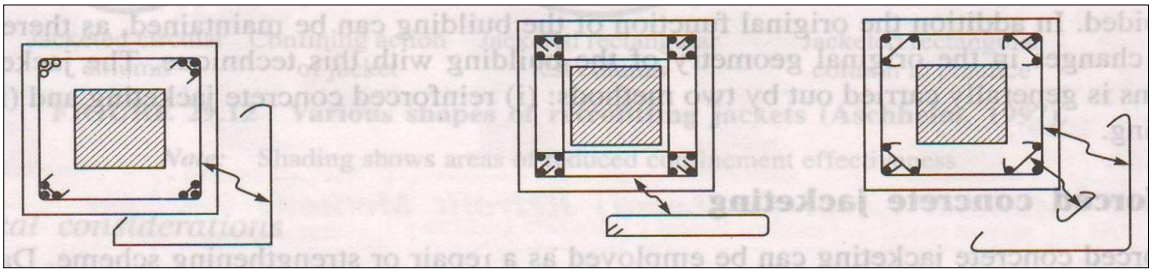


Fig – 11.22 (g) Details for provision of longitudinal reinforcement

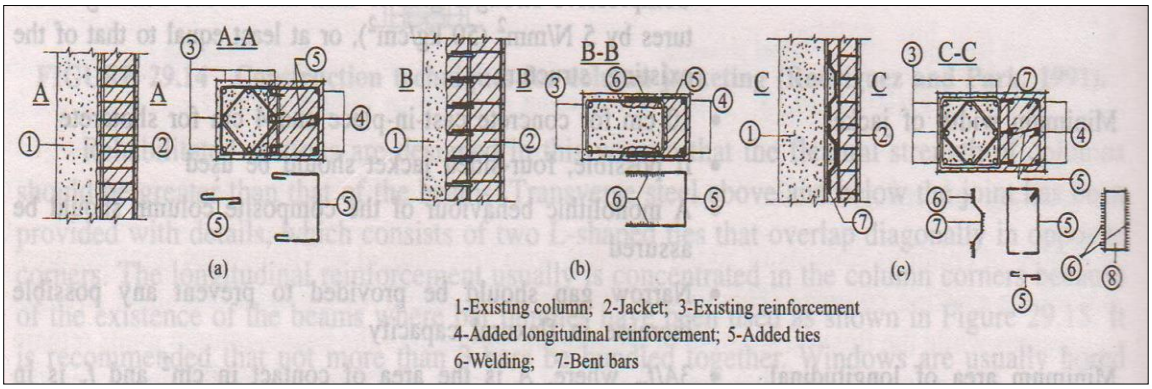


Fig – 11.22 (h) Different methods of column jacketing

(ii) Jacketing of Beam

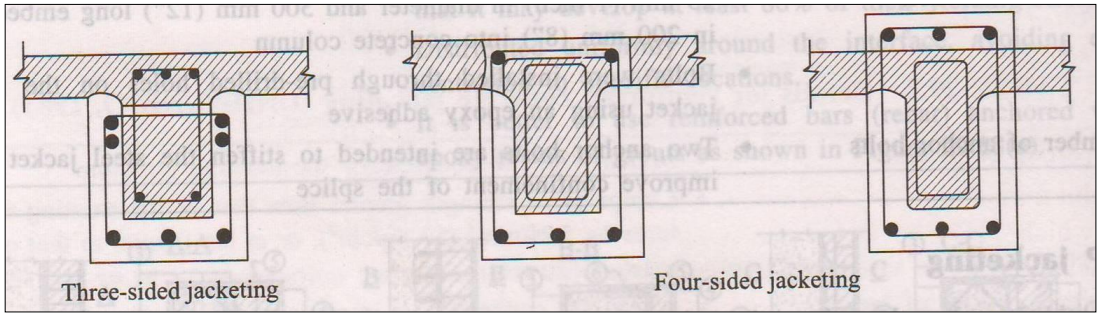
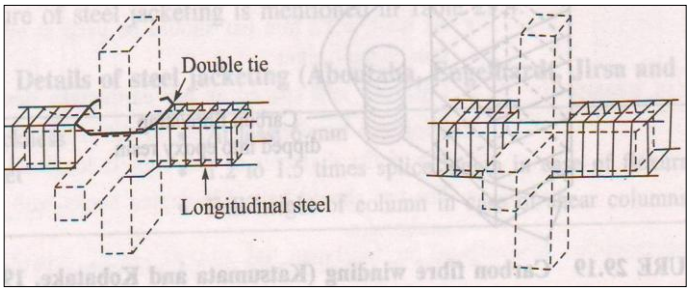


Fig – 11.23 Different ways of beam jacketing



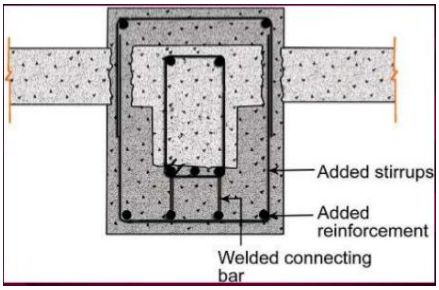


Fig – 11.24 Continuity of longitudinal steel in jacketed beams

(iii) Jacketing of Beam-Column Joint

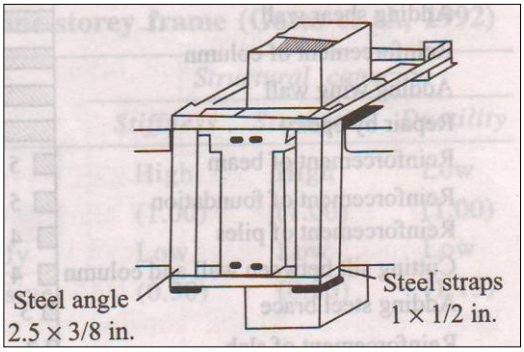


Fig – 11.25 Steel cage assembled in the joint

11.2.2.2 Table showing the details of reinforced concrete jacketing

Properties of jackets	• match with the concrete of the existing structure • compressive strength greater than that of the existing structures by 5 N/mm^2 (50 kg/cm^2), or at least equal to that of the existing structure
Minimum width of jacket	• 10 cm for concrete cast-in-place and 4 cm for shotcrete • If possible, four sided jacket should be used • A monolithic behaviour of the composite column should be assured • Narrow gap should be provided to prevent any possible increase in flexural capacity
Minimum area of longitudinal reinforcement	• $3A/f_{y,}$ where, A is the area of contact in cm^2 and f_y *is in* kg/cm^2 • Spacing should not exceed six times of the width of the new elements (the jacket in the case) up to the limit of 60 cm • Percentage of steel in the jacket with respect to the jacket area should be limited between 0.015 and 0.04 • At least, a 12 mm bar should be used at every corner for a four sided jacket
Minimum area of transverse reinforcement	• Designed and spaced as per earthquake design practice • Minimum bar diameter used for ties is not less than 10 mm diameter anchorage • Due to the difficulty of manufacturing 135 degree hooks on the field, ties made up of multiple pieces, can be used
Shear stress in the interface	• Provide adequate shear transfer mechanism to assured monolithic behaviour • A relative movement between both concrete interfaces (between the jacket and the existing element) should be prevented • Chipping the concrete cover of the original member and roughening its surface may improve the bond between the old and the new concrete • For four sided jacket, the ties should be used to confine and for shear reinforcement to the composite element • For 1, 2, 3 side jackets, as shown in Figures, special reinforcement should be provided to enhance a monolithic behaviour
Connectors	• Connectors should be anchored in both the concrete such that it may develop at least 80% of their yielding stress • Distributed uniformly around the interface, avoiding concentration in specific locations • It is better to use reinforced bars (rebar) anchored with epoxy resins of grouts as shown in Figure (a)

11.2.2.3 Practical aspects in choosing appropriate techniques

Certain issues of practical importance that may help to avoid mistakes in choosing the appropriate technique are as follows:

1) The strengthening of columns by using FRPs or steel jackets is unsuitable for flexible structures where failure would be controlled by deflection. In this case, the strengthening should aim to increase the stiffness.

2) It is not favourable to use steel cages or confine with FRPs when an increase in the flexural capacity of vertical elements is required.

3) The application of confinement (with FRPs or steel) to circular or rectangular columns would increase the ductility and the shear strength and would limit the slippage of overlapping bars when the lap length has been found to be insufficient. However, a significant contribution cannot be expected for columns of rectangular cross section with a large aspect ratio, or those with L-shaped cross sections.

