MALLA REDDY ENGINEERING COLLEGE
(AUTONOMOUS)

DEPARTMENT OF CIVIL ENGINEERING

M.TECH. STRUCTURAL ENGINEERING

<u>COURSE MATERIAL</u>

SUBJECT CODE: A1118

SUBJECT NAME: EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

A1118

EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

COURSE MATERIAL

Introduction to Earthquake Engineering

In this lecture the earthquake engineering has been briefly discussed.

1.1 Introduction

The effects of earthquakes on people and their environment and methods of reducing those effects are included in the earthquake engineering. Earthquake engineering is a very broad field drawing on aspects of

- Geology
- seismology
- geotechnical engineering
- structural engineering
- risk analysis
- other technical fields

Its practice also requires consideration of

- social factor
- economic factor
- political factor

Terminology

Earthquakes: Earthquakes are defined as, 'Ground shaking and radiated seismic energy caused mostly by sudden slip on a fault, volcanic or any sudden stress change in the earth'.

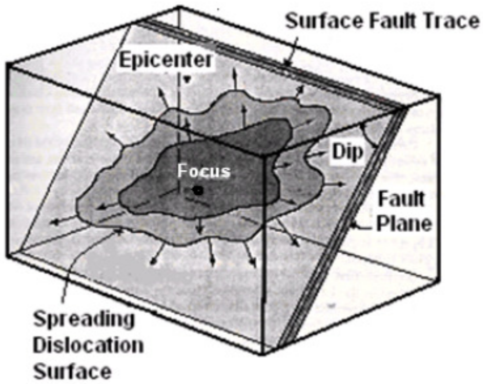


Figure 1.1 Origin of an earthquake

Seismology: The science dealing with the study of earthquakes in all their aspects is called Seismology. It is an interdisciplinary science which is partly geology and partly physics. The word seismic is commonly used to qualify anything related to an earthquake, such as seismic waves, seismic intensity, seismic zoning, seismic region and so on.

Epicenter: The point or place on the surface vertically above the focus of a particular earthquake is termed as its epicentre as shown in Figure 1.1. It is that (geographical) place on the surface of the earth where the vibration from a particular earthquake reaches first of all. It is often the location of maximum damage in that event.

Hypocenter or Focus: An earthquake is generally due to some disturbance or displacement in the rocks at some depth below the surface of the Earth. Shock waves originate from that place or point of disturbance and then travel in all directions causing the vibrations. The place or point of origin of an earthquake below the surface of the Earth is termed as its focus (or hypocenter). In modern seismology, focus signifies a zone rather than a point of origin. It may lie from a few hundred meters to hundreds of kilometres below the surface.

Magnitude: It is the term expressing the rating of an earthquake on the basis of amplitude of seismic waves recorded as seismograms. The method (of determining rating of an earthquake) was first used by Charles F Richter in 1935 who developed a scale of magnitude for local use on the basis of study of records of earthquakes of California, USA. Subsequently that scale was improved upon and is presently used internationally for describing the size of an earthquake. In precise terms and as understood today, the Richter Magnitude is the logarithm to the base of 10 of the maximum seismic wave amplitude recorded on a seismograph at a distance of 100 km from the epicentre of a particular earthquake.

Intensity: It is the rating of the effects of an earthquake at a particular place based on the observations of the affected areas, using a descriptive scale like Modified Mercalli Scale.

Occurrence of earthquakes in the World:

All places on the earth are not equally seismic. Earthquakes are generally found to occur along specific regions called 'Seismic Belts'. There are three main belts around the globe along which majority of earthquakes have occurred. They are:
1. Circum Pacific Belt or Ring of Fire;
2. Alpide Belt and
3. North and South in the Middle of the Atlantic Ocean.

Figure 1.1 shows the earthquake losses by countries around the world. From the Figure 1.1, it is seen that the losses in terms of death and money can be very enormous.

Occurrence of earthquakes in India:

In India, the main seismic zone runs along Himalayan mountain range, northeast India, Andaman-Nicobar islands and Rann of Kutch region. Fig 1.2 shows the earthquake distribution in and around India and Fig 1,3 shows Devastation during Bhuj earthquake in 2001. Table 1.1 shows some significant earthquakes in India.

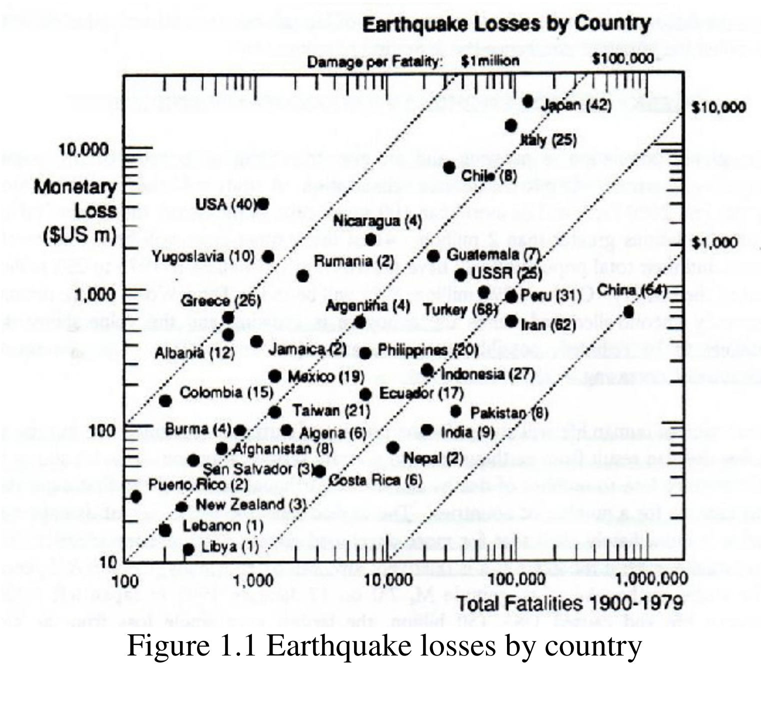


Figure 1.1 Earthquake losses by country

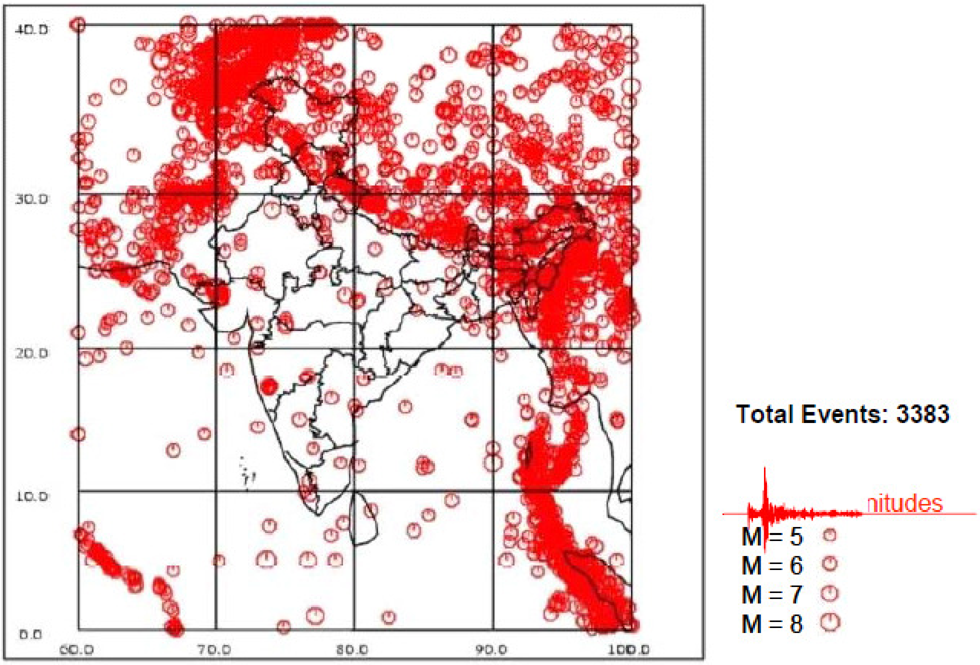


Figure 1.2 Plot of Earthquakes (M>= 5.0) From IMD Catalogue for the period from 1800 to Sept, 2001

Figure 1.3 Bhuj earthquake in India 2001

Table 1.1 Significant Historical Earthquakes in India

Date	Location	Magnitude	Deaths	Comments
893 B.C	India		180,000	Widespread damage; many killed in collapse of earthen homes
1819	Kutch Gujrat	8.0	2000	Kutch earthquake, first well

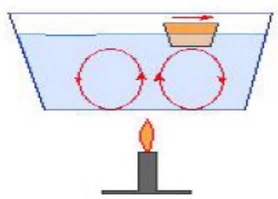
				documented observations of faulting
1897	Shillong Plateau	8.7	1542	
1905	Kangra, Himachal Pradesh	8.0	20000	
1934	Bihar-Nepal Border	8.3	1000	9000 injured
1950	Assam	8.5	532	
1988	Indo-Nepal border	6.5	1000	
1991	Uttarkashi, Uttar Pradesh	6.6	760	5000 injured
1993	Latur-Osmanabad, Maharastra	6.3	7601	15846 injured
1997	Jabalpur, Madhya Pradesh	6.0	55	500 injured
1999	Chamoli district, Uttar Pradesh	6.8	1000	400 injured
2001	Bhuj, Gujrat	7.9	19727	166000 injured

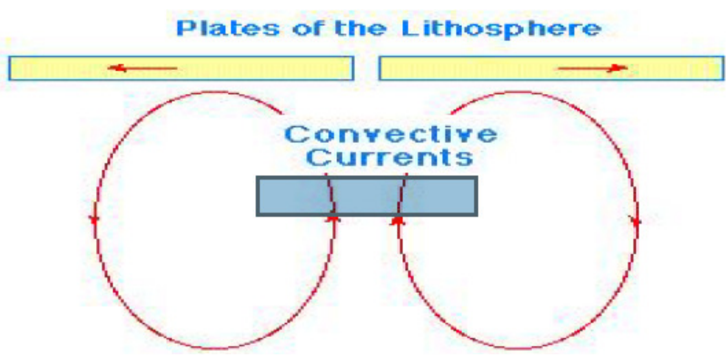
Plate Tectonics

The theory of plate tectonics was originally proposed in 1912 by a German scientist, A. Wegner. Plate Tectonics is the theory supported by a wide range of evidence that considers the earth's crust and upper mantle to be composed of several large, thin, relatively rigid plates that move relative to one another. It is based on some theoretical assumptions that explain the forces, which cause accumulation of stresses inside the earth (Fig 1.4).

These assumptions are as given below:

• Drifting of continents and mountain building process
• Shortening of Earth’s crust due to cooling and contraction.
• Disturbance of mass distribution on the Earth’s surface as a result of erosion of high lands and deposition of sediment in the sea.
• Generation of heat by radioactive material inside the Earth’s crust.





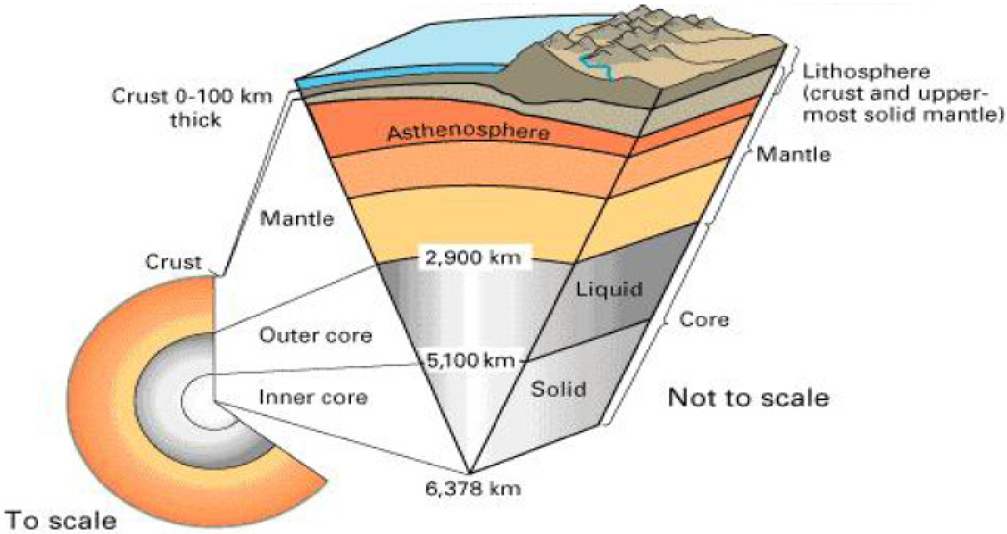


Figure 1.4: Theory of plate tectonics

Plate Boundaries

The edges of the oceanic and/or continental plate boundaries mark the regions of destructive earthquake activity and volcanic activity. Fig 1.5 shows the plate boundaries across the globe.

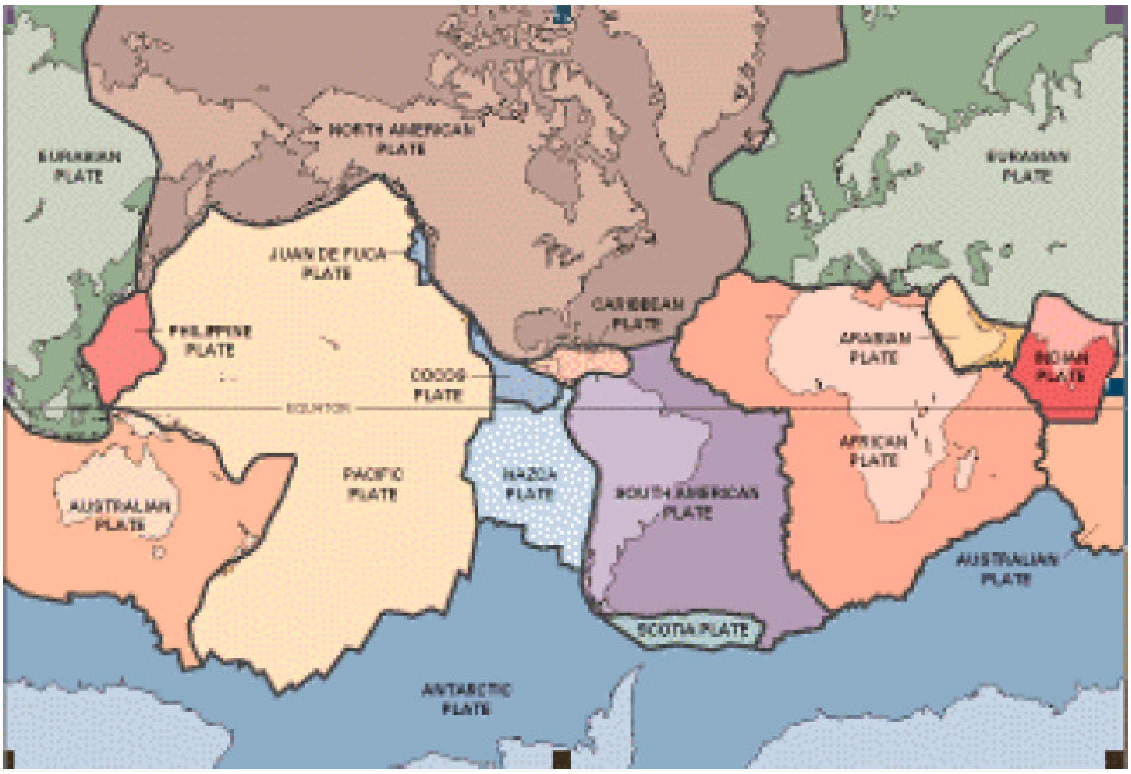


Figure 1.5 Plate Boundaries

About 80% of the seismic energy is released by earthquakes occurring along the plate boundaries. These earthquakes are called as inter-plate earthquake, directly associated with forces related to the interaction of the plates. (Circum-Pacific belt, Mid-Atlantic ridge and Alpine- Himalayan belt) Sporadically, earthquakes also occur at rather large distances from the respective plate margins, these so called intra-plate earthquake, show a diffuse geographical distribution. (Central USA: New Madrid, 1812, Northeastern Continental China: Tangshan, 1976 and central India: Latur, 1993).