4) In the case of columns that have heavily rusted reinforcement, strengthening with FRP jackets (or the application of epoxy glue) will protect the reinforcement from further oxidation. However, if the corrosion of the reinforcement is at an advanced stage, it is probable that strengthening may not stop the premature failure of the element.

5) The construction of FRP jackets around vertical elements will increase the ductility but it cannot increase the buckling resistance of the longitudinal reinforcement bars. Thus, if the stirrups are too thin in an existing element, failure will probably result from the premature bending of the vertical reinforcement. In this case, local stress concentrations from the distressed bars will build up between the stirrups and will lead to a local failure of the jacket. Consequently, if bending of the vertical reinforcement has been evaluated as the most likely cause of column failure, the preferable choice for strengthening of the element would be to place a steel cage.

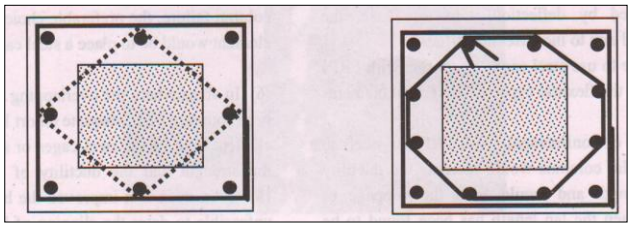
6) In areas where the overlapping of reinforcement bars has been found to be inadequate (short lap lengths), confining the element with FRPs, steel cages or steel jackets will improve the strength and the ductility of the region considerably. However, even if it improved the behaviour, it is eventually unfeasible to deter the slipping of bars. Consequently, when the lap length of bars has been found to be smaller than 30% of code requirements, the solution of welding of bars must be selected. Moreover, it must be pointed out that confinement cannot offer anything to longitudinal bars that are not in the corners of the cross section.

7) Experimentally, the procedure of placing FRP sheets to strengthen weak beam-column joints has proved to be particularly effective. In practice, however, this technique has been found to be difficult to apply due to the presence of slabs and transverse beams. The same problems arise when placing steel plates. Other techniques, such as the construction of reinforced concrete jackets or the reconstruction of joints with additional interior reinforcement, appear

to be more beneficial. In cases where only a light damage to the joints has been found, repairing with an epoxy resin appears to be particularly effective solution.

8) The placing of new concrete in contact with an existing element (by shotcreting and especially by pouring) will require prior aggravation of the old surface to a depth of at least 6 mm. This should be performed by sandblasting or by using suitable mechanical equipment (for example, a scabbler and not just simply a hammer and a chisel). This is to remove the exterior weak skin of the concrete and to expose the aggregate.

9) When placing a new concrete jacket around an existing column, it is not always possible to follow code requirements and place internal rectangular stirrups to enclose the middle longitudinal bars, as shown in Fig-11.26(a). In this case, it is proposed to place two middle bars in each side of the jacket, so that octagonal stirrups can be easily placed, as demonstrated in Fig-11.26(b).



(a) **(b)**

Fig – 11.26 Placement of internal stirrups in rectangular cross section

In the case where columns have a cross section with a large aspect ratio, the middle longitudinal bars can be connected by drilling holes through the section in order to place a S-shaped stirrup, as shown in Fig – 11.27. After placing stirrups, the remaining void can be filled with epoxy resin. In order to ease placement, the S-shaped stirrup can be prefabricated with one hook and, after placing, the second hook can be formed by hand.

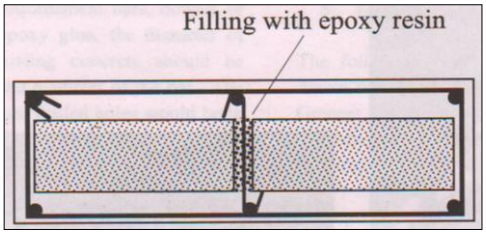
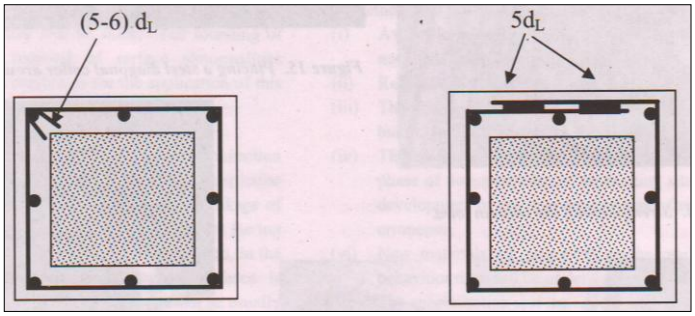


Fig – 11.27 Placement of an internal stirrup in a rectangular cross section with a large aspect ratio

10) If a thin concrete jacket is to be placed around a vertical element and the 135 deg hooks at the ends of the stirrups are impeded by the old column, it would be acceptable to decrease the hook anchorage from 10 times the bar diameter to 5 or 6 times the bar diameter, as shown in Fig – 11.28(a). Otherwise, the ends the stirrups should be welded together or connected with special contacts (clamps), as presented in Fig – 11.28(b), that have now appeared on the market.



(a) **(b)**

Fig – 11.28 Reducing hook lengths and welding the ends of stirrups

11) When constructing a jacket around a column, it is important to also strengthen the column joint. As shown in Fig – 11.29, this can be accomplished by, where possible, extending the longitudinal reinforcement bars around the joint. In addition, as also shown in Fig – 11.29, stirrups must be placed in order to confine the concrete of the jacket around the joint.



Fig – 11.29 Strengthening the column joint

In the case where the joint has been found to be particularly weak, a steel diagonal collar can be placed around the joint before placing the reinforcement, as shown in Fig – 11.30.

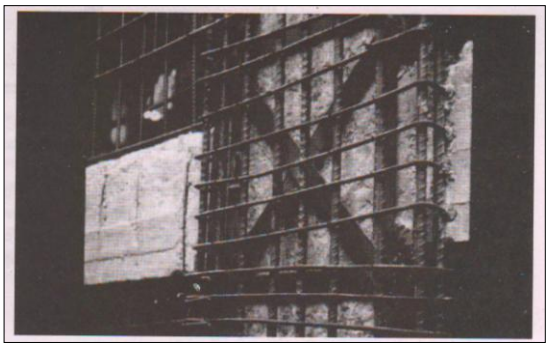


Fig – 11.30 Placing a steel diagonal collar around a weak column joint

12) It is preferable that a new concrete jacket is placed continuously from the foundation to the top of the building. If this is not possible (due to maintaining the functioning of the building), it is usual to stop the jacket at the top of the ground floor level. In this case, there is a need to anchor the jacket's longitudinal bars to the existing column. This can be achieved by anchoring a steel plate to the base of the column of the floor level above and then welding the longitudinal bars to the anchor plate, as shown in Fig – 11.31.

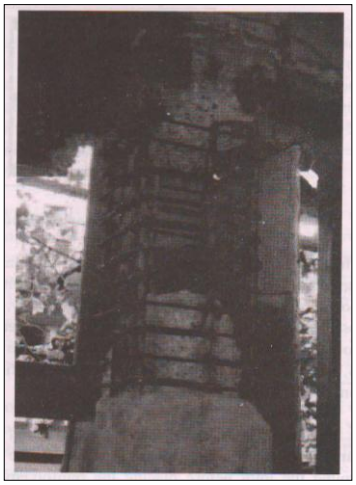


Fig – 11.31 Removal of defective concrete from a heavily damaged column

13) In the case where there is a need to reconstruct a heavily damaged column, after first shoring up the column, all the defective concrete must be removed so that only good concrete

remains, as shown in Fig – 11.32. Any buckled reinforcement bars must be welded to the existing bars. Finally, the column can be recast by placing a special non-shrink concrete.

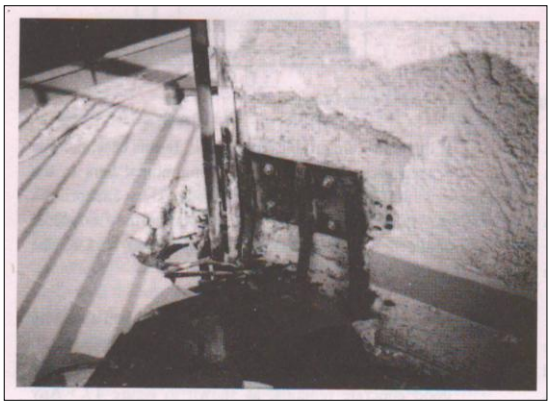


Fig – 11.32 Welding longitudinal bars to an anchor plate

14) In order to anchor new reinforcement bars, dowels or anchors with the use of epoxy glue, the diameter of holes drilled into the existing concrete should be roughly 4 mm larger than the diameter of the bar. The best way to remove dust from drilled holes would be to spray water at the back of the hole. The best results (higher adhesive forces) are achieved when the walls of the hole have been roughened slightly with a small wire brush.

15) Care is required when shotcreting in the presence of reinforcement. There is a danger of an accumulation of material building up behind the bars. This is usually accredited to material sticking to the face of bars and may be due to either a low velocity, a large firing distance or insufficient pressure from the compressor.

16) The placing of steel plates and especially FRP sheets or fabrics requires special preparation of the concrete surface to which they will be stuck. The rounding of corners and the removal of surface abnormalities constitute minimal conditions for the application of this technique.

17) Two constructional issues that concern the connection of new walls to the old frame require particular attention. The first problem is due to the shrinkage of the new concrete and the appearance of cracks at the top of the new wall immediately below the old beam, in the region where a good contact between surfaces is essential. Here, the problem of shrinkage can be usually dealt with by placing concrete of a particular composition where special admixtures (for example, expansive cements) have been used. Alternatively, the new wall could be placed to about 20 cm below the existing beam and after more than 7 days (taking into account temperature and how new concrete shrinks with time), the void can be filled with an epoxy or polyster mortar. In some cases, depending on site conditions (ease of access, dry conditions, etc.), the new wall can be placed to a height of 2 to 5 mm below the beam and the void filled with resin glue using the technique of resin injection. The second problem concerns the case of walls from ready-mix concrete and the difficulty of placing the higher part of the wall due to insufficient access. For this reason alone, the use of shotcrete should be the preferred option.

11.3 आर.सी. भवनों के घटकों में सामान्य भूकंपी क्षतियाँ और उनके उपचार / Common seismic damage in components of R.C. Buildings and their remedies

Possible damages in component of R.C. Buildings which are frequently observed after the earthquakes are as follows:

(i) R. C. Column

The most common modes of failure of column are as follows.

Mode -1: Formation of plastic hinge at the base of ground level columns

Mechanism: The column, when subjected to seismic motion, its concrete begins to disintegrate and the load carried by the concrete shifts to longitudinal reinforcement of the column. This additional load causes buckling of longitudinal reinforcement. As a result the column shortens and looses its ability to carry even the gravity load.



Fig – 11.33 Formation of plastic hinge at the base

Reasons: Insufficient confinement length and improper confinement in plastic hinge region due to smaller numbers of ties.

Remedies: This type of damage is sensitive to the cyclic moments generated during the earthquake and axial load intensity. Consideration is to be paid on plastic hinge length or length of confinement.

Mode – 2: Diagonal shear cracking in mid span of columns

Mechanism: In older reinforced concrete building frames, column failures were more frequent since the strength of beams in such constructions was kept higher than that of the columns. This shear failure brings forth loss of axial load carrying capacity of the column. As the axial capacity diminishes, the gravity loads carried by the column are transferred to neighbouring elements resulting in massive internal redistribution of forces, which is also amplified by dynamic effects causing spectacular collapse of building.

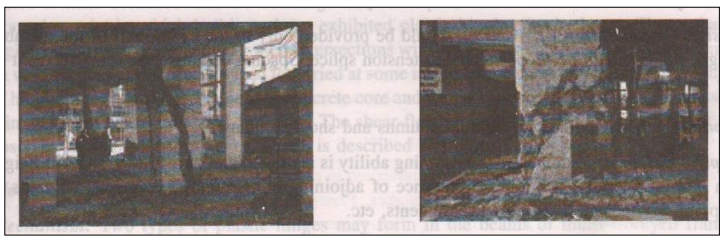


Fig – 11.34 Diagonal shear cracking in mid span of columns

Reason: Wide spacing of transverse reinforcement.

Remedies: To improve understanding of shear strength, as well as to understand how the gravity loads will be supported after a column fails in shear.

Mode – 3: Shear and splice failure of longitudinal reinforcement

Mechanism: Splices of column longitudinal reinforcement in older buildings were commonly designed for compression only with relatively light transverse reinforcement enclosing the lap.

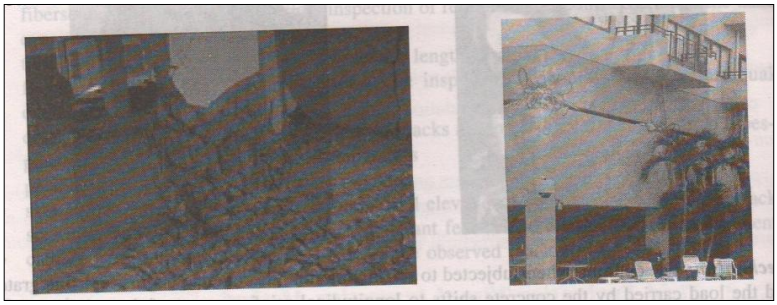


Fig - 11.35 Shear and splice failure of longitudinal reinforcement

Under earthquake motion, the longitudinal reinforcement may be subjected to significant tensile stresses, which require lap lengths for tension substantially exceeding those for compression. As a result slip occurs along the splice length with spalling of concrete.

Reasons: Deficient lap splices length of column longitudinal reinforcement with lightly spaced transverse reinforcement, particularly if the splices just above the floor slab especially the splices just above the floor slab, which is very common in older construction.

Remedies: Lap splices should be provided only in the center half of the member length and it should be proportionate to tension splice. Spacing of transverse reinforcement as per IS 13929:1993.

Mode – 4: Shear failures in captive columns and short columns

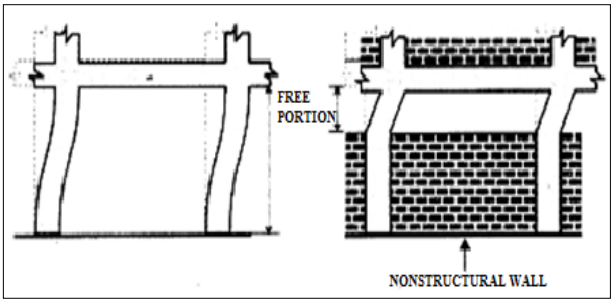


Fig – 11.36 Restriction to the Lateral Displacement of a Column Creating a Captive-Column Effect

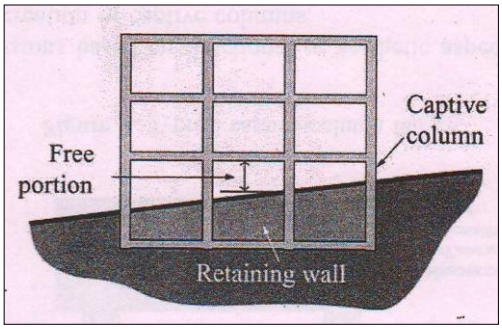


Fig – 11.37 Captive-column effect in a building on sloping terrain

Captive column: Column whose deforming ability is restricted and only a fraction of its height can deform laterally. It is due to presence of adjoining non-structural elements, columns at slopping ground, partially buried basements, etc.

A captive column is full storey slender column whose clear height is reduced by its part-height

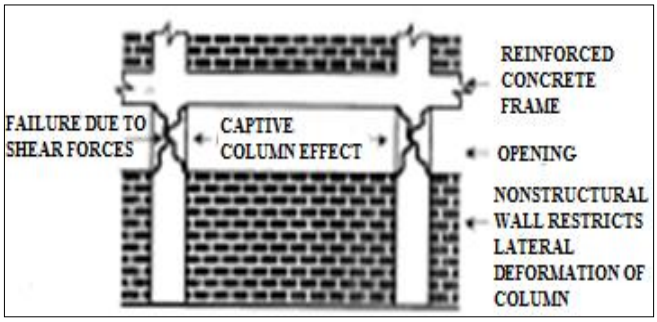


Fig – 11.38 Typical captive-column failure



Fig – 11.39 Column damage due to captive- column effect

contact with a relatively stiff non-structural element, such as a masonry infill wall, which constraints its lateral deformation over the height of contract.

The captive column effect is caused by a non-intended modification to the original structural configuration of the column that restricts the ability of the column to deform laterally by partially confining it with building components. The column is kept "captive" by these components and only a fraction of its height can deform laterally, corresponding to the "free" portion; thus the term captive column. Figure as given below shows this situation.