Types of Plate Boundaries

Earthquakes are usually caused when the underground rocks suddenly break along a plane of weakness, called fault. There are three types of plate boundaries as explained in the Fig 1.6 to 1.8 given below

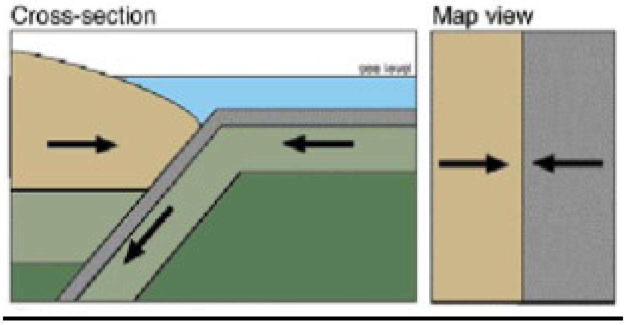


Figure 1.6 a Convergent plate boundaries

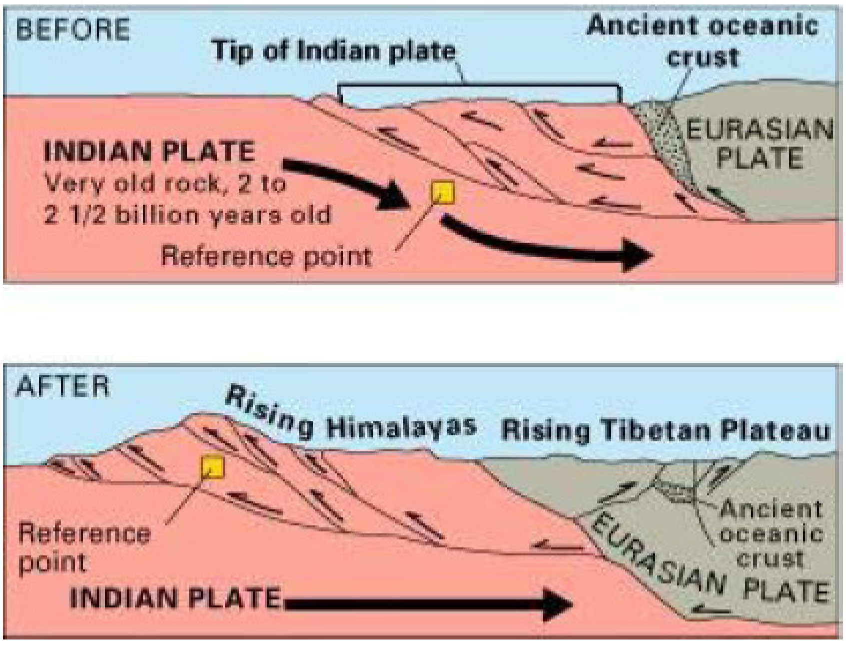


Figure 1.6 b Examples of convergent plate boundaries

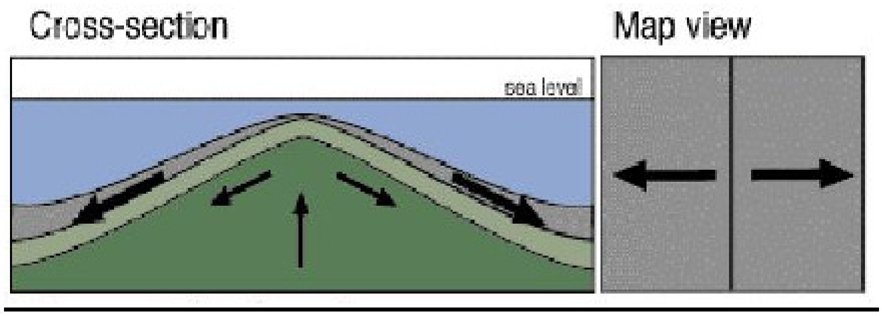


Figure 1.7 Divergent plate boundaries

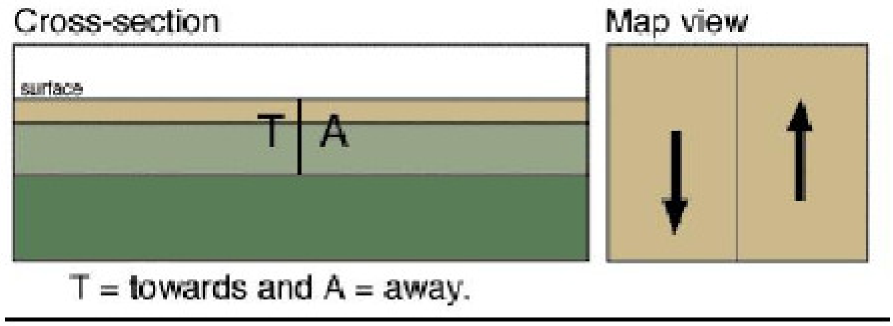


Figure 1.8 Transform plate boundaries

Faults

A fault is nothing but a crack or weak zone inside the Earth. When two blocks of rock or two plates rub against each other along a fault, they don't just slide smoothly they stick a little. As the tectonic forces continue to prevail, the plate margins exhibit deformation as seen in terms of bending, compression, tension and friction. The rocks eventually break giving rise to an earthquake, because of building of stresses beyond the limiting elastic strength of the rock. The building up of stresses and subsequent release of the strain energy in the form of earthquake is a continuous process, which keeps on repeating in geological time scale.

Types of Faults

Different types of faults are (Fig 1.9): Dip Slip Faults; Normal; Reverse; Strike Slip Faults; Right Lateral and Left Lateral.

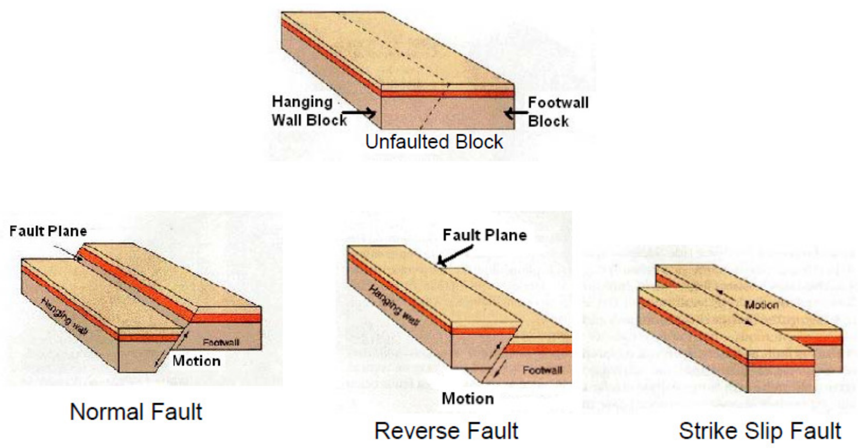


Figure 1.9 Types of faults

Fig 1.10 shows the tectonic map of India with known fault planes.

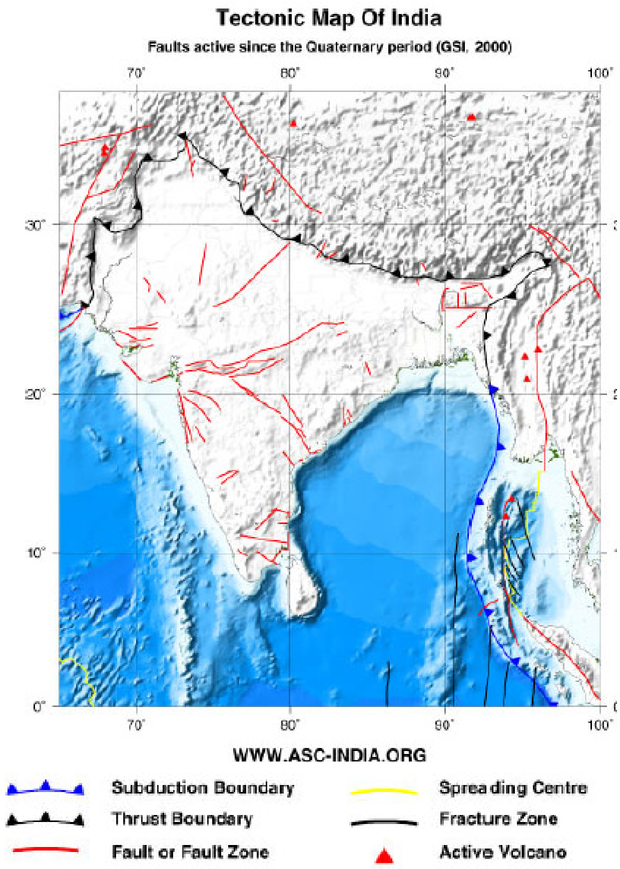


Figure 1.10 Tectonic map of India showing known fault planes

Earthquake Hazard Maps

Under the initiative of the Ministry of Urban Development, a Vulnerability Atlas of India was prepared in which the earthquake, cyclone and flood hazard maps for every state and Union Territory of India have been prepared to a scale of 1:2.5 million. The seismic zoning map was periodically updated and the latest (2007) map is shown in fig 1.11.

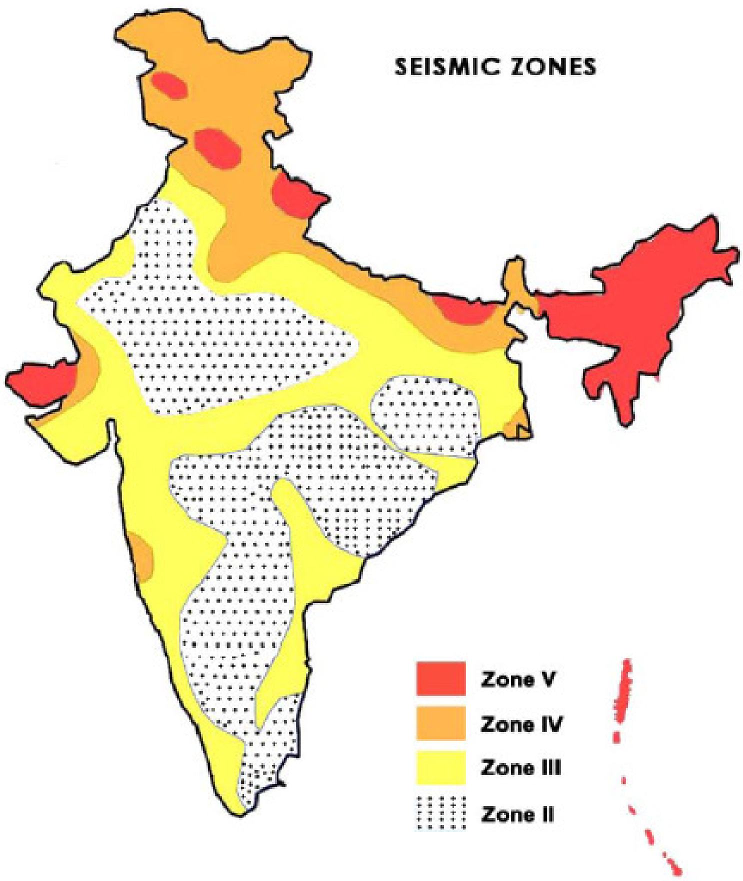


Figure 1.11 Seismic Zoning map of India - 2007

An earth shaking may occur due to various reasons: tectonic plate movements, volcanic activity, impact of meteorites, collapse of caves, rock-burst in mines, landslides/rock-falling, nuclear explosion etc. An earthquake is a phenomenon related to strong vibrations occurring on the ground due to sudden release of energy.

Most earthquakes originate within the crust. At depth beneath the Moho, the number falls abruptly and dies down to zero at a depth of about 700 km.

Classification based on focal depth

1. Shallow-focus: Shallow-focus earthquakes, which constitute about 80% of total activity, have their foci at a depth between 0 to 70 km and occur at oceanic ridges, collision and subduction zones and transform faults
2. Intermediate focus: Intermediate-focus earthquakes (focal depth between 71 and 300 km) and
3. Deep-focus: Deep-focus earthquakes (focal depth greater than 300 km) occur at subduction zones

Classification based on magnitude

Classification	Magnitude (on Richter Scale)
Micro earthquake	less than 3.0
Slight	3.1-4.9
Moderate	5.0-6.9

Great	7.0-8.0
Very Great	Greater than 8.0

Classification based on epicentral distance

Classification	Range
Local shock	< 4.0
Near shock	4.0 to 10.0
Distant shock	10.0 to 20.0
Teleseismic shock	> 20.0

There are two methods of describing how large an earthquake is, as given below:

1. The intensity of an earthquake: It is a subjective parameter that is based on an assessment of visible effects. It is therefore depends on factors other than the actual size of the earthquake.
2. The magnitude of an earthquake: It is determined instrumentally and is more objective measure of its size.

Intensity:

Intensity is another term expressing rating of an earthquake, though broadly in a qualitative manner, on the basis of its effects on living and non living things of the region visited by it. The effects are generally related to the degree of shaking experienced by these objects during an earthquake. These may range from simple harmless vibrations that can be recorded only by sensitive instruments to mild jerks and shakes to a complete devastation of a given area. Very strong buildings may collapse in a matter of seconds. Landslides may be triggered, rivers may change their courses and the land may burst upon or be displaced horizontally or twisted in any possible direction. All these and other similar effects when taken together, provide a measure for judging the qualitative intensity of an earthquake.

Two scales of seismic intensity are in vogue at present: the ten point Rossi Forrel Scale and the twelve point Mercalli scale. The first scale was jointly proposed by De Rossi of Italy and Forrel of Switzerland in 1883. Till 1931, this scale was used commonly. Since 1931, however, the other scale, proposed by Mercalli in 1902 and subsequently modified by many other seismologists has found a more common application.

In each scale, intensity is indicated by a set of whole numbers in ascending order, each number signifying a series of effects mentioned against it. Although repeated attempts have been made to correlate intensity numbers with instrumental records (ground acceleration etc), there is no general agreement as yet on such proposed relationships. Only broad range correlations may be made.

In Table 1.2 is given an outline of the Mercalli Scale of Seismic Intensities. It is often abbreviated as MM Scale meaning Modified Mercalli Scale and is quite comprehensive in its full form.

Table 1.2 Modified Mercalli Scale of Earthquake Intensities (Abridged)

Class	Ground Acceleration (mm/sec/sec)	Type Name	Typical general effects
I	<10	Instrumental	Recorded by seismographs only
II	10-25	Very feeble	Felt by some persons at rest specially those staying in upper floors
III	25-50	Slight	Felt by everyone at rest; vibrations taken like those of passing by vehicles
IV	50-100	Moderate	Felt by people in motion, like jolting, slight rattling of doors and windows as due to passing of heavy vehicles
V	100-250	Rather strong	Felt outdoors and indoors, those asleep wake up in shock; cracking of doors and windows; spilling of liquids; overturning and falling of unstable objects; ringing of church bells; stopping of pendulum clocks. However no damage to buildings
VI	250-500	Strong	General panic; people leave houses; widespread displacement of movable objects within the homes; trees can be seen shaking; poor class buildings get damaged; plaster starts cracking; movement becomes difficult
VII	500-1000	Very strong	Breaking of furniture; Hanging objects quiver; poor construction and chimneys fall; falling of brick projections and loose objects; cause loss of life
VIII	1000-2500	Destructive	General fall of chimneys, tanks and projects; steering of vehicles in motion and disordered; branches fall off from big trees; masonry buildings of C and D class start collapsing
IX	2500-5000	Ruinous	Cracks develop in the ground. Many buildings get destroyed and some strong ones get damaged; underground pipes get broken; wide cracks develop in the ground; reservoir and drainage systems get disturbed; considerable loss of life
X	5000-7500	Disastrous	General destruction of all classes of buildings, bridges, dams and embankments; landslides triggered in hilly regions and on slopes; bending of railway tracks
XI	7500-9800	Very disastrous	General fissuring of ground; very widespread damages to construction; destruction of property and its life
XII	>9800	Catastrophic	Total destruction in the region in terms of life; and installations; objects thrown into the air; Nothing left in tact

Isoseismals:

A map intensity is represented by lines representing equal intensities called as isoseismals. (Fig 1.12). Thus while using MM Scale, for instance, effects listed against values VI are observed at ten different places during a particular earthquake. A line joining all these points will be an isoseismal with a value of VI. In the same region, effects (hence intensity) may vary from place to place especially when moving outward99996s from the seriously affected areas. By joining places or points of similar effects, other isoseismal values can also be obtained. An “Isoseismal record” of the area is thus made available. This (record) may show lines that are closely spaced or are located at good distances from each other. Further they may form a regular and even pattern or they may be irregular in nature.

The outline of isoseismals is controlled by following factors:

- Nature of the shocks: A shock of shallow origin (having focus say within ten km) may give rise to high isoseismals but these would be generally confined to a small area. It may be a locally strong earthquake. A shock of deep origin, however, may produce moderate isoseismals spread broadly over a much larger area even on a regional scale.

- Nature of the rocks: If the geological constitution of the area where shock takes place is of broadly uniform nature, the isoseismals may be regular, even circular. But where geology shows structural or lithological variations in different directions, the isoseismals may be irregular and elliptical in character.

When the isoseismals record is properly plotted for a given earthquake, the location of the epicentre can be broadly determined. It shall be lying at the centre of the highest isoseismals. The focus would be then directly beneath the epicentre so located.

Magnitude:

The strength of an earthquake or strain energy released by it is usually measured by a parameter called ‘magnitude’ determined from the amplitudes and periods of seismic waves of different types.

A magnitude is a logarithmic measure of size of an earthquake or explosion base on instrument measurement. The logarithmic scale is used because earthquakes vary in size over a wide range. In actual practice, a certain earthquake record is selected as standard and assigned a magnitude value A_o. The magnitude of an observed earthquake M is then expressed as a ratio of the standard shock. In other words,

$$M = \log A - \log A_o$$

Where A is the record trace of the amplitude of an observed earthquake at a given distance as recorded on a standard instrument and A_o is the record trace of a standard earthquake.

In the Magnitude scale, either the value of amplitude of body waves or that of surface waves may be used, and the Magnitude is observed accordingly as M_b and M_s respectively. The M_s or surface wave amplitude is used for local or shallow earthquakes and the M_b or body wave magnitude is used to record stronger and teleseismic i.e distant, widespread earthquakes. When mentioned without a specification, the magnitude M often signifies a body wave value for a major shock.

The strongest earthquakes recorded so far were of magnitude of 8.9 for the great Sanriku earthquake of Japan (March 2, 1933). The Assam earthquakes of 1897 and 1950 have been assigned M-values of 8.70 and 8.75 respectively. According to a study, 95 earthquakes recorded in twentieth century till 2000 had their magnitude M_s lying between 8.0 and 8.7.