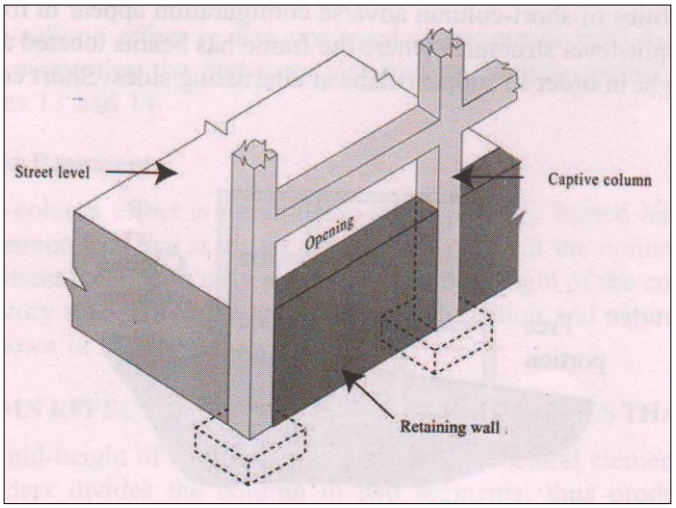


Fig – 11.40 Captive column caused by ventilation openings in a partially buried basement

Short column: Column is made shorter than neighbouring column by horizontal structural elements such as beams, girder, stair way landing slabs, use of grade beams, and ramps.

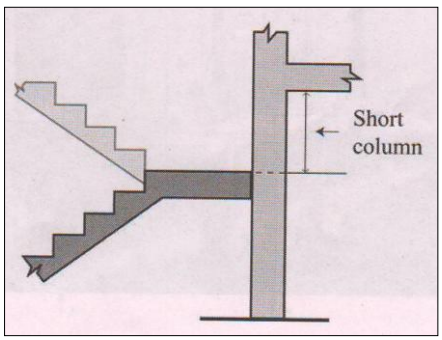


Fig – 11.41 Short column created by a stairway landing

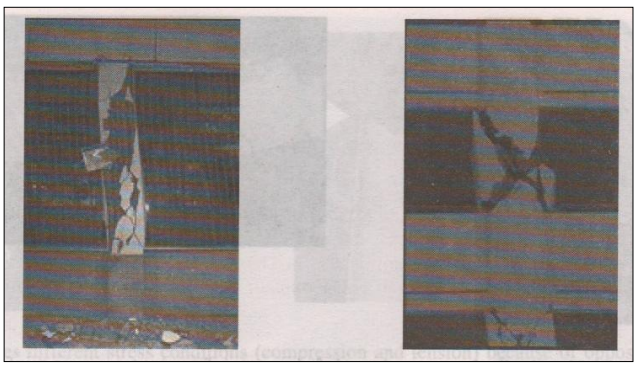


Fig – 11.42 Shear failures in captive columns

For split-level buildings, in order to circumvent the short-column effect, the architect should avoid locating a frame at the vertical plane where the transition between levels occurs. For buildings on slopes, special care should be exercised to locate the sloping retaining walls in such a way that no captive-column effects are induced. Where stiff non-structural walls are still employed, these walls should be separated from the structure, and in no case can they be interrupted before reaching the full height of the adjoining columns.

Mechanism: A reduction in the clear height of captive or short columns increases the lateral stiffness. Therefore, these columns are subjected to larger shear force during the earthquake since the storey shear is distributed in proportion to lateral stiffness of the same floor. If these columns, reinforced with conventional longitudinal and transverse reinforcement, and subjected to relatively high axial loading, fail by splitting of concrete along their diagonals, if the axial loading level is low, the most probable mode of failure is by shear sliding along full depth cracks at the member ends. Moreover, in the case of captive column is so effective that usually damage is shifted to the short non-confined upper section of the column.

Reasons: Large shear stresses, when the structure is subjected to lateral forces are not accounted for in the standard frame design procedure.

Remedies: The best solution for captive column or short column is to avoid the situation otherwise use separation gap in between the non-structural elements and vertical structural element with appropriate measures against out-of-plane stability of the masonry wall.

(ii) R. C. Beams

The shear-flexure mode of failure is most commonly observed during the earthquakes, which is described as below:

Mode – 5: Shear-flexure failure

Mechanism: Two types of plastic hinges may form in the beams of multi-storied framed construction depending upon the span of beams. In case of short beams or where gravity load supported by the beam is low, plastic hinges are formed at the column ends and damage occurs in the form of opening of a crack at the end of beam otherwise there is formation of plastic hinges at and near end region of beam in the form of diagonal shear cracking.

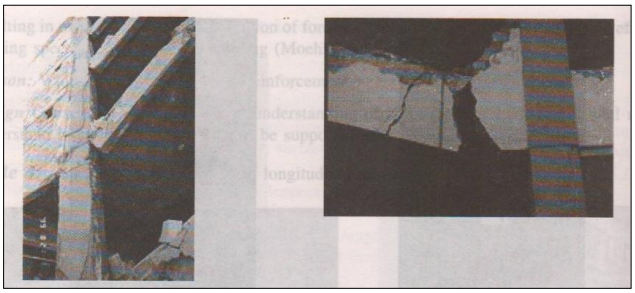


Fig – 11.43 Shear-flexure failure

Reasons: Lack of longitudinal compressive reinforcement, infrequent transverse reinforcement in plastic hinge zone, bad anchorage of the bottom reinforcement in to the support or dip of the longitudinal beam reinforcement, bottom steel termination at face of column.

Remedies: Adequate flexural and shear strength must be provided and verification by design calculation is essential. The beams should not be too stiff with respect to adjacent columns so that the plastic hinging will occur in beam rather than in column. To ensure that the plastic hinges zones in beams have adequate ductility, the following considerations must be considered.

- Lower and upper limits on the amount of longitudinal flexural tension steel
- A limit on the ration of the steel on one side of the beam to that of on the other side
- Minimum requirements for the spacing and size of stirrups to restrain buckling of the longitudinal reinforcement

(iii) R. C. Beam-Column Joints

The most common modes of failure in beam-column joint are as follows.

Mode – 6: shear failure in beam-column joint



Fig - 11.44 Shear failure in beam-column joint

Mechanism: The most common failure observed in exterior joints are due to either high shear or bond (anchorage) under severe earthquakes. Plastic hinges are formed in the beams at the column faces. As a result, cracks develop throughout the overall beam depth. Bond deterioration near the face of the column causes propagation of beam reinforcement yielding in the joint and a shortening of the bar length available for force transfer by bond causing horizontal bar slippage in the joint. In the interior joint, the beam reinforcement at both the column faces undergoes different stress conditions (compression and tension) because of opposite sights of seismic bending moments results in failure of joint core.
Reasons: Inadequate anchorage of flexural steel in beams, lack of transverse reinforcement.

Remedies: Exterior Joint – The provision on anchorage stub for the beam reinforcement improves the performance of external joints by preventing spalling of concrete cover on the outside face resulting in loss of flexural strength of the column. This increases diagonal strut action as well as reduces steel congestion as the beam bars can be anchored clear of the column bars.

(iv) R. C. Slab

Generally slab on beams performed well during earthquakes and are not dangerous but cracks in slab creates serious aesthetic and functional problems. It reduces the available strength, stiffness and energy dissipation capacity of building for future earthquake. In flat slab construction, punching shear is the primary cause of failure. The common modes of failure are:

Mode – 7: Shear cracking in slabs

Mechanism: Damage to slab oftenly occurs due to irregularities such as large openings at concentration of earthquake forces, close to widely spaced shear walls, at the staircase flight landings.

Reasons: Existing micro cracks which widen due to shaking, differential settlement.