Also, depending upon the level of magnitudes, epicentral distance and the characteristics of seismographs, there are mainly four magnitude scales in use. They are:
• Local (Richter) magnitude (ML)
• Body Wave magnitude (mb)
• Surface wave magnitude (MS)
• Moment Magnitude (MW)

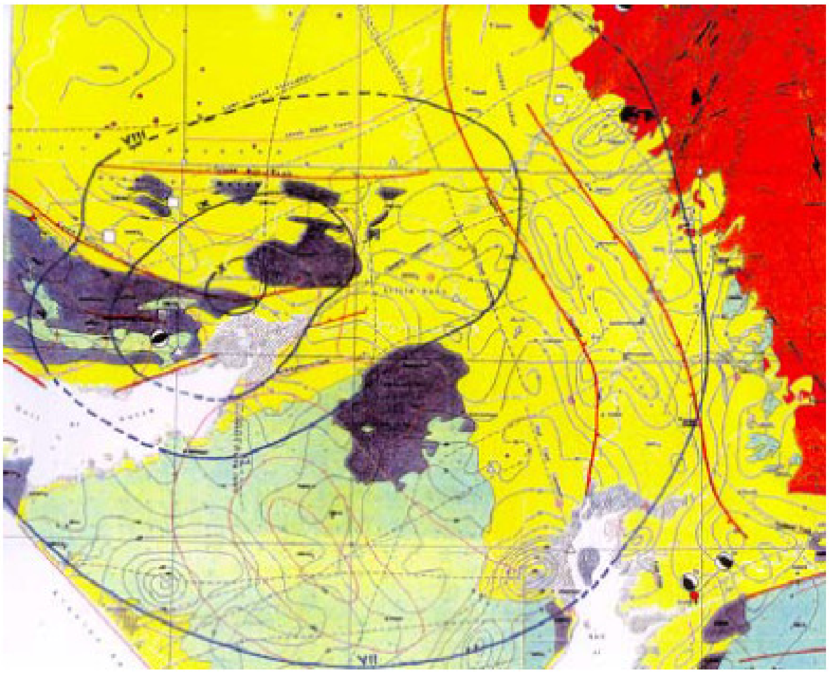


Figure 1.12 Isoseismals drawn for Gujrat earthquake (January 26, 2001)

We know that, sudden release of energy causes an earthquake. Part of energy released during an earthquake, at its origin, fractures the rock in that region. The rest travels away from the focus in all directions in the form of elastic waves. These are called seismic waves. The velocity of propagation of these waves depends upon the density and the elastic properties of the medium through which they travel. Different types of seismic waves are described below:

Body Waves
Those waves that travel through rocks are called body waves. Body waves are of two kinds, longitudinal and transverse.

Longitudinal waves: these are sometimes referred to as P waves, or primary waves or push waves. As the wave advances each particle in the solid medium is displaced in the direction of motion of these waves, as in the case of sound waves. (Fig 1.13 a). The P Waves velocities V_p is controlled by the relationship

$$V_p = \frac{\sqrt{\lambda + 2\mu}}{\rho}$$

Where λ and μ are elastic constants related to the rigidity of the medium and ρ is its density. It is obvious that P-waves would travel faster in rigid rocks.

Transverse waves: these are also known as S waves or secondary waves. These are like ripples observed in a pond. The particle motion within the transmitting medium is at right angles to the direction of wave propagation. (Fig 1.13 b) For example, the ripples one observes when a stone is thrown in a pond. If a cork is placed in water it moves up and down while the wave travels at right angles to the cork movement. So in any medium the longitudinal wave travels faster than the transverse wave and hence at any point of observation one first observes longitudinal waves. Their velocity V_p is controlled by the relationship

$$V_p = \frac{\sqrt{\mu}}{\rho}$$

As liquids do not have any rigidity i.e μ = 0, the transverse waves cannot travel through them. This is a proven fact with S-waves and has helped engineers a lot in understanding the nature of the core of the earth.

The P and S waves are sometimes collectively referred as body waves because they travel deep into the body of the Earth before re-emerging on the surface. They are recorded at far off distances from the focus in major earthquakes after refractions and reflections from deeper zones of the earth. It is well established fact that their velocities increase considerably with depth. This is shown in Table 1.3. The sudden change in their behaviour at a depth of 2900 KM (2898 km to be precise) is of great significance in the understanding of the internal structure of the earth.

Table 1.3 Velocities of P and S waves

Depth (km)	Velocity (km/sec) P-waves	Velocity (km/sec) S-waves
33	7.75	4.35
200	8.26	4.60
600	10.25	5.66
1000	11.42	6.36
1500	12.15	6.68
2000	12.79	6.93
2500	13.40	7.02
2898	13.46	-
3000	8.22	-

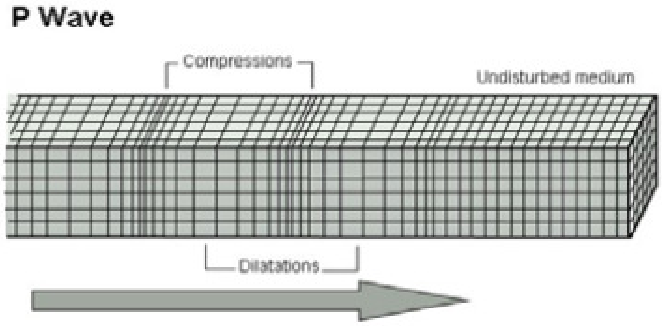


Figure 1.13 a P Wave

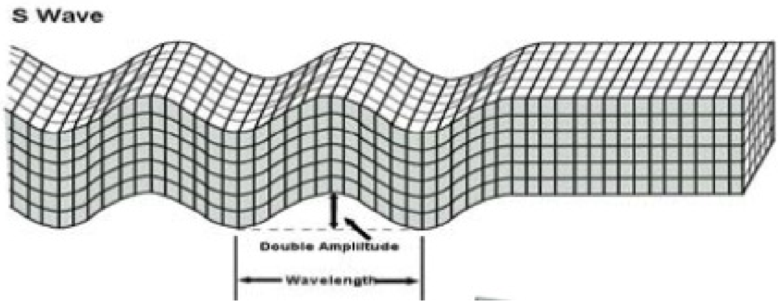


Figure 1.13 b S Wave

Surface Waves

Those waves that travel on the surface of the earth or elastic boundaries are called surface waves. They travel only at the surface or at the boundary of two different media and not into earth's interior. Two types of surface waves are Love waves and Rayleigh waves as shown in fig 1.13 c Fig 1.13 d.

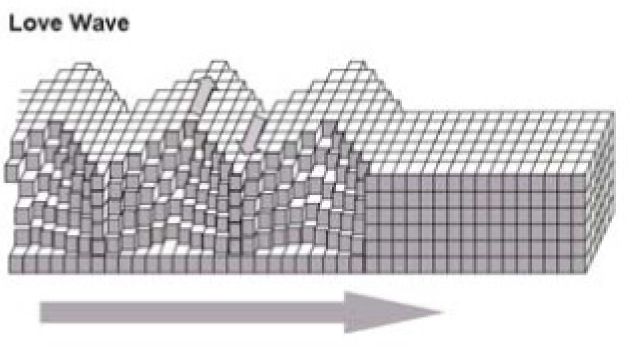


Figure 1.13 c Love Wave

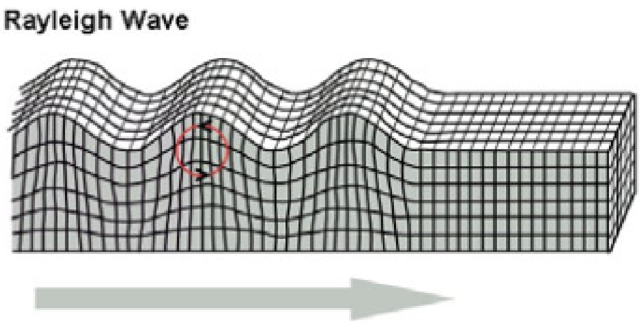


Figure 1.13 d Rayleigh Wave

Figure 1.13 Seismic Waves

The velocity of surface waves is controlled by the frequency of waves and the structure (layered or otherwise) of the ground. In the homogenous non-layered structures, Rayleigh waves travel fast and form the prominent L-waves. Their velocity may reach as much as 0.92 V_s where V_s is the velocity of shear or transverse waves in the same medium.

Energy Release

The energy released at the time of earthquake is not same for all earthquakes. Some earthquakes are so small that they can be detected only with the help of very sensitive instruments. However, the energy released at the time of a large earthquake is indeed enormous. To measure the size of an earthquake, seismologists use the Richter magnitude scale. It is a measure of the total energy released during an earthquake and is determined by the maximum amplitude of recorded seismic waves, instrumentally recorded, plus an empirical actor that takes into account the weakening of seismic waves as they spread away from the focus. This logarithmic scale is expressed in Arabic numerals. If the magnitude is increased by a factor of one, the energy released is increased by a factor of 30. There is no longer limit to the magnitude but the upper limit seems to be about 8.9, as earthquakes with magnitude greater than this have not yet been recorded.

Seismograph

Elastic waves transmitted from a single earthquake can be recorded all over the world using earthquake recording instruments called seismographs. The prototype of the modern seismograph was built in Japan about 100 years ago. Basically, the seismograph has a mass which is loosely coupled to the earth through a spring. The inertia of the mass keeps it fixed in position as the earth moves. Modern seismographs are quite complex in the construction and can record very feeble ground motions which have travelled long distances. They can magnify the ground motion upto million times before recording it. A study of seismograms, that is, the records produced by seismographs, can yield information not only about the time and place of occurrence of an earthquake, but also about the rocks through which earthquake energy travels.

Accelerograph

Rate of change of velocity with time is known as acceleration and a strong motion earthquake instrument recording accelerations is called as accelerograph. The record from an accelerograph showing acceleration as a function of time is accelerograms.

Conclusion

Study of elementary seismology tells us the overall underlying level of seismic hazard which may differ from the available evidence of historical seismicity, notable in areas experiencing present day quiescent periods. India is highly prone to earthquakes and the main seismic zone runs along Himalayan mountain range, northeast India, Andaman-Nicobar islands and Rann of Kutch region. The occurrences of earthquakes worldwide is best explained by theory of Plate Tectonics supported by a wide range of evidence that considers the earth's crust and upper mantle to be composed of several large, thin, relatively rigid plates that move relative to one another. The tectonic forces build up when two plates rub against each other along a fault which is a crack or weak zone inside the earth. The building of such stresses and subsequent release of the strain energy in the form of earthquake is a continuous process, which keeps on repeating in geological time scale Earthquakes can be classified on the basis of focal depth, magnitude and epicentral distance. Size of earthquake can be measured in terms of magnitude and intensity. Many instruments like seismograph and acceleraograph etc. have been designed to measure ground shaking in detail.

Home assignments:

Q 1: What is an earthquake ?
Q 2: Write briefly about Seismic Waves ?
Q 3: Name the major plates of the earth.
Q 4: Explain plate tectonics theory and its mechanisms.
Q 5: What is meant by the focus and epicentre of an earthquake ?
Q 6: Name the two kinds of body waves and explain how they differ.
Q 7: Discuss the main characteristics of seismic waves.
Q 8: Distinguish between Body waves and surface waves.
Q 9: Distinguish between Rayleigh and Love Waves.
Q 10: Discuss briefly the two measures of an earthquake.
Q 11: Write short notes on Seismograph.
Q 12: Write short notes on Modified Mercalli scale.

In this lecture, behaviour of earthquake resistant structure has been discussed.

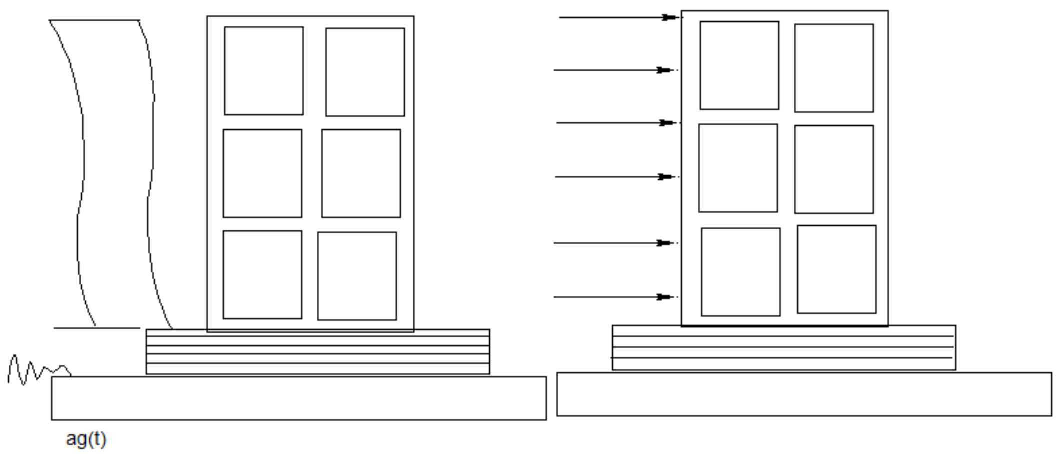
2.1 Aim: The design aim of earthquake resistant structures is to

- provide appropriate dynamic and structural characteristics for acceptable response levels under the design earthquake
- exercise some degree of control on magnitude and distribution of stiffness and mass, relative strengths of members and their ductility to achieve the desired results.

2.2 Dynamic actions on buildings- wind versus earthquake

Dynamic actions are caused on buildings by both wind and earthquakes. But design for wind forces and for earthquake effects are distinctly different. The intuitive philosophy of structural design uses force as the basis, which is consistent in wind design, wherein the building is subjected to a pressure on its exposed surface area; this is force type loading. However, in earthquake design the building is subjected to random motion of the ground at its base (Figure 2.1) which induces inertia forces in the building that in turn cause stresses; this is displacement type loading. Another way of expressing this difference is through the load-deformation curve of the building- the demand on the building is force (i.e vertical axis) in force type loading imposed by wind pressure and displacement (i.e horizontal axis) in displacement type loading imposed by earthquake shaking.

Wind force on the building has a non-zero mean component superposed with a relatively small oscillating component (Figure 2.2). Thus under wind forces, the building may experience small fluctuations in the stress field but reversal of stresses occurs only when the direction of wind reverses, which happens only over a large duration of time. On the other hand, the motion of the ground during the earthquake is cyclic about the neutral position of the structure. Thus, the stresses in the building due to seismic actions undergo many complete reversals and that too over the small duration of earthquake.



a) Earthquake ground movement at the base b) wind pressure on exposed area

Figure 2.1 Difference in the design effects on a building during natural actions of a) Earthquake ground movement at the base and b) wind pressure on exposed area

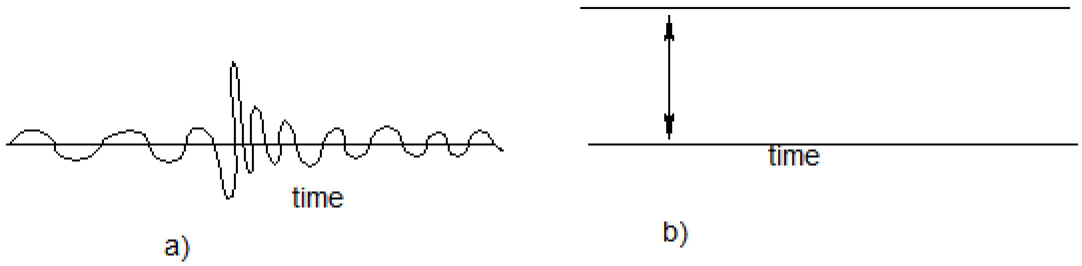


Figure 2.2 Nature of temporal variations of design actions a) Earthquake ground motion, zero mean cycle b) wind pressure- non zero mean, oscillatory

2.3 Basic aspects of seismic design

The mass of the building being designed controls seismic design in addition to the building stiffness because earthquake induces inertia forces that are proportional to the building mass. Designing buildings to behave elastically during earthquakes without damage may render the project economically unviable. As a consequence, it may be necessary for the structure to undergo damage and thereby dissipate the energy input to it during the earthquake. Therefore, the traditional earthquake resistant design philosophy requires that normal buildings should be able to resist (Figure 2.3)

a) Minor (and frequent) shaking with no damage to structural and non-structural elements
b) Moderate shaking with minor damage to structural elements and some damage to non-structural elements and
c) Severe (and frequent) shaking with damage to structural elements but with no collapse (to save life and property inside/adjoining the building)

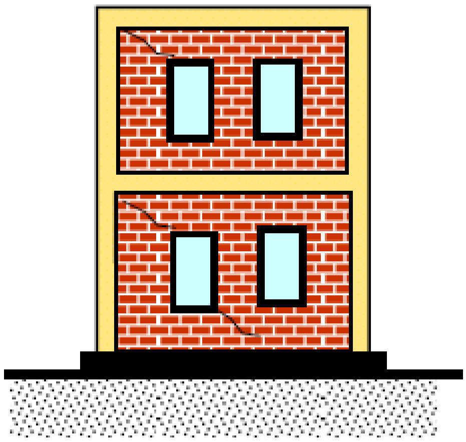


Figure 2.3 a) Earthquake-Resistant Design Philosophy for buildings: (a) Minor (Frequent) Shaking No/Hardly any damage

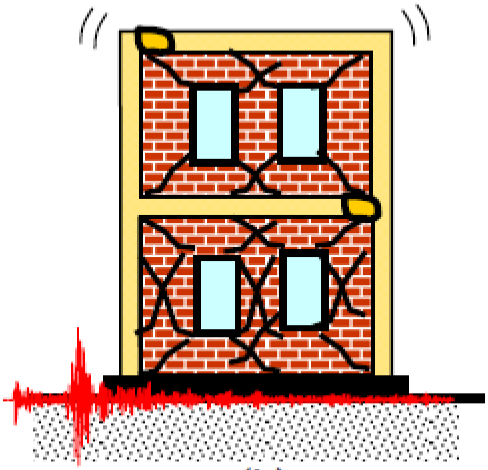


Figure 2.3 (b) Moderate Shaking – Minor structural damage, and some non-structural damage

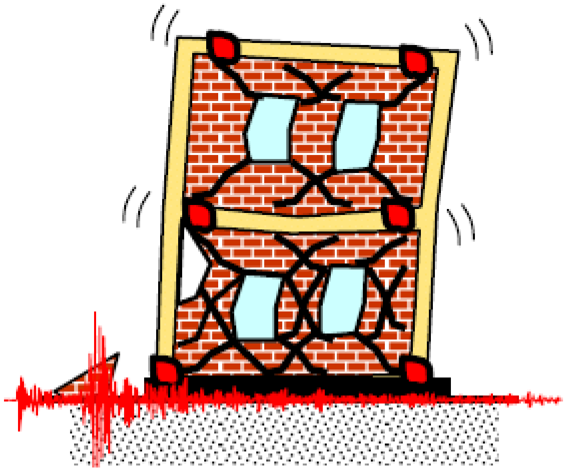


Figure 2.3 (c) Severe (Infrequent) Shaking – Structural damage, but NO collapse

Therefore, buildings are designed only for a fraction (~8-14%) of the force that they would experience if they were designed to remain elastic during the expected strong ground shaking (Figure 2.4) and thereby permitting damage (Figure 2.5). But sufficient initial stiffness is required to be ensured to avoid structural damage under minor shaking. Thus, seismic design balances reduced cost and acceptable damage to make the project viable. This careful balance is achieved based on extensive research and detailed post-earthquake damage assessment studies. A wealth of information is translated into precise seismic design provisions. In contrast, structural damage is not acceptable under design wind forces. For this reason, design against earthquake effects is called as earthquake resistant design and not earthquake proof design.

The design for only a fraction of the elastic level of seismic forces is possible only if the building can stably withstand large displacement demand through structural damage without collapse and undue loss of strength. This property is called ductility (Figure 2.6). It is relatively simple to design structures to possess certain lateral strength and initial stiffness by appropriately proportioning the size and material of the members. But achieving sufficient ductility is more involved and requires extensive laboratory tests on full scale specimen to identify preferable methods of detailing.

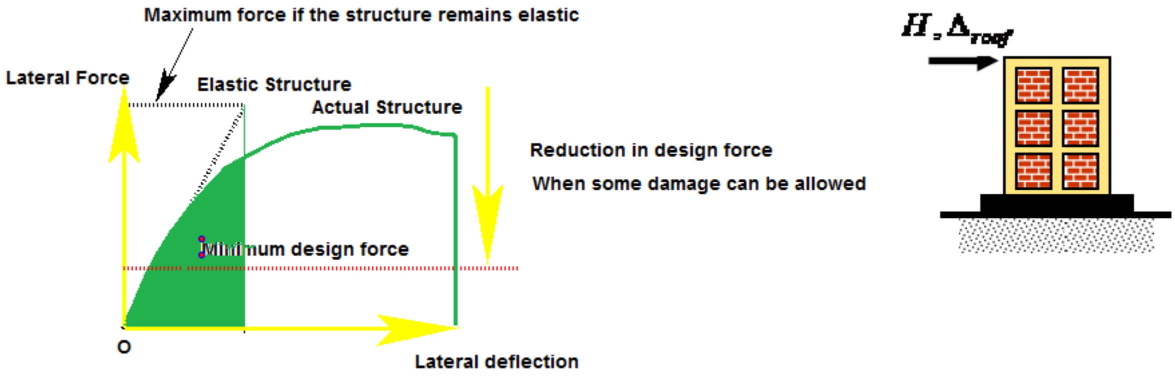


Figure 2.4 Basic strategy of earthquake design: Calculate maximum elastic forces and reduce by a factor to obtain design forces.

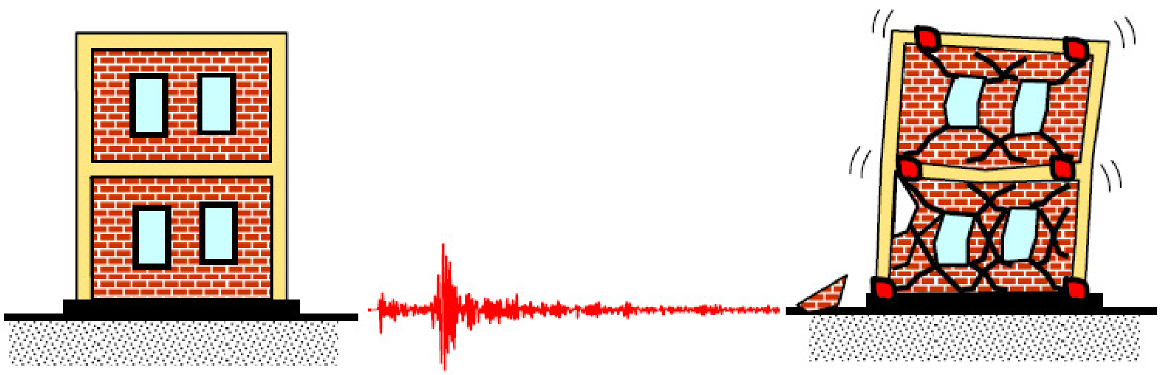


Figure 2.5 Resistant and NOT Earthquake-Proof: Damage is expected during an earthquake in normal constructions (a) undamaged building, and (b) damaged building.

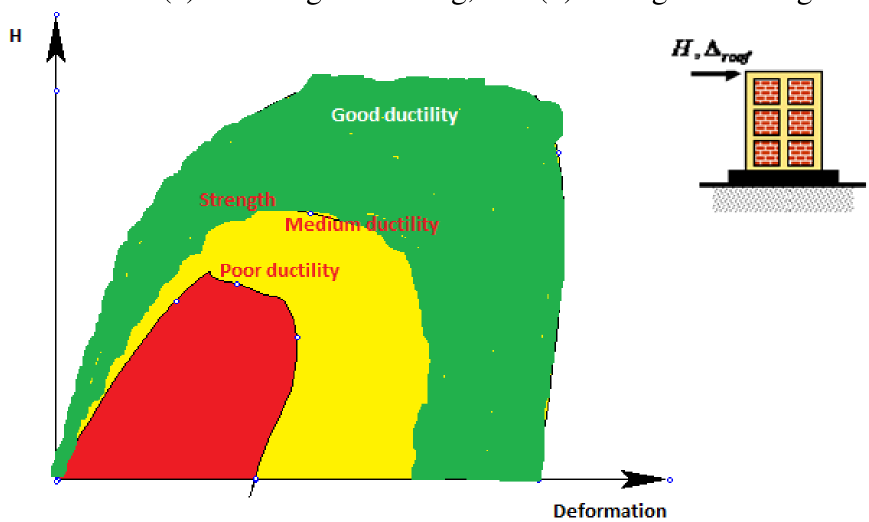
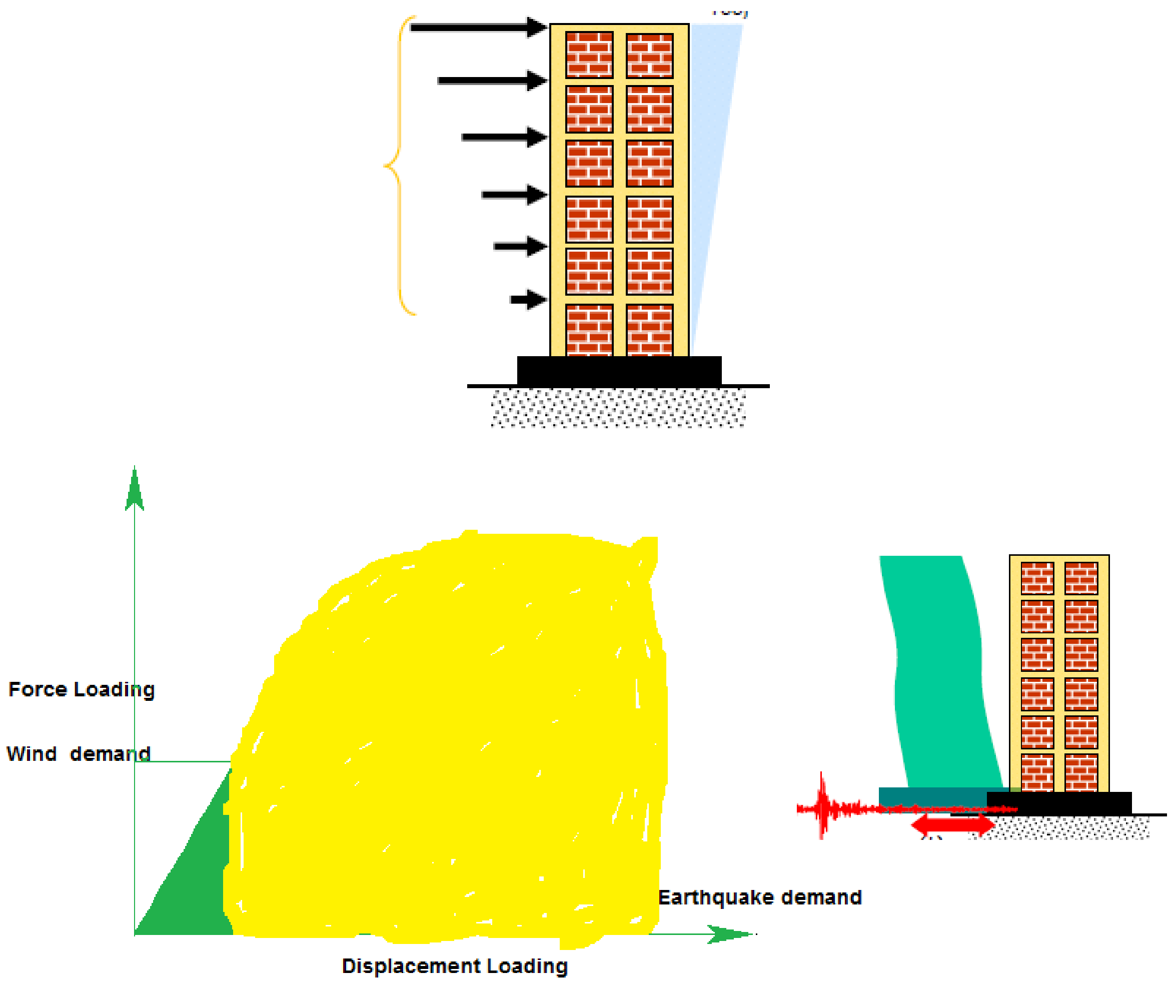
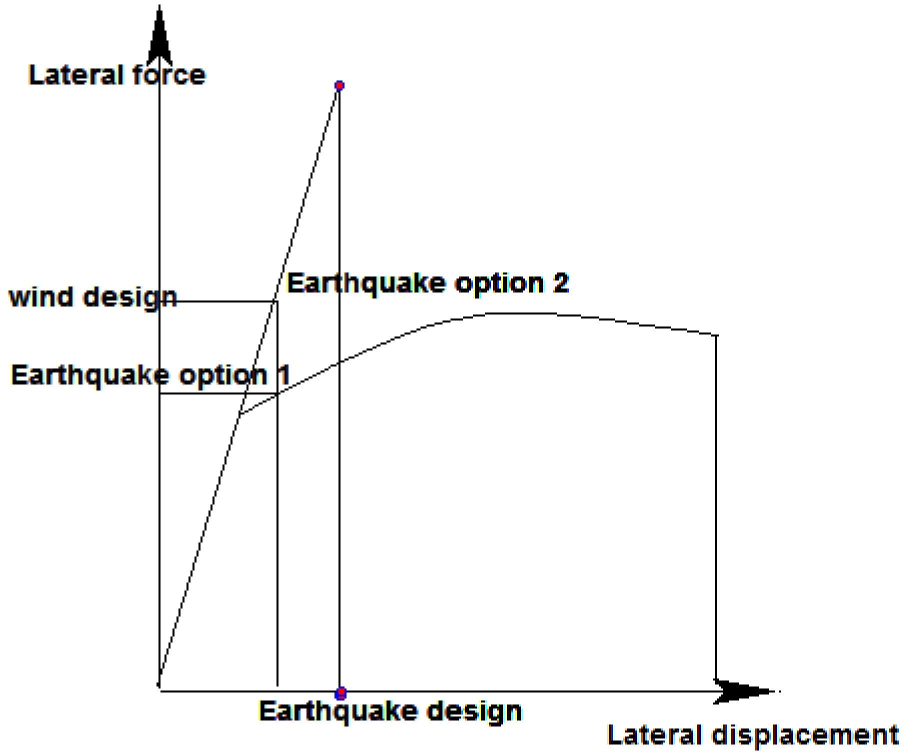


Figure 2.6 Ductility: Buildings are designed and detailed to develop favourable failure mechanisms that possess specified lateral strength, reasonable stiffness and, above all, good post-yield deformability.

In summary the loading imposed by earthquake shaking under the building is of displacement type and that by wind and all other hazards is of force type. Earthquake shaking requires buildings to be capable of resisting certain relative displacement within it due to the imposed displacement at its base, while wind and other hazards require buildings to resist certain level of force applied on it (Figure 2.7.a) while it is possible to estimate with precision the maximum force that can be imposed on a building, the maximum displacement imposed under the building is not as precisely known. For the same maximum displacement to be sustained by a building (Figure 2.7.b) wind design requires only elastic behaviour in the entire range of displacement but in earthquake design there are two options. Namely design the building to remain elastic or to undergo inelastic behaviour. The latter option is adopted in normal buildings and the former in special buildings like critical buildings of nuclear power plants.



(a)



(b)

Figure 2.7 Displacement Loading versus Force Loading: Earthquake shaking imposes displacement loading on the building, while all other hazards impose force loading on it

2.4 The Four Virtues of Earthquake Resistant Buildings

There are four aspects of buildings that architects and design engineers work with to create the earthquake-resistant design of a building, namely seismic structural configuration, lateral stiffness, lateral strength and ductility, in addition to other aspects like form, aesthetics, functionality and comfort of building. Lateral stiffness, lateral strength and ductility of buildings can be ensured by strictly following most seismic design codes. But, good seismic structural configuration can be ensured by following coherent architectural features that result in good structural behaviour.

- **Seismic Structural Configuration**

Seismic structural configuration entails three main aspects, namely (a) geometry, shape and size of the building, (b) location and size of structural elements, and (c) location and size of significant non-structural elements (Figure 2.8). Influence of the geometry of a building on its earthquake performance is best understood from the basic geometries of *convex* and *concave* lenses from school-day physics class (Figure 2.9). The line joining any two points within area of the convex lens, lies completely within the lens. But, the same is not true for the concave lens; a part of the line may lie outside the area of the concave lens. Structures with *convex* geometries are preferred to those with concave geometries, as the former demonstrate superior earthquake performance. In the context of buildings, convex shaped buildings have direct load paths for transferring earthquake shaking induced inertia forces to their bases for any direction of ground shaking, while concave buildings necessitate bending of load paths for shaking of the ground along certain directions that result in stress concentrations at all points where the load paths bend.