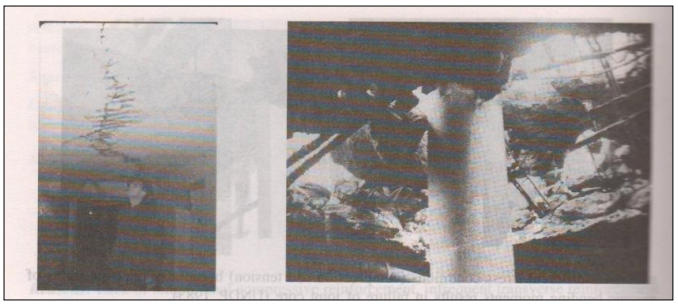


Fig – 11.45 Shear cracking in slabs

Remedies:

- Use secondary reinforcement in the bottom of the slab
- Avoid the use of flat slab in high seismic zones, provided this is done in conjunction with a stiff lateral load resisting system

(v) R. C. Shear Walls

Shear walls generally performed well during the earthquakes. Four types of failure modes are generally observed:

Mode – 8: Four types of failure modes are generally observed

(i) Diagonal tension-compression failure in the form of cross-shaped shear cracking
(ii) Sliding shear failure cracking at interface of new and old concrete
(iii) Flexure and compression in bottom end region of wall and finally
(iv) Diagonal tension in the form of X shaped cracking in coupling beams

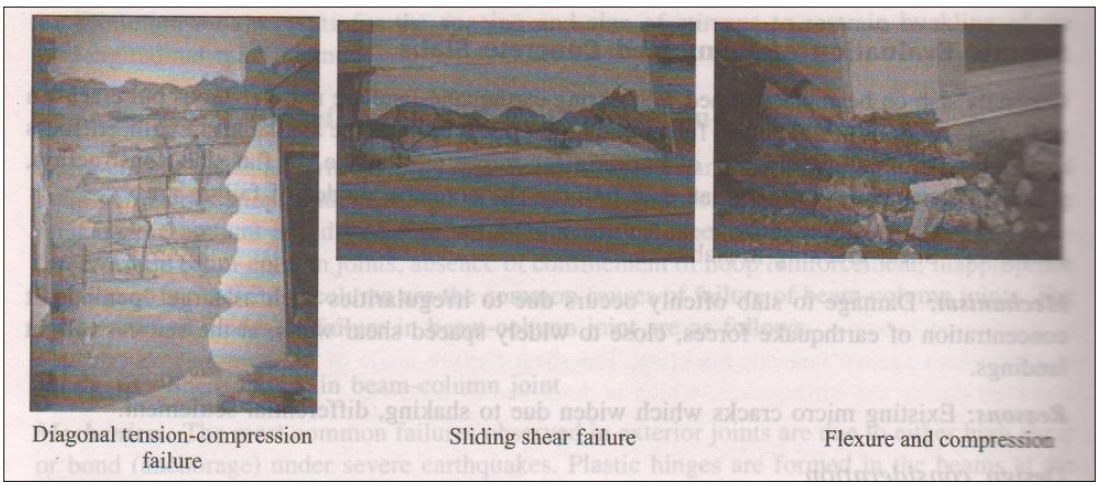


Fig – 11.46 Diagonal tension-compression/ Sliding shear/ Flexure and compression failure

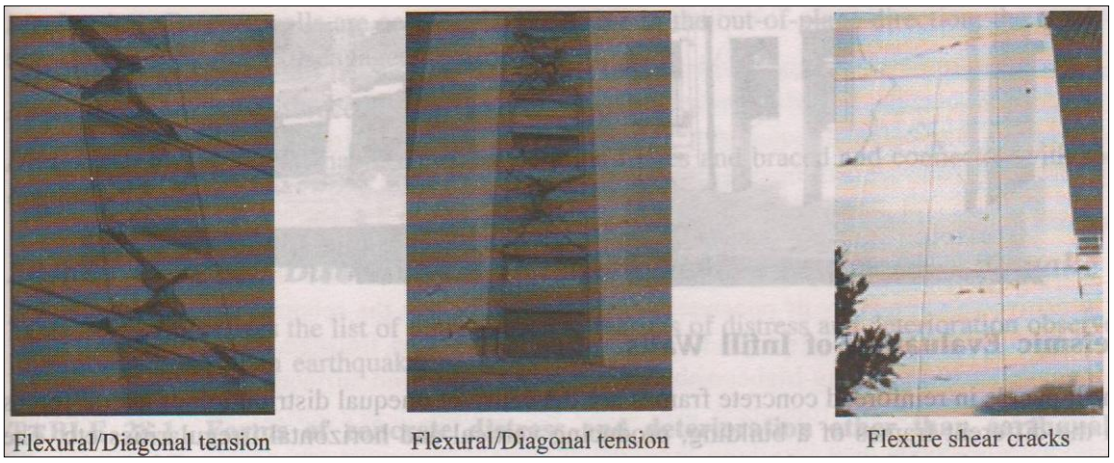


Fig – 11.47 Diagonal tension in the form of X shaped

Mechanism: Shear walls are subjected to shear and flexural deformation depending upon the slenderness ratio. Therefore, the damage in shear walls may generally occurs due to inadequate shear and flexure capacity of wall. Slender walls are governed by their flexural strength and cracking occurs in the form of yielding of main flexure reinforcement in the plastic hinge region, normally at the base of the wall. Squat walls are governed by their shear strength and failure takes place due to diagonal tension or diagonal compression in the form of inclined cracking. Coupling beams between shear walls or piers may also damage due to inadequate shear and flexure capacity. Sometimes damage occurs at the construction joints in the form of slippage and related drift.

Reasons:

- Flexural/boundary compression failure: Inadequate transverse confining reinforcement to the main flexural reinforcement near the outer edge of wall in boundary elements
- Flexure/diagonal tension: Inadequate horizontal shear reinforcement
- Sliding shear: Absence of diagonal reinforcement across the potential sliding planes of the plastic hinge zone
- Coupling beams: Inadequate stirrup reinforcement and no diagonal reinforcement
- Construction joint: Improper bonding between two surfaces

Remedies:

- The concrete shear walls must have boundary elements or columns thicker than walls, which will carry the vertical load after shear failure of wall.
- A proper connection between wall versus diaphragm as well as wall versus foundation to complete the load path.
- Proper bonding at construction joint in the form of shear friction reinforcement.
- Provision of diagonal steel in the coupling beam.

(v) Infill Walls

Infill panels in reinforced concrete frames are the cause of unequal distribution of lateral forces in the different frames of a building, producing vertical and horizontal irregularities etc. the common mode of failure of infill masonry are in plane or shear failure.

Mode – 9: Shear failure of masonry infill

Mechanism: Frame with infill possesses much more lateral stiffness than the bare frame, and hence initially attracts most of the lateral force during an earthquake. Being brittle, the infill starts to disintegrate as soon as its strength is reached. Infills that were not adequately tied to the surrounding frames, sometimes dislodges by out-of-plane seismic excitations.

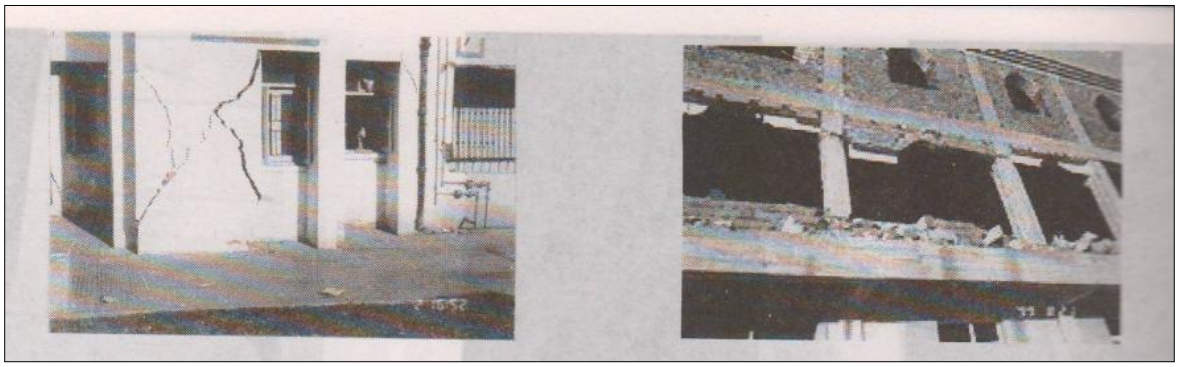


Fig – 11.48 Shear failure of masonry infill

Reasons: Infill causes asymmetry of load application, resulting in increased torsional forces and changes in the distribution of shear forces between lateral load resisting system.

Remedies: Two strategies are possible either complete separation between infill walls and frame by providing separation joint so that the two systems do not interact or complete anchoring between frame and infill to act as an integral unit. Horizontal and vertical reinforcement may also be used to improve the strength, stiffness, and deformability of masonry infill walls.

(vi) Parapets

Un-reinforced concrete parapets with large height-to-thickness ratio and not in proper anchoring to the roof diaphragm may also constitute a hazard. The hazard posed by a parapet increases in direct proportion to its height above building base, which has been generally observed.

The common mode of failure of parapet wall is against out-of-plane forces, which is described as follows:

Mode – 10: Brittle flexure out-of-plane failure

Mechanism: Parapet walls are acceleration sensitive in the out-of-plane direction; the result is that they may become disengaged and topple.

Reasons: Not properly braced.

Remedies: Analysed for acceleration forces and braced and connected with roof diaphragm.

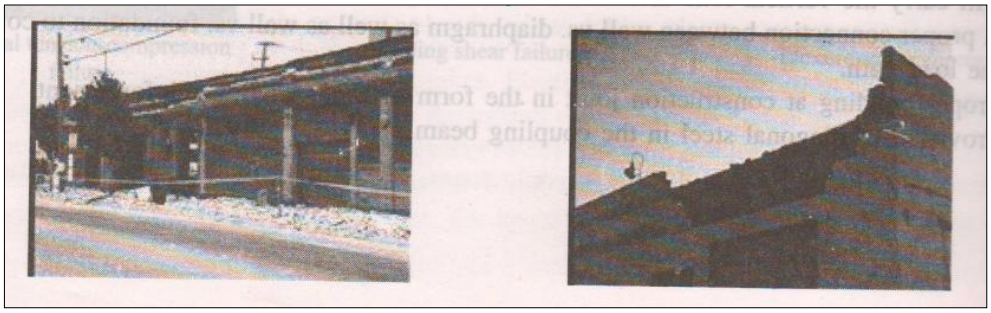


Fig – 11.49 Brittle flexure out-of-plane failure

11.4 चिनाई संरचनाओं की रेट्रोफ़िटिंग / Retrofitting of Masonry Structures

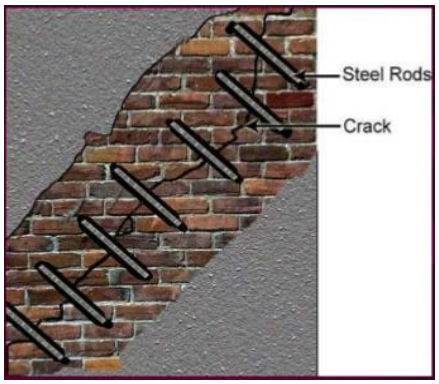
(a) Principle of Seismic Safety of Masonry Buildings

- Integral box action
- Integrity of various components
  - Roof to wall
  - Wall to wall at corners
  - Wall to foundation
- Limit on openings

(b) Methods for Retrofitting of Masonry Buildings

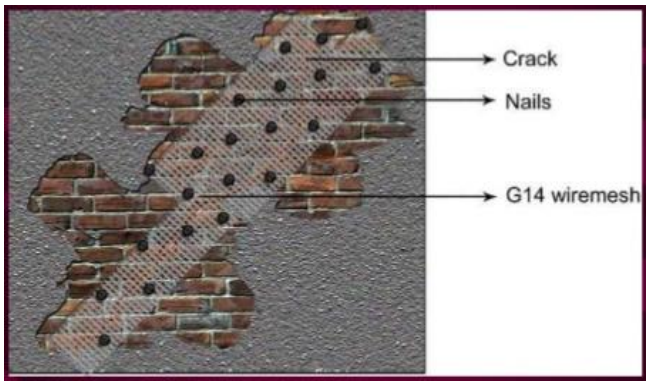
Repairing (Improving existing masonry strength)

- Stitching of cracks
- Grouting with cement or epoxy
- Use of CFRP (Carbon Fibre Reinforced Polymer) strips



(a)

Fig – 11.50 (a) Stitching of cracks



(b)

Fig – 11.50 (b) Repair of damaged member in masonry walls

(c) Retrofitting of Earthquake vulnerable buildings

- External binding or jacketing
- Shotcreting
- Strengthening of wall intersections
- Strengthening by cross wall
- Strengthening by buttresses
- Strengthening of arches

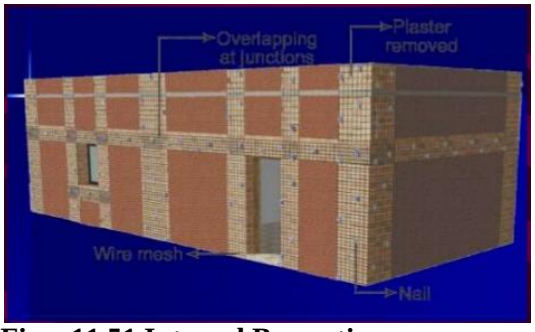
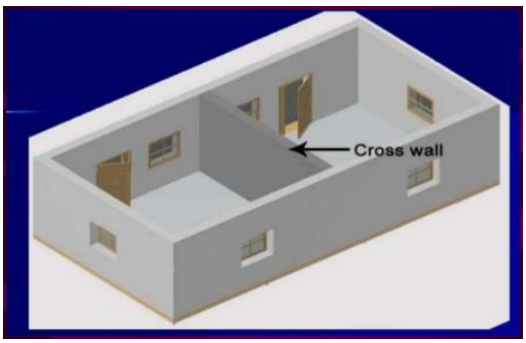


Fig – 11.51 Integral Box action



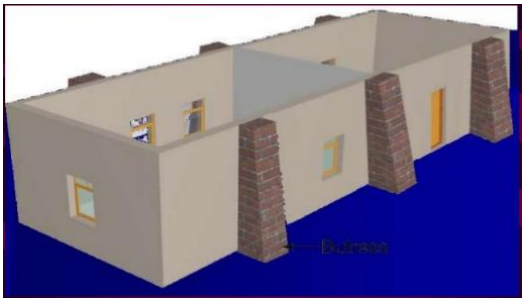
(a)

Fig - 11.52 (a) Strengthening of Wall intersections



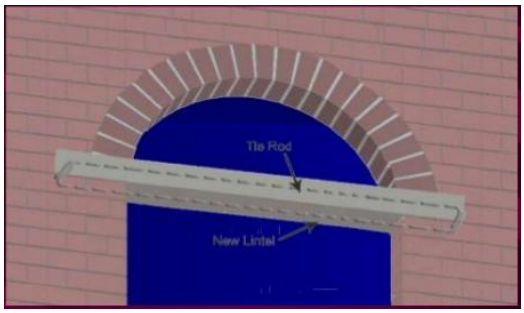
(b)

Fig - 11.52 (b) Strengthening by cross wall



(a)

Fig – 11.53 (a) Strengthening by Buttresses



(b)

Fig – 11.53 (b) Strengthening of Arches

***

परिशिष्ट / Annexure – I

भारतीय भूकंपी संहिताएँ / Indian Seismic Codes

Development of building codes in India started rather early. Today, India has a fairly good range of seismic codes covering a variety of structures, ranging from mud or low strength masonry houses to modern buildings. However, the key to ensuring earthquake safety lies in having a robust mechanism that enforces and implements these design code provisions in actual constructions.

➢ भूकंपी डिजाइन कोड का महत्व / Importance of Seismic Design Codes

Ground vibrations during earthquakes cause forces and deformations in structures. Structures need to be designed to withstand such forces and deformations. Seismic codes help to improve the behaviour of structures so that they may withstand the earthquake effects without significant loss of life and property. An earthquake-resistant building has four *virtues* in it, namely:

(a) *Good Structural Configuration*: Its size, shape and structural system carrying loads are such that they ensure a direct and smooth flow of inertia forces to the ground.
(b) *Lateral Strength*: The maximum lateral (horizontal) force that it can resist is such that the damage induced in it does not result in collapse.
(c) *Adequate Stiffness*: Its lateral load resisting system is such that the earthquake-induced deformations in it do not damage its contents under low-to moderate shaking.
(d) *Good Ductility*: Its capacity to undergo large deformations under severe earthquake shaking even after yielding, is improved by favourable design and detailing strategies.

Seismic codes cover all these aspects.