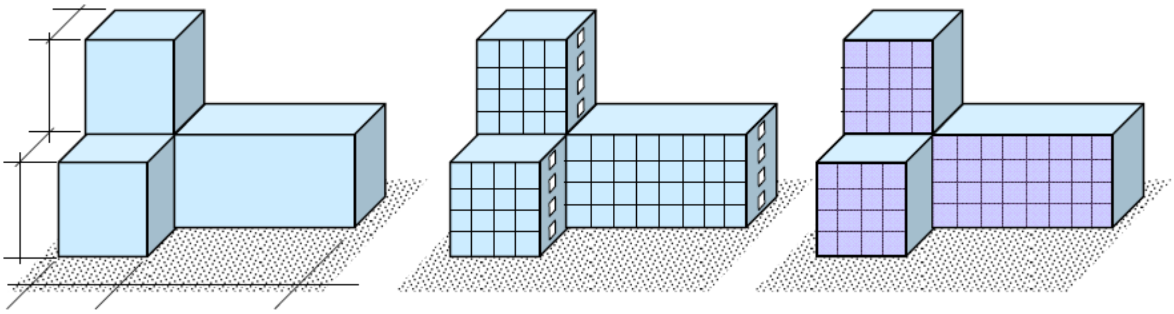
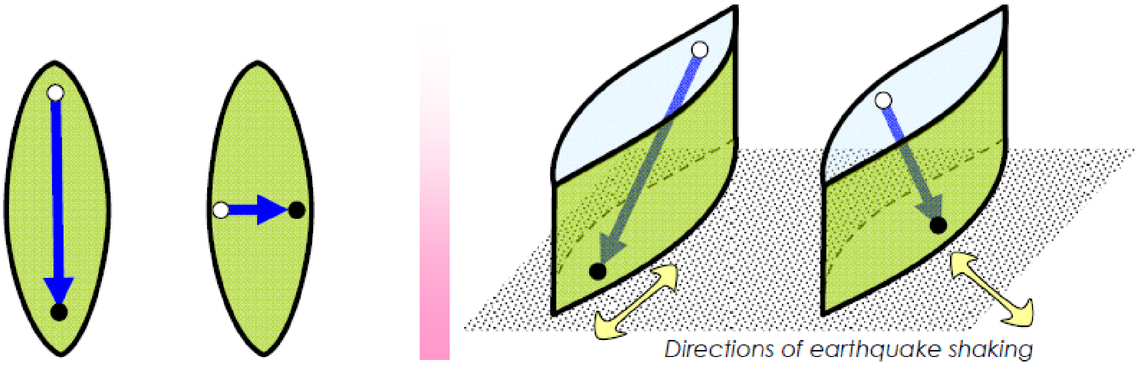
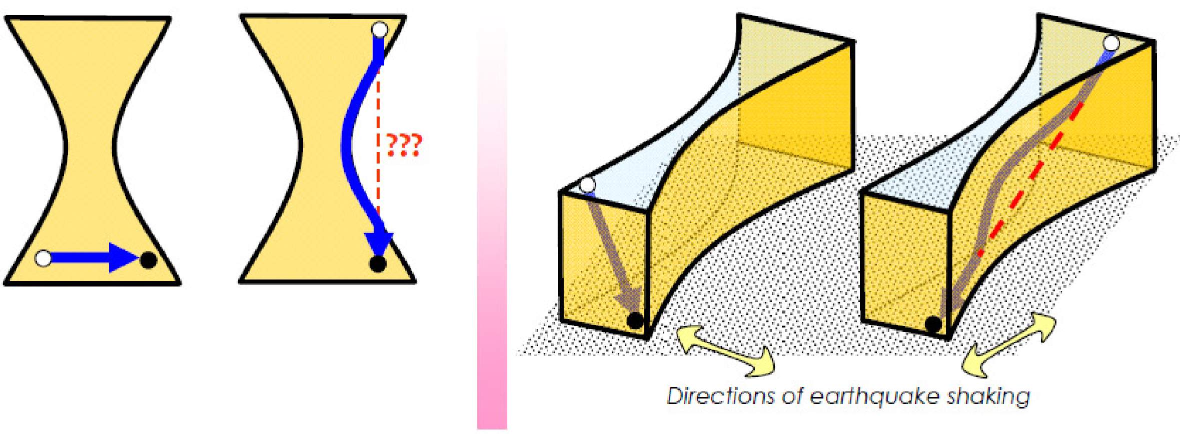


Figure 2.8 Components of seismic structural configuration: (a) overall geometry, (b) structural elements (e.g., moment resisting frames and structural walls), and (c) significant non-structural elements (e.g., façade glass)



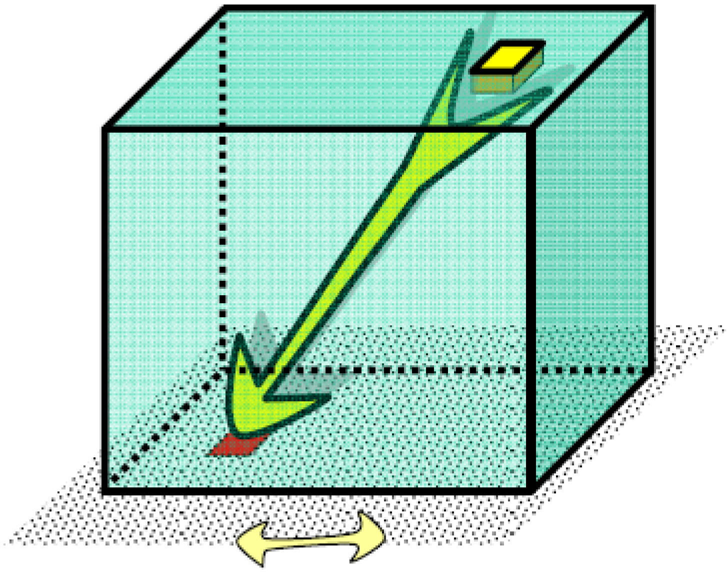
(a)



(b)

Figure 2.9 Basic forms of seismic structural configuration: Two geometries of architectural forms (a) convex, and (b) concave

Based on the above discussion, normally built buildings can be placed in two categories, namely simple and complex (Figure 2.10). Buildings with rectangular plans and straight elevation stand the best chance of doing well during an earthquake, because inertia forces are transferred without having to bend due to the geometry of the building (Figure 2.10a). But, buildings with setbacks and central openings offer geometric constraint to the flow of inertia forces; these inertia force paths have to bend before reaching the ground (Figure 2.10b, 10c)



(a)

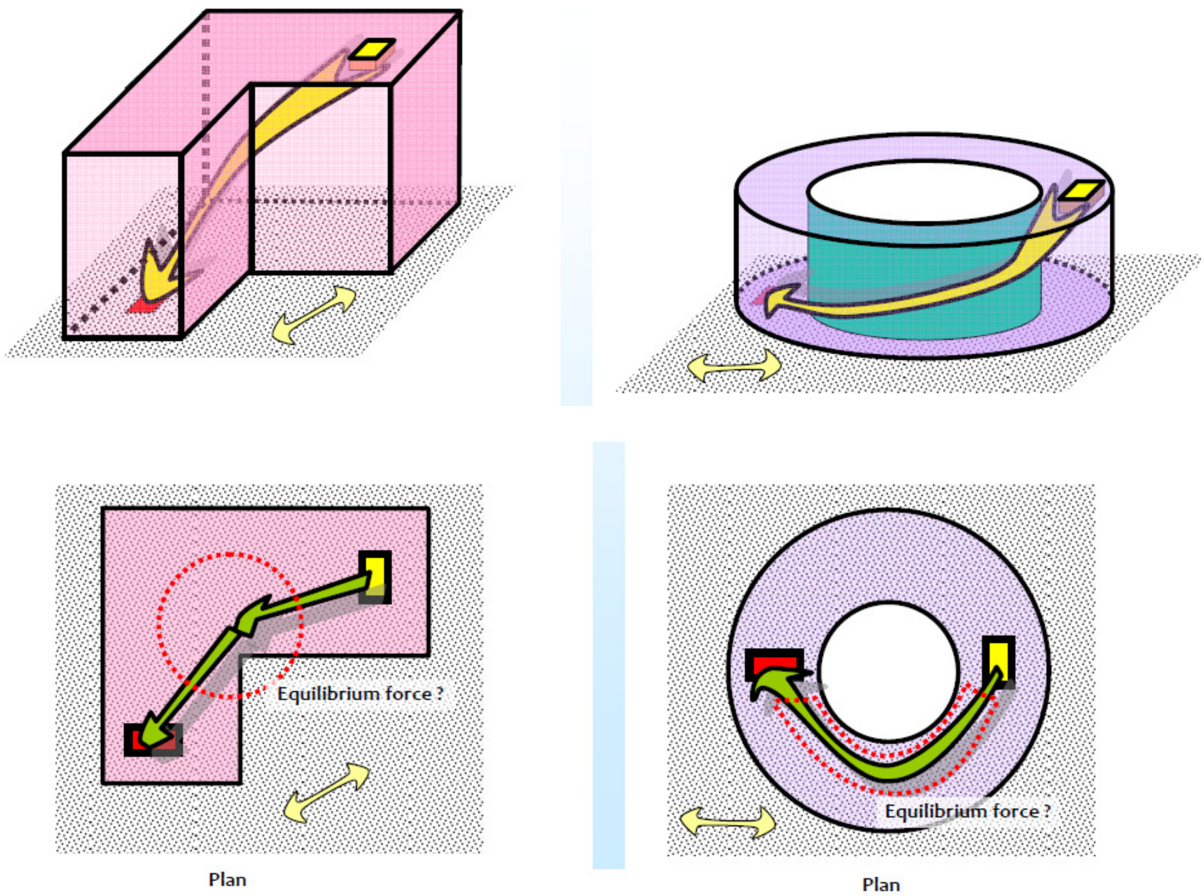


Figure 2.10 Classification of buildings: (a) Simple, and (b), (c) Complex

- Structural Stiffness, Strength and Ductility

The next three overall properties of a building, namely lateral stiffness, lateral strength and ductility, are illustrated in Figure 2.11, through the lateral load – lateral deformation curve of the building. Lateral stiffness refers to the initial stiffness of the building, even though stiffness of the building reduces with increasing damage. Lateral strength refers to the maximum resistance that the building offers during its entire history of resistance to relative deformation. Ductility towards lateral deformation refers the ratio of the maximum deformation and the idealised yield deformation. The maximum deformation corresponds to the maximum deformation sustained by it, if the load-deformation curve does not drop, and to 85% of the ultimate load on the dropping side of the load-deformation response curve after the peak strength or the lateral strength is reached, if the load-deformation curve does drop after reaching peak strength.

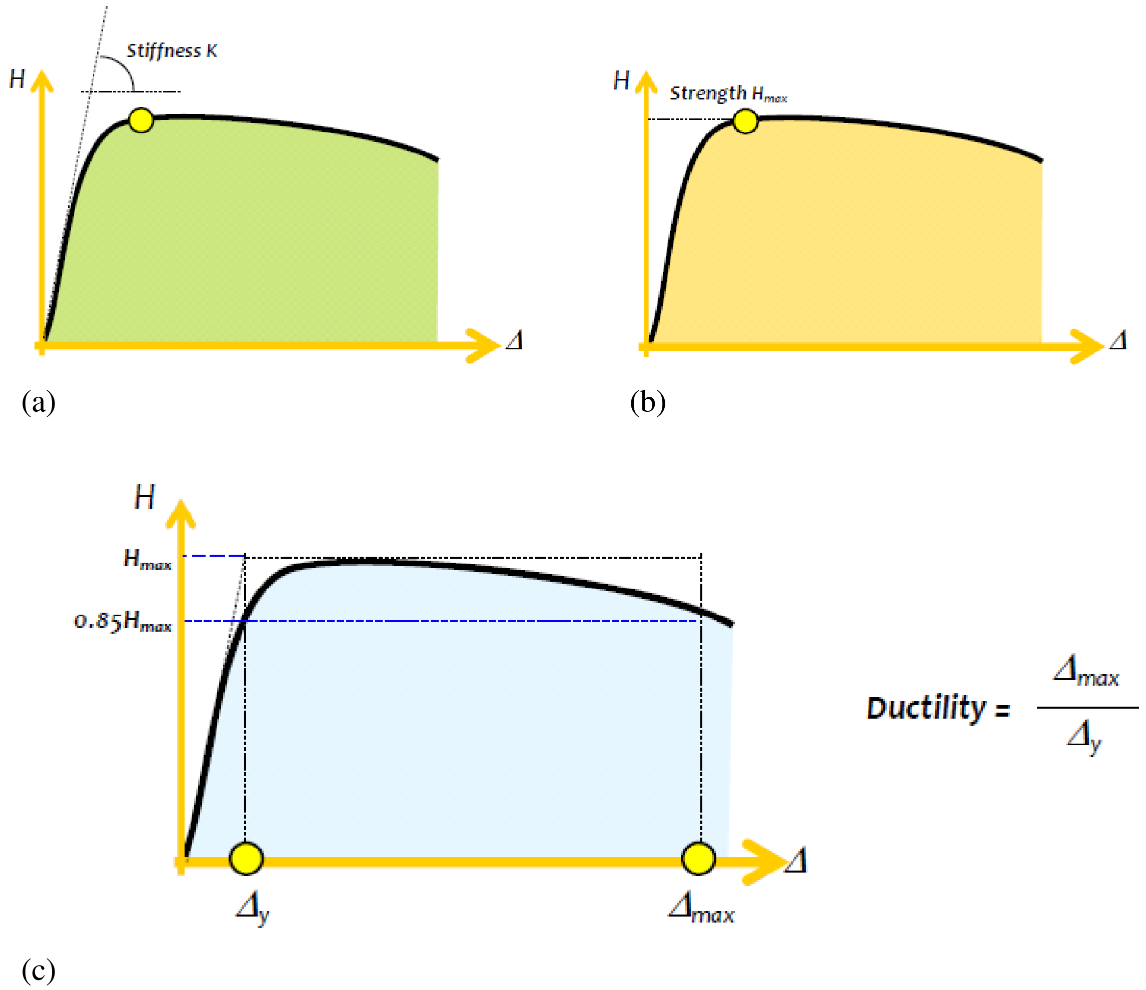


Figure 2.11 Structural Characteristics: Overall load deformation curves of a building, indicating (a) lateral stiffness, (b) lateral strength, and (c) ductility towards lateral deformation

2.4.1 **What are the four virtues ?**

All buildings are vertical cantilevers projecting out from the earth's surface. Hence, when the earth shakes, these cantilevers experience whiplash effects especially when the shaking is violent. Hence special care is required to protect them from this Jerry movement. Buildings intended to be earthquake resistant have competing demands. Firstly, buildings become expensive if designed not to sustain any damage during strong earthquake shaking. Secondly they should strong enough not to sustain any damage during weak earthquake shaking. Thirdly they should be stiff enough to not swing too much even during weak earthquakes and fourthly they should not collapse during the expected strong earthquake shaking to be sustained by them even with significant structural damage. These competing demands are accommodated in buildings intended to be earthquake resistant by incorporating four desirable characteristics in them. These characteristics called the four virtues of earthquake resistant buildings are

1. Good seismic configurations with no choices of architectural form of the building that is detrimental to good earthquake performance and that does not introduce newer complexities in the building behaviour than what earthquake is already imposing.
2. At least a minimum lateral stiffness in each of its plan directions (uniformly distributed in both plan directions of the buildings) so that there is no discomfort to occupants of the building and no damage to contents of the building.
3. At least a minimum lateral strength in each of its plan directions (uniformly distributed in both plan directions of the building) to resist low intensity ground shaking with no damage and not too strong to keep the cost of construction in check with a minimum vertical strength to be able to continue to support the gravity load and thereby prevent collapse under strong earthquake shaking and
4. Good overall ductility in it to accommodate the imposed lateral deformation between the base and the roof of the building; along with the desired mechanism of behaviour at ultimate stage.

Behaviour of buildings during earthquakes depend critically on these four virtues. Even if only one of these is not ensured, the performance of the building is expected to be poor.

Out of these, configuration is arguably the key issue that is within the realm of professional responsibility of the architect. Hence the issues of plan configuration and vertical configuration are to be emphasized and the architects are to be sensitized to the inherent inadequacies of certain types of very commonly used irregular configurations, such as plans with odd shapes and buildings with substantial offsets/overhangs (Figs. 2.12–2,14). Whereas the architects are to be taught the reasons for the poor earthquake performance of such types of configurations, they are also to be exposed to the ways of mitigating the negative effects of such design decisions through the use of rational structural systems (Fig. 2.15).