➢ भारतीय भूकंपी संहिताएँ / Indian Seismic Codes

Seismic codes are unique to a particular region or country. They take into account the local seismology, accepted level of seismic risk, building typologies, and materials and methods used in construction. The first formal seismic code in India, namely IS 1893, was published in 1962. Today, the Bureau of Indian Standards (BIS) has the following seismic codes:

1. IS 1893 (Part I) : 2002, *Indian Standard Criteria for Earthquake Resistant Design of Structures* (5 Revision)
2. IS 4326 : 1993, *Indian Standard Code of Practice for Earthquake Resistant Design and Construction of Buildings* (2nd Revision)
3. IS 13827 : 1993, *Indian Standard Guidelines for Improving Earthquake Resistance of Earthen Buildings*
4. IS 13828 : 1993, *Indian Standard Guidelines for Improving Earthquake Resistance of Low Strength Masonry Buildings*
5. IS 13920 : 1993, *Indian Standard Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces*

6. IS 13935 : 1993, *Indian Standard Guidelines for Repair and Seismic Strengthening of Buildings*

The regulations in these standards do not ensure that structures suffer *no damage* during earthquake of *all magnitudes*. But, to the extent possible, they ensure that structures are able to respond to earthquake shakings of *moderate intensities* without *structural damage* and of *heavy intensities* without *total collapse*.

- **IS 1893 (Part I) : 2002**

IS 1893 is the main code that provides the seismic zone map and specifies seismic design force. This force depends on the mass and seismic coefficient of the structure; the latter in turn depends on properties like seismic zone in which structure lies, importance of the structure, its stiffness, the soil on which it rests, and its ductility. For example, a building in Bhuj will have 2.25 times the seismic design force of an identical building in Bombay. Similarly, the seismic coefficient for a single-storey building may have 2.5 times that of a 15-storey building.

The revised 2002 edition, Part 1 of IS1893, contains provisions that are general in nature and those applicable for buildings. The other four parts of IS 1893 will cover:

a) Liquid-Retaining Tanks, both elevated and ground supported (*Part 2*)
b) Bridges and Retaining Walls (*Part 3*)
c) Industrial Structures including Stack Like Structures (*Part 4*) and
d) Dams and Embankments (P*art 5*)

These four documents are under preparation. In contrast, the 1984 edition of IS1893 had provisions for all the above structures in a single document.

Provisions for Bridges

Seismic design of bridges in India is covered in three codes, namely *IS 1893 (1984)* from the BIS, *IRC 6 (2000)* from the Indian Roads Congress, and *Bridge Rules (1964)* from the Ministry of Railways. All highway bridges are required to comply with IRC 6, and all railway bridges with Bridge Rules. These three codes are conceptually the same, even though there are some differences in their implementation. After the 2001 Bhuj earthquake, in 2002, the IRC released interim provisions that make significant improvements to the IRC6 (2000) seismic provisions.

- **IS 4326 : 1993 (Reaffirmed 2003)**

This code covers general principles for earthquake resistant buildings. Selection of materials and *special features* of design and construction are dealt with for the following types of buildings: timber constructions, masonry constructions using rectangular masonry units, and buildings with prefabricated reinforced concrete roofing/flooring elements. The code incorporates Amendment No. 3 (January 2005).

- **IS 13827 : 1993 and IS 13828 : 1993**

Guidelines in IS 13827 deal with empirical design and construction aspects for *improving* earthquake resistance of *earthen houses*, and those in IS 13828 with general principles of design and special construction features for *improving* earthquake resistance of buildings of *low-strength masonry*. This Masonry includes burnt clay brick or stone masonry in weak mortars, like clay-mud. These standards are applicable in seismic zones III, IV and V. Constructions based on them are termed non-engineered, and are not totally free from collapse under seismic shaking intensities VIII (MMI) and higher. Inclusion of features mentioned in these guidelines may only enhance the seismic resistance and reduce chances of collapse.

- **IS 13920 : 1993 (Reaffirmed 2003)**

In India, reinforced concrete structures are designed and detailed as per the Indian Code IS 456 (2002). However, structures located in *high seismic regions* require ductile design and detailing. Provisions for the ductile detailing of *monolithic* reinforced concrete frame and shear wall structures are specified in IS 13920 (1993). After the 2001 Bhuj earthquake, this code has been made mandatory for all structures in zones III, IV and V. Similar provisions for seismic design and ductile detailing of steel structures are not yet available in the Indian codes.

- **IS 13935 : 1993**

These guidelines cover general principles of seismic strengthening, selection of materials, and techniques for repair/seismic strengthening of masonry and wooden buildings. The code provides a brief coverage for *individual reinforced concrete members* in such buildings, but does not cover *reinforced concrete frame* or *shear wall buildings* as a whole. Some guidelines are also laid down for *non-structural* and *architectural components* of buildings.

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परिशिष्ट / Annexure – II

Checklist/ Multiple Choice Questions for Points to be kept in mind during Construction of Earthquake Resistant Building

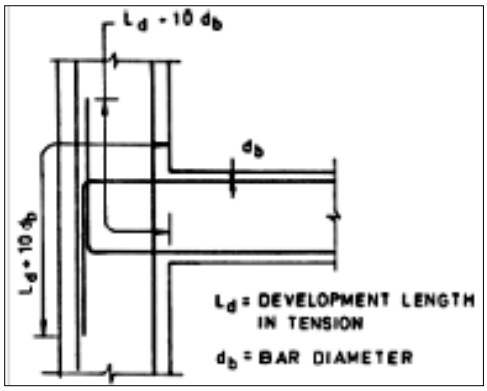
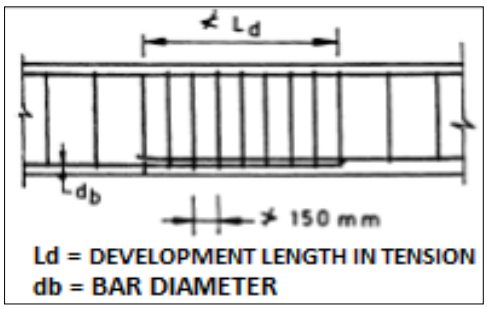
S. No.	Description	Observer Remarks
1.	Seismic Zone in which building is located. i) Zone II – Least Seismically Prone Region ii) Zone III – iii) Zone IV – iv) Zone V – Most Seismically Prone Region	Choose Zone
2.	Environment condition to which building is exposed. a) Mild b) Moderate c) Severe d) Very Severe e) Extreme	Choose Condition
3.	Whether the building is located in Flood Zone	Yes/No.
4.	Whether the building is located in Land Slide Zone i.e. building is on hill slope or Plane Area	Yes/No.
5.	Type of soil at founding level a) Rock or Hard Soil b) Medium Soil c) Soft Soil	Choose type of soil
6.	Type of Building : I) Load Bearing Masonry Building a) Brick Masonry Construction b) Stone Masonry construction II) R.C.C. Framed Structure a) Regular frame b) Regular Frame with shear wall c) Irregular Frame d) Irregular Frame with shear wall e) Soft Story Building	Choose type of building
7.	No. of Story above Ground Level with provision of Future Extension	Mention Storey
8.	Category of Building considering Seismic Zone and Importance Factor (As per Table – 10.2) i) Category B – Building in Seismic Zone II with Importance Factor 1.0 ii) Category E- Building in Seismic Zone II with Importance Factor 1.0 and 1.50	Choose category
9.	Bricks should not have compressive strength less than 3.50 MPa	Yes/No.
10.	Minimum wall thickness of brick masonry i) 1 Brick – Single Storey Construction ii) 1 ½ Brick – In bottom storey up to 3 storey construction & 1 Brick in top storey with brick masonry.	Choose appropriate
11.	Height of building is restricted to : i) For A, B & C categories – G+2 with flat roof G+1 plus anti for pitched roof, when height of each story not exceed 3 m ii) D category – G+1 with flat Roof - Ground plus attic for pitched roof	Choose appropriate
12.	Max. Height of Brick masonry Building – 15 m (max. 4 storey)	Yes/No.
13.	Mortar mix shall be as per Table – 10.2 for category A to E	Choose Mortar

14.	Height of Stone Masonry wall i) For Categories A&B – a) When built in Lime-Sand or Mud mortar – Two storey with flat roof or One Storey plus attic b) When build in cement /sand 1:6 mortar - One story higher ii) For Categories C&D – a) When built in cement Sand 1:6 Mortar - Two storey with flat roof or One Storey plus attic for pitched roof b) When build in lime /sand or Mud mortar - One story with flat roof or One Story plus attic	Choose appropriate
15.	Through stone at full length equal to wall thickness in every 600 mm lift at not more than 1.20 m apart horizontally has been provided.	Yes/No.
16.	Through stone and Bond Element as per Fig. 10.24 has been provided	Yes/No.
17.	Horizontal Bands: a) Plinth Band b) Lintel Band c) Roof Bond d) Gable Bond For Over Strengthening Arrangement for Category D & E Building have been provided.	Yes/No.
18.	Bond shall be made up of Reinforced Concrete of Grade not leaner than M15 or Reinforced brick work in cement mortar not leaner than 1:3.	Yes/No.
19.	Bond shall be of full width of wall not less than 75 mm in depth and reinforced with steel as shown in Table – 10.6	Yes/No.
20.	Vertical steel at corners & junction of wall, which are up to 340 mm (1 ½ brick) thick shall be provided as shown in Table – 10.1	Yes/No.
21.	General principal for planning building are : i) Building should be as light as possible. ii) All parts of building should be tied together to act as one unit. iii) Projecting part should be avoided. iv) Building having plans with shape L, T, E and Y shall preferably be separated in to rectangular parts. v) Structure not to be founded on loose soil, which will subside or liquefy during Earthquake resulting in large differential settlement. vi) Heavy roofing material should be avoided. vii) Large stair hall shall be separated from Rest of the Building by means of separation or crumple section. viii) All of the above ix) None of the above	Choose Correct
22.	Structural irregularities may be : i) Horizontal Irregularities ii) Vertical Irregularities iii) All of the above iv) None of the above	Choose Correct

23.	Horizontal Irregularities are i) Asymmetrical plan shape (e.g. L,T,U,F) ii) Horizontal resisting elements (diaphragms) iii) All of the above iv) None of the above	Choose Correct
24.	Horizontal Irregularities result in i) Torsion ii) Diaphragm deformation iii) Stress Concentration iv) All of the above v) None of the above	Choose Correct
25.	Vertical Irregularities are i) Sudden change of stiffness over height of building. ii) Sudden change of strength over height of building. iii) Sudden change of geometry over height of building. iv) Sudden change of mass over height of building. v) All of the above vi) None of the above	Choose Correct
26.	Soft story in one : i) Which has lateral stiffness < 70% of story above ii) Which has lateral stiffness < 80% of average lateral stiffness of 3 storeys above. iii)All of the above vi) None of the above	Choose Correct
27.	Extreme soft storey in one: i) Which has lateral stiffness < 60% of storey above ii) Which has lateral stiffness < 70% of average lateral stiffness of 3 storeys above. iii)All of the above iv)None of the above	Choose Correct
28.	Weak Storey is one : i) Which has lateral strength < 80% of storey above ii) Which has lateral strength < 80% of storey above iii)All of the above iv)None of the above	Choose Correct
29.	Natural Period of Building: It is the time taken by the building to undergo one complete cycle of oscillation during shaking.	True/ False
30.	Fundamental Natural Period of Building: Natural period with smallest Natural Frequency i.e. with largest natural period is called Fundamental Natural Period.	True/ False
31.	Type of building frame system: i) Ordinary RC Moment Resisting Frame (OMRF) ii) Special RC Moment Resisting Frame (SMRF) iii) Ordinary Shear Wall with OMRF iv) Ordinary Shear Wall with SMRF v) Ductile Shear wall with OMRF vi) Ductile Shear wall with SMRF vii) All of the above	Choose Correct

32.	Zone factor to be considered for : i) Zone II – 0.10 ii) Zone III – 0.16 iii) Zone IV – 0.24 iv) Zone V – 0.36	True/ False
33.	Importance Factor i) Important building like school, hospital, railway station 1.5 ii) All other buildings 1.0	True/ False
34.	Design of Earthquake effect is termed as : i) Earthquake Proof Design or ii) Earthquake Resistant Design	Choose Correct
35.	Seismic Analysis is carried out by : i) Dynamic analysis procedure [Clause 7.8 of IS1893 (Part I): 2002] ii) Simplified method referred as Lateral Force Procedure [Clause 7.5 of IS 1893 (Part I) :2002]	True/ False
36.	Dynamic Analysis is performed for following buildings: (a) Regular Building : > 40 m height in Zone IV & V > 90 height in Zone II & III (b) Irregular Building: > 12 m, all framed building in Zone IV & V > 40 m all framed building in Zone II and III	True/ False
37.	Base Shear for Lateral Force Procedure is $V_B = A_h W = \frac{Z}{2} \frac{I}{R} \frac{Sa}{g}$	True/ False
38.	Distribution of Base Shear to different Floor level is $Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2}$	True/ False
39.	Concept of capacity design is to: Ensure that brittle element will remain elastic at all loads prior to failure of ductile element.	True/ False
40.	'Strong Column – Weak Beam' Philosophy is: For a building to remain safe during Earthquake shacking, columns should be stronger than beams and foundation should be stronger than columns	True/ False
41.	Rigid Diaphragm Action is: Geometric distortion of Slab in horizontal plane under influence of horizontal Earthquake force is negligible. This behaviour is known as Rigid Diaphragm Action.	True/ False
42.	Soft storied buildings are : Column on Ground Storey do not have infill walls (of either masonry or RC)	True/ False
43.	Soft Storey or Open Ground Story is also termed as weak storey.	True/ False
44.	Short columns in building suffer significant damage during an earth-quake	True/ False

45.	Building can be protected from damage due to Earthquake effect by using : a) Base Isolation Devices b) Seismic Dampers	True/ False
46.	Idea behind Base Isolation is: To detach building from Ground so that E/Q motion are not transmitted through the building or at least greatly reduced.	True/ False
47.	Base Isolation is done through: Flexible Pads connected to building and foundation.	True/ False
48.	Seismic Dampers are: (i) Special devices to absorb the energy provided by Ground Motion to the building. (ii) They act like hydraulic shock absorber in cars.	True/ False
49.	Commonly used Seismic Dampers are: (i) Viscous Dampers (ii) Friction Dampers (iii) Yielding Dampers	True/ False
50.	For Ductility Requirement: (i) Min. Grade of Concrete shall be M20 for all buildings having more than 3 storeys in height (ii) Steel Reinforcement of Grade Fe 415 or less only shall be used (iii) Grade Fe 500 & Fe 550 having elongation more than 14.5% may be used	True/ False
51.	For Ductility Requirement, Flexure Members shall satisfy the following requirement: (i) width of member shall not be less than 200 mm (ii) width to depth ratio > 0.3 (iii) depth of member D < $1/4^{th}$ of clear span (iv) Factored Axial Stress on the member under Earthquake loading shall not be greater than 0.1 f_{ck}	True/ False
52.	For Ductility Requirement, Longitudinal reinforcement in Flexure Member shall satisfy the following requirements: i) Top and bottom reinforcement consist of at least 2 bars throughout member length. ii) Tensile Steel Ratio on any face at any section shall not be less than $\rho_{min} = (0.24 \sqrt{f_{ck}}) / f_y$ iii) Max. Steel ratio on any face at any section shall not exceed $\rho_{max} = 0.025$ iv) + ve steel at Joint face must be at least equal to half the –ve steel at that face. v) Steel provided at each of the top & bottom face of the member at any section along its length shall be at least equal to $1/4^{th}$ of max. –ve moment steel provided at the face of either joint.	True/ False

	(vi) Detailing of Reinforcement at Beam-Column Joint  (vii) Detailing of Splicing  	
53.	For Ductile Requirement in compression member. i) Minimum diversion of member shall not be less than 200 mm ii) In Frames with beams c/c Span > 5m or unsupported length of column > 4 m, shortest dimension shall not be less than 300 mm iii) Ratio of shortest cross sectional dimension to the perpendicular dimension shall probably not less than 0.4	True/ False
54.	For Ductile Requirement, Longitudinal reinforcement in compression member shall satisfy the following requirements: i) Lap splice shall be provided only in the central half of the member length proportional as tension splice. ii) Hoop shall be provided over entire splice length at spacing not greater than 150 mm iii) Not more than 50% bar shall be spliced at one section.	True/ False
55.	When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat.	True/ False

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टिप्पणी / NOTES

टिप्पणी / NOTES

हमारा उद्देश्य

अनुरक्षण प्रौधौगिकी और कार्यप्रणाली को उन्न्यन करना तथा उत्पादकता और रेलवे की परिसम्पत्तियों एवं जनशक्ति के निष्पादन में सुधार करना जिससे अंतर्विषयों में विश्वसनीयता, उपयोगिता और दक्षता प्राप्त की जा सके।

Our Objective

To upgrade Maintenance Technologies and Methodologies and achieve improvement in productivity and performance of all Railway assets and manpower which inter-alia would cover Reliability, Availability, and Utilisation.



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