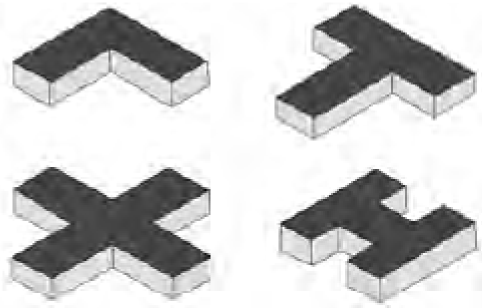


Figure 2.12 Buildings with re-entrant corners

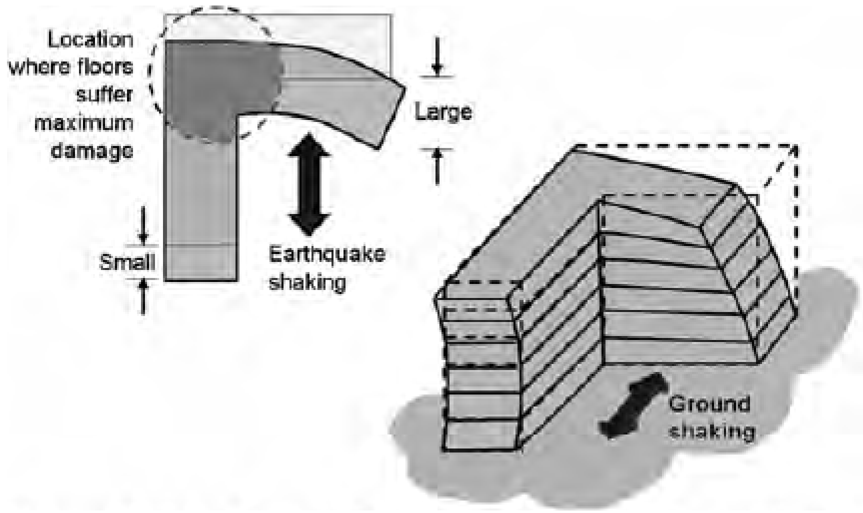


Figure 2.13 Poor earthquake behaviour of buildings with re-entrant corners

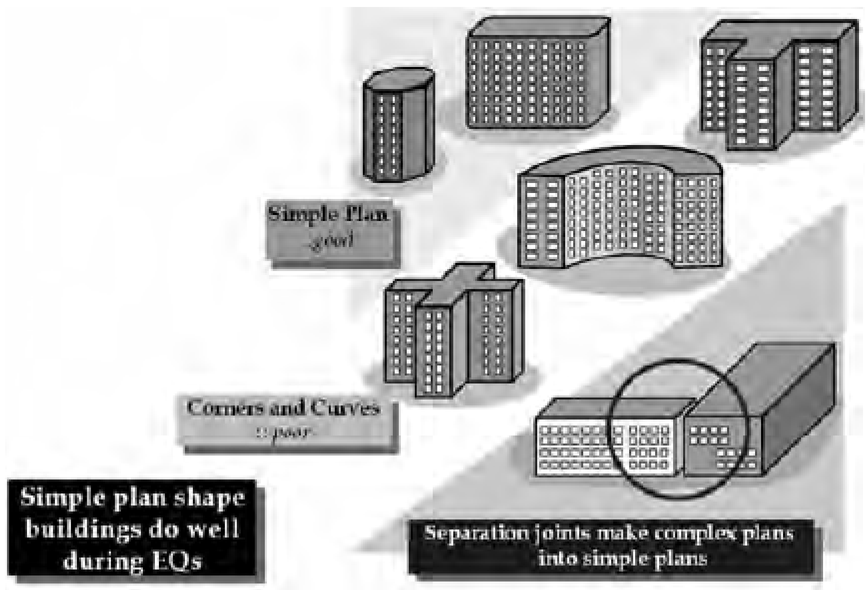


Figure 2.14 Plan configuration for earthquake performance

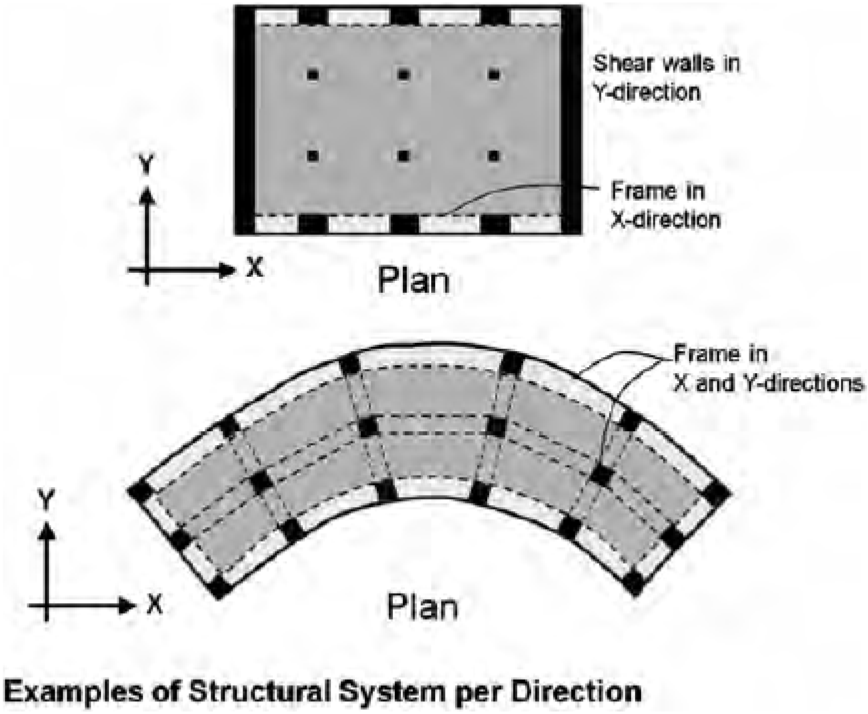


Figure 2.15 Illustration of how to use two lateral-load-resisting systems in plans

- **Who controls the four virtues ?**

Henry Degenkolb, a noted earthquake engineer of USA aptly summarized the immense importance of seismic configuration in his words “if we have a poor configuration to start with, all the engineer can do is to produce a band-aid improve a basically poor solution as best as he can. Conversely if we start off with a good configuration and reasonable framing system, even a poor engineer can not harm its ultimate performance too much. Likewise, Nathan M Newmark and Emilo Rosenbleuth, eminent professors of earthquake engineering in USA and Mexico respectively batted for the concepts of earthquake resistant design in their forward to their book. If a civil engineer is to acquire fruitful experience in a brief span of time, expose him to the concepts of earthquake engineering, no matter if he is later not to work in earthquake country.

In many countries like India, in the design of a new building, the architect is the team leader and the engineer a team member. And in the design of retrofit of an existing building, the engineer is the team leader and architect is a team member. What is actually needed is that both the architect and the engineer work together to create the best design with good interaction at all stages of the process of the design of the building. Here the architect brings in perspectives related to form, functionality, aesthetics and contents while the engineer brings the perspectives of safety and desired earthquake performance during an expected earthquake. There is a two way influence of the said parameters handled both by the architect and the engineer; their work has to be in unison.

- **How to achieve four virtues** ?

The four virtues are achieved by inputs provided at all stages of the development of the building namely in its planning, design, construction and maintenance. Each building to be built is only one of the kind ever and no research and testing is performed on that building unlike factory made products like aircrafts, ships and cars. The owner of the building trusts the professionals (i.e architect and engineer) to have done due diligence to design and construct the building. Thus professional experience is essential to be able to conduct a safe design of the building because it affects the safety of persons and property.

Traditionally in countries that have advanced earthquake safety initiatives, governments have played critical role through the enforcement of technological regime wherein the municipal authorities arrange to examine if all requisite technical inputs have been met with to ensure safety in the building before allowing the building to be built; the construction to be continued at different stage or the users to occupy the building. These stages are a) conceptual design stage b) design development stage through peer review of the structural design and constructions stage through quality control and quality assurance procedures put in place. Senior professionals (both architects and engineers) are required to head the team of professionals to design a building. These senior professions should have past experiences of having designed buildings to resist strong earthquakes under the tutelage of erstwhile senior professionals.

Home Assignments

Q1 Explain about the behaviour of buildings under wind and earthquake loading

Q2 What are basic aspects of seismic design ?

Q3 Discuss about the four virtues of earthquake resistant buildings.

Q4 Explain how seismic structural configuration affects the performance of buildings

In this lecture, cyclic behaviour of concrete and reinforcement will be covered.

3.1 Cyclic behaviour of Concrete and Reinforcement

3.1.1 Plain concrete

Plain concrete is a brittle material. During the first cycle the stress strain curve is the same as that obtained from static tests. If the specimen is unloaded and reloaded in compression, stress strain curves similar to those shown in Figure 3.1 are obtained. It can be seen that slope of the stress strain curves as well as the maximum attainable stress decrease with the number of cycles. Thus, the stress strain relationship for plain concrete subjected to repeated compressive loads is cycle dependent. The decrease in stiffness and strength of plain concrete is due to the formation of cracks. The compressive strength of concrete depends on the rate of loading. As the rate of loading increases, the compressive strength of concrete increases but the strain at the maximum stress decreases. Plain concrete can not be subjected to repeated tensile loads since its tensile strength is practically zero.

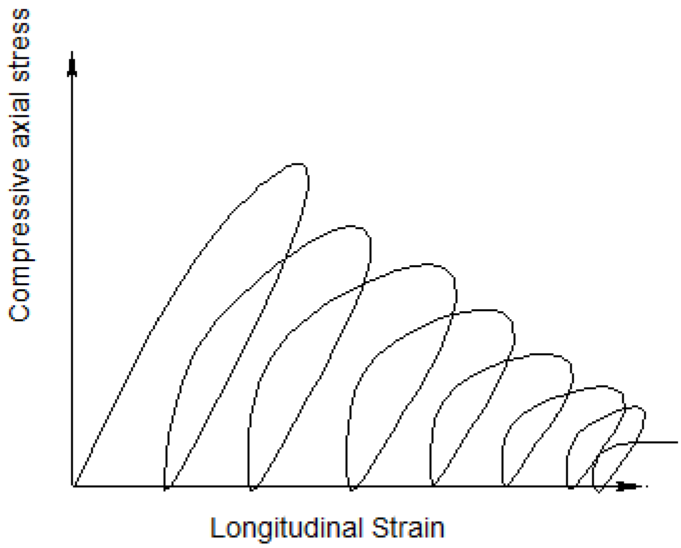


Figure 3.1 Plain concrete section under repeated compressive loading

3.1.2 Reinforcement

Reinforcing steel has much more ductility than plain concrete. The ultimate strain in mild steel is of the order of 25 % whereas, in concrete it is of the order of 0.3%. In the first cycle, the reinforcing steel shows stress strain curve similar to that obtained in the static test. After the specimen has reached its yield level and direction of load is reversed, that is, unloading begins, it can be seen in Figure 3.2 that the unloading curve is not straight but curvilinear. This curvature in the unloading segment of stress-strain curve is referred to as the Bauschinger effect after the discoverer of the phenomenon. Figure 3.2 shows one complete cycle of loading and unloading which is referred to as a hysteresis loop. The area within a hysteresis loop exhibits energy absorbed by the specimen in a cycle. In subsequent cycles, practically the same path is repeated. Thus the stress strain relationship for mild reinforcing steel subjected to repeated reversed loading is cycle independent until the specimen buckles or fails due to fatigue. It is also observed that same hysteresis loops are obtained for a specimen which is first loaded in tension followed by compression as when it is first loaded in compression followed by tension. The yield strength of reinforcement is also affected by the rate of loading.

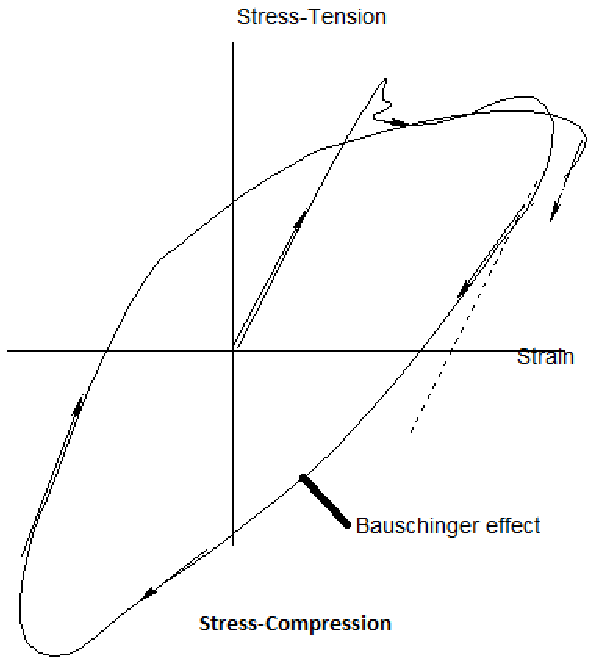


Figure 3.2 Hysteresis behaviour of reinforcing steel

3.1.3 Reinforced Concrete

Earlier it was noticed that plain concrete can be subjected only to repeated compressive loading cycles and not to repeated tensile loading cycles due to its poor tensile strength. However, reinforcing steel can be subjected to repeated reversible tensile and compressive loading cycles and exhibits stable hysteresis loops. Thus the cyclic behaviour of reinforced concrete members is significantly improved due to the presence of reinforcing steel.

Figure 3.3 shows typical load deflection curves for a cantilever reinforced concrete beam subjected to reversed cyclic loading. Reinforcing steel is present on both faces since one face is in tension during the first half loading cycle and the other face is in tension during the remaining half of the loading cycle. It can be seen in this figure that slope of a load deflection curve that is stiffness of the beam decreases with number of cycles. Moreover curves tend to pinch in near zero load. These two effects are distinct characteristics of reinforced concrete beams as well as columns and are referred to as stiffness degradation and pinching effects. The nonlinear behaviour of reinforced concrete is affected mainly by the degree of cracking in concrete, strain hardening and Bauschinger effect in reinforcing steel, effectiveness of bond and anchorage between concrete and reinforcing steel and the presence of high shear. It is not possible in quantity the contribution of each of these parameters towards the nonlinearity of reinforced concrete. Since stiffness degradation starts right after the first cycles and progresses rapidly, it becomes still more necessary to improve the capability of reinforced concrete to sustain inelastic deformations in order to avoid its collapse.

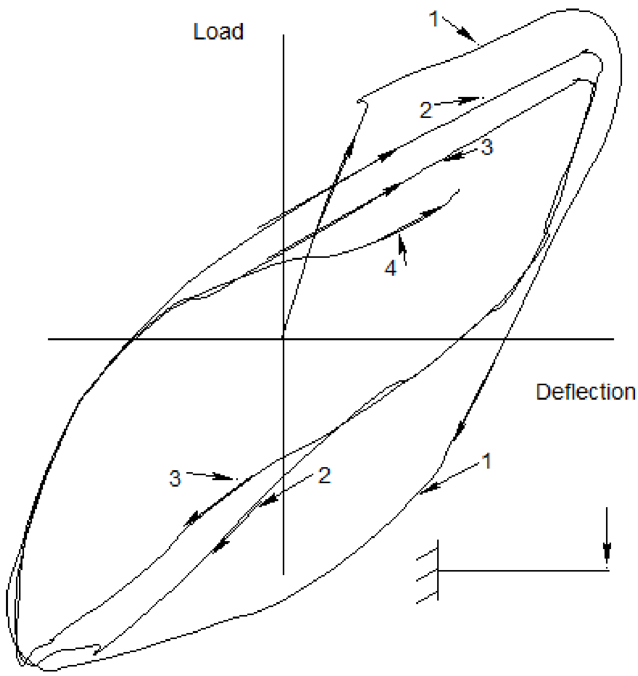


Figure 3.3 Hysteresis behaviour of a cantilever beam

Home Assignments

Q1 Explain about plain concrete section under repeated compressive loading

Q2 Discuss about hysteresis behaviour of reinforcing steel

Q3 Write short notes on cyclic behaviour of reinforced concrete members

In this lecture, significance of ductility will be covered.

4.1 Significance of ductility

Ductility is the capacity of building materials, systems, structures or members to undergo large in-elastic deformations without significant loss of strength or stiffness. It is an essential attribute of a structure that must respond to strong ground motions. It serves as the shock absorber in a building, for it reduces the transmitted force to a sustainable magnitude. The resultant sustainable force is traditionally used in design a hypothetically elastic representation of building. Therefore, the survivability of a structure under strong seismic action relies on the capacity to deform beyond the elastic range and to dissipate seismic energy through plastic deformation.

Formally ductility refers to the ratio of the displacement just prior to ultimate displacement or collapse, to the displacement at first damage or yield. This is a very important characteristic of a building since it greatly reduces the effect or response that is produced in the structure by an earthquake. This vibration as well as the accompanying deflection, is reduced by the energy that is absorbed by the large inelastic deflections of a ductile structure. Some materials such as steel and wood are inherently ductile, while others, such as masonry and concrete are brittle and fail suddenly. Building elements constructed with ductile materials have a reserve capacity to resist earthquake overloads. Therefore, buildings constructed of ductile elements such as steel and adequately reinforced concrete tend to withstand earthquakes much better than those constructed of brittle materials such as unreinforced masonry.

One way of achieving ductility in structural members is by designing elements with known limits which deform in a ductile manner. For example, in RCC members, the amount and location of steel should be such that the failure of the member occurs by steel reaching its strength in tension before concrete reaches its strength in compression. This is referred to as ductile failure. In RCC buildings the seismic inertia forces generated at floor levels are transferred through the various beams and columns to the ground. The correct building components need to be made ductile. The failure of a beam causes localized effects. However, the failure of a column can affect the stability of the whole building. Therefore it is better to make beams ductile rather than columns. Such a design method is known as strong-column and weak-beam design method.

Ductility can also be achieved by avoiding any possibility of brittle failure as shown in Table 4.1. As an example a tension bolt in a steel beam column connection should be at a safe stress level when the beam has reached its ultimate moment. For the entire structural system to be ductile, the following requirements must be met.

a) Any mode of failure should involve the maximum possible redundancy
b) Brittle type failure modes such as overturning should be adequately safe guarded so that ductile failure occurs first.

Table 4.1 Type of brittle failure

Structure	Overturning
Foundation	Rotational shear failure
Structural steel	Bolt shear or tension failure Member buckling Member tension failure Member shear failure Connection tearing
Reinforced Concrete	Bolt or anchorage failure Member tension failure Member shear failure Out-of-plane bending failure
Masonry	Toppling

Ductility is often measured by hysteretic behaviour of critical components such as a column beam assembly of a moment frame. The hysteretic behaviour is usually examined by observing the cyclic moment-rotation (or free-deflection) behaviour of the assembly. The slopes of the curves represent the stiffness of the structure and the enclosed areas are sometimes full and flat, or they may be lean and pinched. Structural assemblies with curves enclosing a large area representing large dissipated energy are regarded as superior systems for resisting seismic loading.

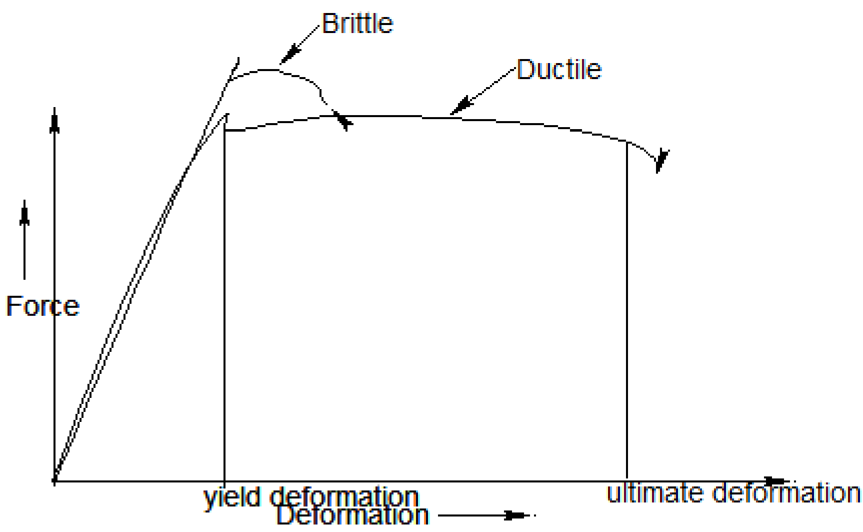


Figure 4.1 Brittle and ductile force deformation behaviour

In the typical force-deformation relation as shown in Figure 4.1, the force may be load, moment or stress, while the deformation could be elongation, curvature, rotation or strain. Δ_y is the yield deformation corresponding to yielding of the reinforcement in a cross-section or to a major deviation from the linear force-deformation curve for a member or structure. Δ_u

is the ultimate deformation beyond which the force deformation curve has a negative slope. The ductility μ is defined by the equation:

$$\mu = \frac{\Delta_u}{\Delta_y} \text{ (with respect to displacement)}$$

$$\mu = \frac{\phi_u}{\phi_y} \text{ (with respect to curvature)}$$

$$\mu = \frac{\theta_u}{\theta_y} \text{ (with respect to rotation)}$$

Conventional Civil Engineering Structures are designed on the basis of two main criteria-strength and stiffness. In case of earthquake resistant design, a new criterion, the ductility should also be added. The third criterion which is prevention of building collapse is achieved not by limiting maximum stresses or storey drift, but by providing sufficient strength and ductility to ensure that the structures do not collapse in a service earthquake. Based on the third criterion the method of seismic design is known as ductility based design. It is well known that due to economic reasons, structures are not designed to have sufficient strength to remain elastic in severe earthquakes. The structure is designed to possess adequate ductility so that it can dissipate energy by yielding and survive that shock. The ductility based design operates directly with deformation quantities and therefore, gives a better insight to the expected performance of structures rather than simply providing strength as well as the lateral strength design approach does.

Home Assignments

Q1 Discuss about displacement or ductility based design for earthquake resistant structures

Q2 Write short note on Significance of ductility

In this lecture, ductility of beam will be covered.

5.1 Ductility of Beam

The ductility of reinforced concrete beams may be defined in terms of the behaviour of individual cross section or the behaviour of entire beam.

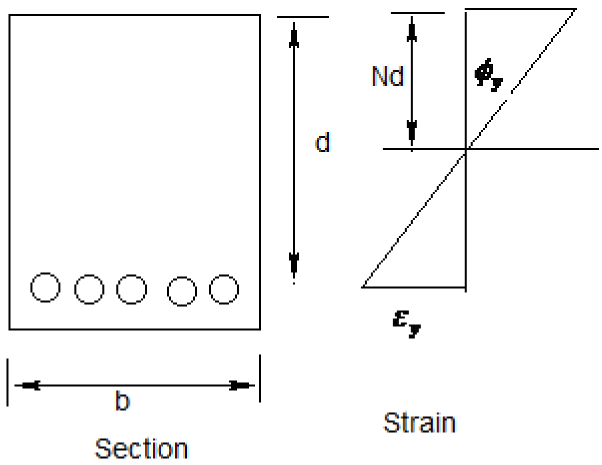


Figure 5.1 a) Yield curvature

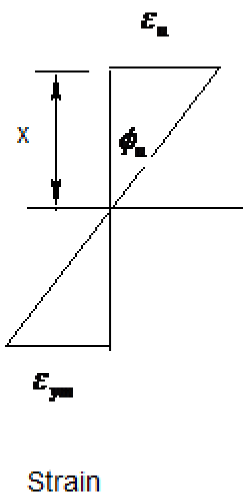


Figure 5.1 b) Ultimate curvature

Let us derive expression for the curvature ductility of a beam. With reference to Figure 5.1, the yield curvature of a simply supported beam can be computed using the elastic theory, i.e

$$\phi_y = \frac{\varepsilon_y}{d - Nd} \tag{5.1}$$

Where ε_y is the yield strain of the tensile reinforcement $\varepsilon_y = \frac{f_y}{E_s}$

d is the effective depth, Nd is the depth of neutral axis computed using elastic theory

The depth of neutral axis is determined by taking the moment of effective areas about the neutral axis

$$\frac{b(Nd)^2}{2} = mA_{st}(d - Nd) \tag{5.2}$$

$$N = -mp + \sqrt{m^2 p^2 + 2mp} \tag{5.3}$$

$$m = \frac{280}{3\sigma_{cbc}} \tag{5.4}$$

$$p = \frac{A_{st}}{bd} \tag{5.5}$$

Similarly with reference to Figure 5.1 b, the ultimate curvature can be computed as

$$\phi_u = \frac{\varepsilon_u}{x} \tag{5.6}$$

Where ε_u is the ultimate strain at crushing of concrete = 0.0035

$$x = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 f_y pd}{0.36 f_{ck}} \le x_{\lim} \tag{5.7}$$

Substituting Equation 5.1 and 5.6 into the expression $\mu = \frac{\phi_u}{\phi_y}$, we get

$$\mu = \frac{\varepsilon_u}{\varepsilon_y}\left(\frac{d - Nd}{x}\right) \tag{5.8}$$

$$\mu = \frac{\varepsilon_u}{\left(f_y / E_s\right)}\left[\frac{1 + mp - \sqrt{m^2 p^2 + 2mp}}{x/d}\right] \tag{5.9}$$

In the case of a doubly reinforced beam, a similar expression for ductility factor can be derived. The addition of compression reinforcement to a beam has relatively little effect on its yield curvature. It does however greatly increase the ultimate curvature. The depth of neutral axis at collapse can be determined from the expression:

$$0.36 f_{ck} bx + f_y^{'} A_{sc} = 0.87 f_y A_{st} \tag{5.10}$$

$$\frac{x}{d} = \left[0.87 p - \frac{f_y^{'}}{f_y} p_c \right] \frac{f_y}{0.36 f_{ck}} \tag{5.11}$$

Where $f_y^{'}$ is the stress in the compression reinforcement.

$$p_c = \frac{A_{sc}}{bd} \tag{5.12}$$

If $f_y^{'} = 0.87 f_y$, Equation 5.11 becomes

$$\frac{x}{d} = (p - p_c) \frac{0.87 f_y}{0.36 f_{ck}} \leq \frac{x_{\lim}}{d} \tag{5.13}$$

Figure 5.1 gives

$$\frac{x}{d - x} = \frac{\varepsilon_u}{\varepsilon_{ym}} \tag{5.14}$$

$$\frac{x}{d} = \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_{ym}} \tag{5.15}$$

Where ε_{ym} is the maximum strain in tensile steel = $\mu_s \varepsilon_y$

μ_s is the strain ductility in steel

Equation 5.13 can be rewritten as

$$(p - p_c) \leq \left[\frac{\varepsilon_u}{\varepsilon_u + \mu_s \varepsilon_y} \right] \frac{0.36 f_{ck}}{0.87 f_y} \tag{5.16}$$

5.2 Variables affecting ductility

1) Tension field ratio

As shown in Figure 5.2, the ductility of a beam cross section increases as the steel ratio p or $(p - p_c)$ decreases. If excessive reinforcement is provided, the concrete will crush before the steel yields leading to a brittle failure corresponding to $\mu = 1$. In other words, a beam should be designed as under-reinforced. The ductility is directly affected by the values of ε_u, f_{ck} and f_y. The ultimate strain ε_u is a function of a number of variables such as the characteristic strength of concrete, rate of loading and strengthening, effect of stirrups. The code recommends a value of 0.0035 for ε_u. It can be seen in Figure 5.2 that ductility increases with the increase in characteristic strength of concrete. Also ductility decreases with the increase in characteristic strength of steel. In fact ductility is inversely proportional to the square of f_y. It suggests that Fe415 grade steel is more desirable from the ductility point of view as compared with Fe500 grade high strength steel.

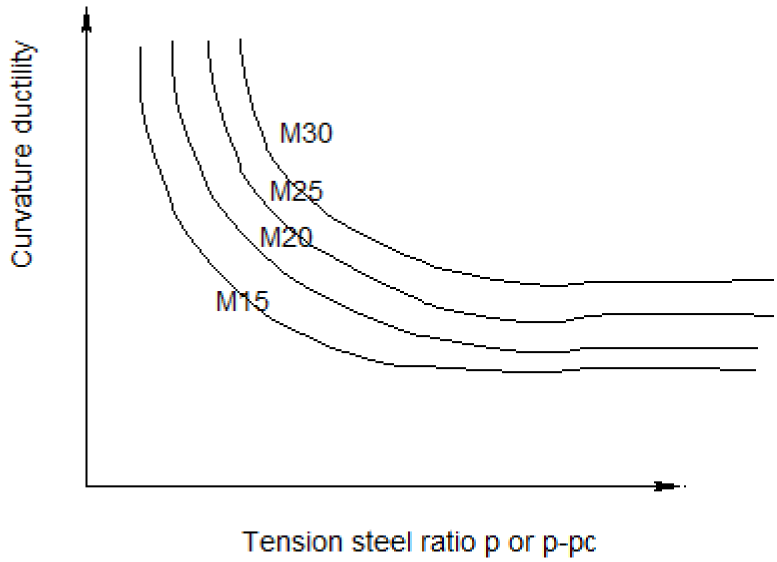


Figure 5.2 Curvature ductility in beams with Fe 415 steel

2) Compression steel ratio p

Figure 5.2 shows that ductility increases with the decrease in $(p - p_c)$ value, that is, ductility increases with the increase in compression steel.

3) Shape of cross section

The presence of an enlarged compression flange in a T-beam reduces the depth of the compression zone at collapse and then increases the ductility. If neutral axis falls in the flange, then ductility can be calculated using Figure 5.2.

4) Lateral reinforcement

Lateral reinforcement tends to improve ductility by preventing premature shear failures, restraining the compression reinforcement against buckling and by confining the compression zone, thus increasing deformation capability of a reinforced concrete beam.

Home Assignments

Q1 Derive the necessary expressions for ductility of a beam section

Q2 Write short notes on variables affecting ductility of a beam

In this lecture, design for ductility will be covered.

6.1 **Design for ductility**

The lateral loads used in seismic design are highly unpredictable under strong earthquakes. The magnitudes of lateral loads experienced by buildings are so large that an elastic design under these loads yields very large size of members. Thus, the structural members cost so much more that the rise of buildings going beyond their elastic limit is accepted with the stipulation that they do not fall down or collapse. The collapse of RCC buildings are generally preventable if the following principles of earthquake resistant design are observed.

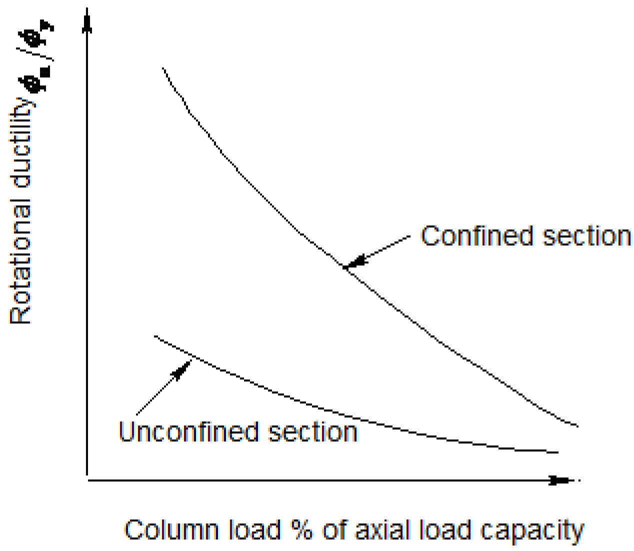
a) Failure should be ductile rather than brittle, ductility with large energy dissipation capacity (with less deterioration in stiffness) must be ensured
b) Flexural failure should precede shear failure
c) Beams should fail before columns
d) Connections should be stronger than the members which fit into them.

6.1.1 **Ductile failure**

Ductility can be defined as the ratio of the displacement at maximum load to the displacement at yield. The ability of a member to undergo large in-elastic deformations with little decrease in strength is called ductile behaviour. The available ductility of a member increases with an increase in compression steel content, concrete compressive strength and ultimate concrete strain. However, it decreases with an increase in tension steel content, steel yield strength and axial load. Reinforced concrete buildings that are properly designed in accordance with IS456: 2000 and IS13920:2002 have the desired strength and ductility to resist major earthquakes. The important points to which attention must be paid to achieve ductility are

a) The ultimate concrete strain increases by confining concrete with stirrups or special reinforcement. The confining reinforcement further increases the shear resistance and provides additional lateral support to the main reinforcement. It also makes the strength in shear greater than the ultimate strength in flexure. Figure 6.1 illustrates the effects of axial load and confinement on rotational ductile capacity.
b) Limitations on the amount of tensile reinforcement or the use of compression reinforcement increase energy absorbing capacity.
c) Use of confinement by hoops or spirals at critical sections of stress concentration such as column beam connections increase the ductility of columns under combined axial load and bending
d) Special attention must be given to details such as splices in reinforcement and the avoidance of planes of weakness that might be caused by bending or terminating all bars at the same section.

If the designer keeps these principles in mind, he will find that the building can be subjected to strong earthquakes with little or no structural damage.



ϕ_u = Curvature at ultimate stress

ϕ_y = Curvature at initiation of yield

Figure 6.1 Variation in rotational ductility for tied columns

6.1.2 **Flexural failure**

The load-deflection characteristics of earthquake-resistant structures are mainly dependent on the moment-curvature relationships of the sections. When the tension steel content is low and /or the compression steel content is high the tension steel reaches the yield strength and then a large increase in curvature can occur at near constant bending moment. This type of failure is known as tension failure. Conversely with high content of tension steel and low content of compression steel, the tension steel does not yield and section fails in a brittle manner if the concrete is unconfined. This is known as compression failure. This implies that the beams should be proportional so as to exhibit the ductile characteristics of a tension failure. Further, to prevent shear failure occurring before bending failure, the design should be such that the flexural reinforcement in a member yields while the shear reinforcement is at a stress less than yield. In beams, a conservative approach to ensure safety in the shear is to make the shear strength equal to the maximum shear demand.

6.1.3 **Weak-beam-strong-column design**

Structures should be proportioned to yield in locations most capable of sustaining inelastic deformations. Observations of failure due to yielding in columns have led to the formation of the weak-beam strong-column design in which column strengths are made at least equal to beam strengths. The intended result is columns that form a stiff, unyielding spine over the height of the building, with inelastic action limited largely to beams. In RCC frame buildings, attempts should be made especially to minimize yielding in columns, because of the difficulty of detailing for ductile response in the presence of high axial loads and the possibility that column yielding may result in the formation of demanding storey-sway mechanisms and collapse. Strength factors are usually specified by codes which try to ensure that beams fail prior to columns. However, to facilitate this situation, mild steel may be used as longitudinal reinforcement for beams and higher strength steels for columns. The greater strength increase due to strain hardening of high-strength steel can be used to an advantage in this manner.

6.1.4 Failure of joints

Figure 6.2 shows the forces acting on an interior beam-column joint. Considerable distress is observed in damaged structures in this zone. The failure may be due to the following reasons:

a) Shear within the joint
b) Anchorage failure of the beam reinforcement in the joint
c) Bond failure of the beam or column reinforcement passing through the joint

The beam-column joints are likely to fail earlier than the members framing into the joint due to the destruction of the joint zone. The shear can be carried through the broken concrete zone by inclining the main reinforcement through the hinge zone towards a point of contraflexure at the centre of the beam.

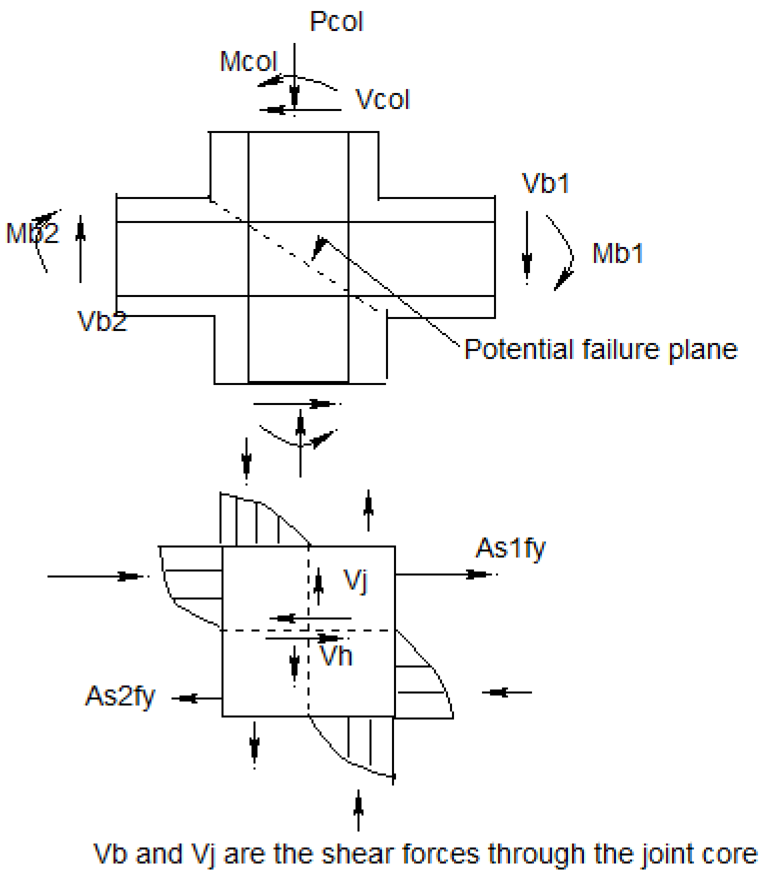


Figure 6.2 Forces on an interior beam-column joint

6.1.5 Joints at discontinuities

Proper joints should be made at discontinuities to avoid pounding of adjoining parts of the building. During seismic shaking two adjacent buildings or two adjacent units of the same building may hit (pound or hammer) each other. The pounding buildings/units can alter the dynamic response of both the buildings/units. Such pounding buildings/units should be separated by a distance equal to R times the sum of the storey displacements of each of them to avoid damaging contact when the two buildings/units deflect towards each other. When floor levels of two similar adjacent units/buildings are at the same level, the response reduction factor R may be replaced by R/2.

Home Assignment

Q1 Write briefly on design for ductility

In this lecture, ductile detailing will be covered.

7.1 Introduction

Provision for ductile detailing in the members of reinforced concrete buildings are given in IS13920:1993. These provisions are for the anchorage and splices of longitudinal reinforcement, spacing, anchorage and splices of lateral reinforcement and joint of member. It is often observed in past earthquakes that the problems in structural detailing may also be a significant cause of damage. The discussions herein focus on the provision of ductile detailing provisions for Reinforced Concrete Buildings and its possible reasons for providing structure which will be helpful to understand the importance of the ductile detailing for earthquake resistant design of structure.

7.2 Ductile Detailing

7.2.1 General Specifications

7.2.1.1 The design and construction of reinforced concrete buildings shall be governed by the provision of IS456:1978 (now IS456:2000) except as modified by the provisions of this code

7.2.1.2 For all buildings which are more than 3 stories in height, the minimum grade of concrete shall be M20

Explanation:

- The concrete strength below M20 may not have the requisite strength in bond or shear to take full advantage of the design provisions
- Bending strength of a reinforced concrete member is relatively insensitive to concrete compressive, tensile and shear strength and durability which are adversely affected by weak concrete

7.2.1.3 Steel reinforcements of grade Fe415 or less shall be used

Explanation:

- For reinforcement, the provisions firstly of adequate ductility and secondly of an upper limit on the yield stress or characteristic strength are essential. It is a general practice to limit the yield stress of reinforcement to 415 MPa.
- Strong steel is not preferable to low strength steel in earthquake prone region because typical stress strain curve of low steel shows the following advantages: a) a long yield plateau b) a greater breaking strain and c) less strength gain after first yield
- Mild steel is more ductile and its reduced post yield strength gain is advantageous. Provided that the yield strength is confined to specified limits design can determine section maximum flexure strengths in order to design other areas of the structure to prevent premature brittle shear failure
- Mild steel should be used as primarily reinforcement in areas where earthquake damage is expected, such as beam in moment resisting frames. Higher strength steel (with a yield strength more than 300 MPa) is appropriate

for other structural elements where flexural yielding cann't occur under earthquake load.

7.2.2 Flexural members

7.2.2.1 General

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

7.2.2.1.1 The factored axial stress on the member under earthquake loading shall not exceed 0.1 f_{ck}

Explanation:

- Generally axial force in the flexural member is relatively very less but if factored axial compressive stress in the frame member exceeds 0.1 f_{ck}, axial force will also be considered besides bending and member will be designed as per design guidelines for columns and frame members subjected to bending and axial load

7.2.2.1.2 The member shall preferably have a width to depth ratio of more than 0.30

Explanation:

- To provide more uniform design approach
- To minimise the risk of lateral instability
- Experience gained from past

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

Explanation:

- To decrease the sensitive to geometric error
- Experience gained from practice with RC frames resisting earthquake induced forces

7.2.2.1.4 The depth D of the member shall preferably be not more than one fourth of the clear span

Explanation:

- To take into account the nonlinearity of strain distribution and lateral buckling
- Experimental evidence indicates that under load reversals or displacement into nonlinear range, the behaviour of continuous members having length to depth ratios of less than four is significantly different from the behaviour of relatively slender members

7.2.2.2 Longitudinal reinforcement

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

Explanation:

- to ensure integrity of the member under reversed loading
- It is a construction requirement rather than behavioural requirement

1.6.2.2.1 b) the tension steel ratio on any face, at any section shall not be less than

$$p_{\min} = \frac{0.24\sqrt{f_{ck}}}{f_y}$$ where f_{ck} and f_y are in MPa

Explanation:

- To provide necessary ductility or to avoid brittle failure upon cracking

7.2.2.2.2 The maximum steel ratio on any face at any section shall not exceed $p_{\max} = 0.025$

Explanation:

- To avoid steel congestion and limit shear stresses in beams of typical proportions
- Practically low steel ratio should be used whenever possible

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

Explanation:

- To ensure adequate ductility at potential plastic hinge regions and to ensure that minimum tension reinforcement is present for moment reversal
- To allow the possibility of the positive moment at the end of a beam due to earthquake induced lateral displacements exceeding the negative moments due to gravity loads
- To produce balanced conditions and limit the incorrect assumptions such as linear strain distribution well defined yield point for the steel, limiting compressive strain in concrete of 0.003 and compressive stress in the shell concrete

7.2.2.2.4 In an external joint both the top and the bottom bars of the beam shall be provided with an 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 degrees bend as shown in Figure 7.1. In an internal joint, both face bars of the beam shall be taken continuously through the column

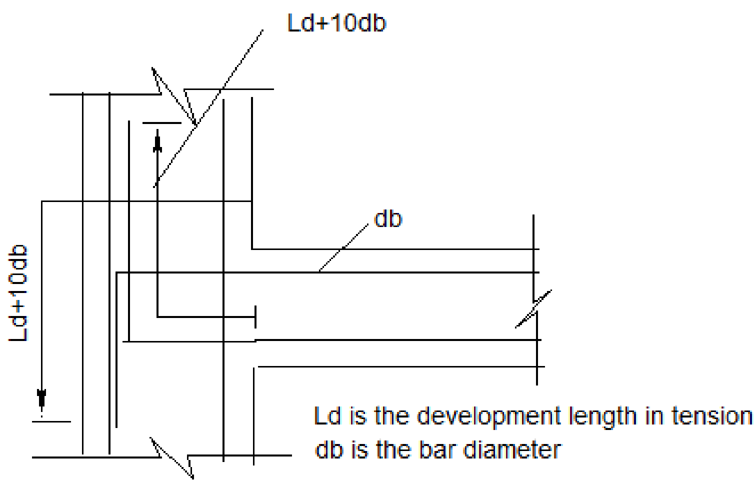


Figure 7.1 Anchorage of beam bars in an external joint

Explanation:

- Such arrangement will make a ductile junction and provide adequate anchorage of beam reinforcement into columns
- The capacity of the beam is developed by embedment in the column and within the compression zone of the beam on the far side of the connection
- The length available for the development of the strength of a beam bars is gradually reduced during cyclic reversals of earthquake actions because of the yield penetration from the face of a column

7.2.2.2.5 The longitudinal bars shall be spliced only if hoops are provided over the entire splice length at spacing not exceeding 150 mm as shown n Figure 7.2. The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided a) within a joint b) within a 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

Explanation:

- Lap splices of reinforcement are prohibited at regions where flexural yielding is anticipated because such splices are not reliable under conditions of cyclic loading into the inelastic range
- Transverse reinforcement for lap splices at any location is mandatory because of the possibility of loss of concrete cover

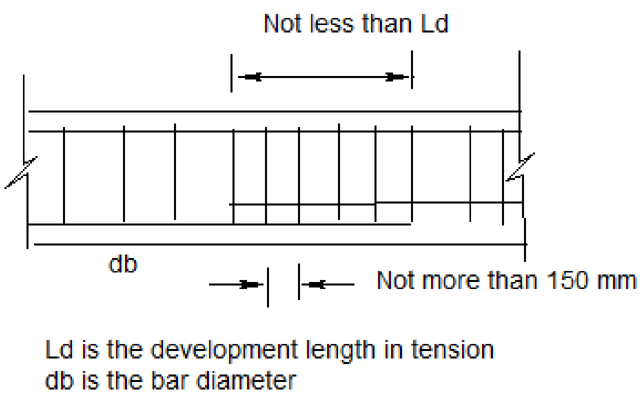


Figure 7.2 Lap, splice in beam

7.2.2.2.6 Use of welded spliced and mechanical connections may also be made. However not more than half the reinforcement shall be spliced at a section where flexural yielding may take place. The location of splices shall be governed by 1.6.2.2.5

Explanation:

- Welded splices are one in which the bars are lap welded or butt welded to develop the breaking strength of the bar
- A mechanical connection is a connection which relies on mechanical interlock with the bar deformation to develop connection capacity
- In a structure undergoing inelastic deformations during an earthquake tensile stresses in reinforcement may approach the tensile strength of the

reinforcement. The requirement of welded spliced and mechanical connections is intended to avoid a splice failure when the reinforcement is subjected to expected stress levels in yielding regions

- The location of welding splices is restricted because tensile stresses in reinforcement in yielding regions cannot exceed the strength requirement

7.2.2.3 Web Reinforcement

7.2.2.3.1 web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135^{o} hook with a 10 diameter extension (but not less than 75 mm) at each end that is embedded in the confined core as shown in Figure 7.3 a. In compelling circumstances it may also be made up of two pieces of reinforcement: A U stirrup with a 135^{o} hook and a 10 diameter extension (but not less than 75 mm) at each end embedded in the confined core and a cross tie as shown in Figure 7.3 b. A cross tie is a bar having a 135^{o} hook with a 10 diameter extension (but not less than 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

Explanation:

- stirrups are required to prevent the compression bar from buckling
- transverse reinforcement is required to confine the concrete in the regions where yielding is expected so as to minimize strength degradation
- to provide shear strength for full flexural capacity of the member

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

Explanation:

- This refers to connection and durability (corrosion of reinforcement) rather than behavioural requirement.

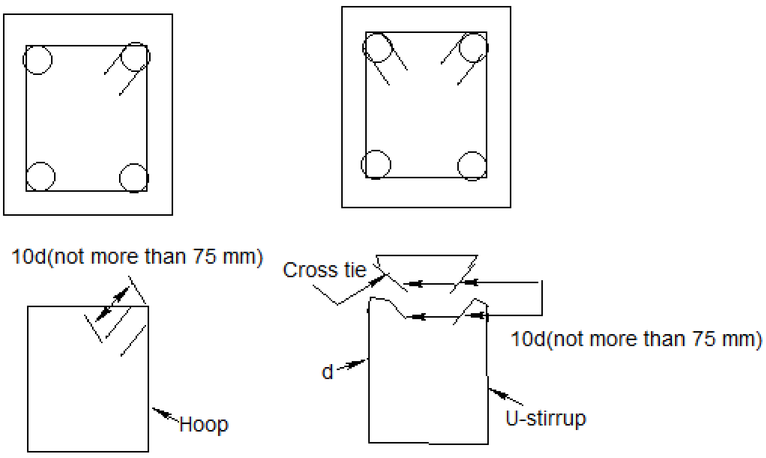


Figure 7.3 Beam web reinforcement

Home Assignments

Q1 In what ways do stirrups help RCC beams

Q2 List the concerns with regards to joints in RCC frames

In this lecture, let us discuss about the simple problems based on the concepts discussed in previous classes.

8.1 Problem: Compare the ductility with respect to curvature of the cross section of the beam as shown in the given figure using a) M20 and Fe250 b) M20 and Fe 415 c) M20 and Fe 500

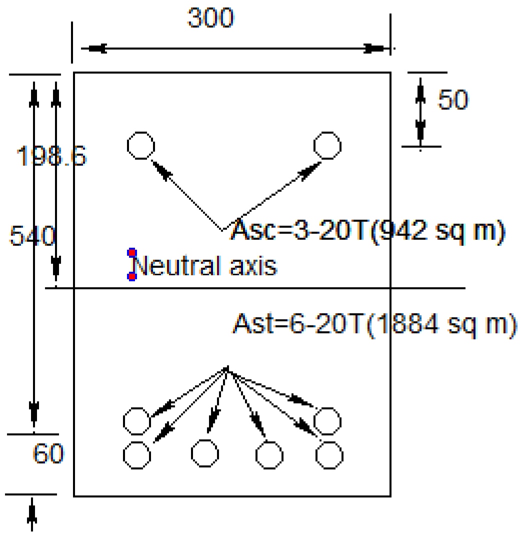


Figure 8.1: Details of doubly reinforced beams

Solution:

Data given:

Breadth b=300 mm

Effective depth d=540 mm

Overall depth D=600 mm

Effective cover d'=50 mm

Area of tensile reinforcement Ast= 1884 mm^2 (6-20T)

Area of compression reinforcement Asc=942 mm^2 (3-20 T)

Characteristic Compressive strength of concrete f_{ck}= 20 MPa

Yield strength of tensile reinforcement f_y = 250 MPa

1) Checking for minimum and maximum percentage reinforcement

Minimum percentage of Ast= $100 \times 0.24 \times \sqrt{\frac{f_{ck}}{f_y}} = 100 \times 0.24 \times \sqrt{\frac{20}{250}} = 0.429$

Maximum percentage of Asc=2.5

For the current problem

$$p_c = \frac{100A_{sc}}{bd} = \frac{942\times100}{300\times540} = 0.581$$

$$p = \frac{100A_{st}}{bd} = \frac{1884\times100}{300\times540} = 1.162$$

Hence Ast and Asc are within minimum and maximum percentages respectively i.e $0.429 \prec 1.162 \prec 2.5$ and $0.429 \prec 0.581 \prec 2.5$

2) Determination of k

The modular ratio is given as $m = \frac{280}{3\sigma_{cbc}} = \frac{280}{3\times7} = 13.33$

Taking moments of compression concrete, compression steel and tension steel about the neutral axis, we have

$$b\left(\frac{x^2}{2}\right) + A_{sc}(1.5m-1)(x-d') = mA_{st}(d-x)$$

$$300\left(\frac{x^2}{2}\right) + 942(1.5m-1)(x-50) = 13.33\times1884(540-x)$$

$$x^2 + 286.7134x - 96373.822 = 0$$

$$x = 198.5862272mm$$

$$k = \frac{x}{d} = \frac{198.586272}{540} = 0.3677$$

3).. Equating the steel compressive forces C due to concrete $(= 0.36 f_{ck} b x_u)$ and compressive steel $(= f_{sc} A_{sc})$ with the tensile force $T(= 0.87 f_y A_{st})$ for a doubly reinforced rectangular section, we have

$$0.36 f_{ck} b x_u + f_{sc} A_{sc} = 0.87 f_y A_{st}$$

$$\frac{x_u}{d} = \frac{0.87 f_y A_{st}}{0.36 f_{ck} bd} - \frac{f_{sc} A_{sc}}{0.36 f_{ck} bd}$$

$$\left(\frac{x_u}{d}\right) = \left(\frac{f_y}{36 f_{ck}}\right)\left[0.87p - \frac{f_{sc} p_c}{f_y}\right] \text{ where } p = \frac{100A_{st}}{bd} \quad p_c = \frac{100A_{sc}}{bd} \text{ and } f_{sc} = 0.87 f_y$$

$$\left(\frac{x_u}{d}\right) = \left(\frac{0.87 f_y}{36 f_{ck}}\right)(p - p_c) \le \left(\frac{x_{u,\max}}{d}\right)$$

For $f_y = 250MPa$

$$\left(\frac{x_u}{d}\right) = \left(\frac{0.87 \times 250}{36 \times 20}\right)(1.162 - 0.581) = 0.1755$$

$$\left(\frac{x_{u,\max}}{d}\right) = 0.53$$

For $f_y = 415MPa$

$$\left(\frac{x_u}{d}\right) = \left(\frac{0.87 \times 415}{36 \times 20}\right)(1.162 - 0.581) = 0.2913$$

$$\left(\frac{x_{u,\max}}{d}\right) = 0.48$$

For $f_y = 500MPa$

$$\left(\frac{x_u}{d}\right) = \left(\frac{0.87 \times 500}{36 \times 20}\right)(1.162 - 0.581) = 0.3510$$

$$\left(\frac{x_{u,\max}}{d}\right) = 0.46$$

4),, Curvature ductility for fy=250 MPa is given by

$$\mu = \left(\frac{\varepsilon_{uc}}{\varepsilon_y}\right)\left(\frac{1-k}{x_u/d}\right) = \left(\frac{0.0035}{250/200000}\right)\left(\frac{1-0.3677}{0.1755}\right) = 10.088$$

5),, Curvature ductility for fy=415 MPa is given by

$$\mu = \left(\frac{\varepsilon_{uc}}{\varepsilon_y}\right)\left(\frac{1-k}{x_u/d}\right) = \left(\frac{0.0035}{250/200000}\right)\left(\frac{1-0.3677}{0.2913}\right) = 3.66$$

6),, Curvature ductility for fy=500 MPa is given by

$$\mu = \left(\frac{\varepsilon_{uc}}{\varepsilon_y}\right)\left(\frac{1-k}{x_u/d}\right) = \left(\frac{0.0035}{500/200000}\right)\left(\frac{1-0.3677}{0.3510}\right) = 2.522$$

From the above, it shows that ductility decreases with the increase in characteristic strength of steel. It suggests that Fe 415 grade steel is more desirable from the ductility point of view as compared with as compared with Fe 500 grade high strength steel. For parametric studies A Matlab program is given as below

% Program for computation of ductility factor

```
b=300
d=540
overd=600
effd=50
nc=3
nt=6
diat=20
diac=20
ast=nt*(pi/4)*diat^2
asc=nc*(pi/4)*diac^2
fck=20
fy=415
minast=(100*0.24*sqrt(fck))/fy
maxasc=2.5
pc=100*asc/(b*d)
p=100*ast/(b*d)
sigcbc=7
m=280/(3*sigcbc)
factor1=(asc*(1.5*m-1)+m*ast)/(b/2)
factor2=((-asc*(1.5*m-1)*effd)+(-m*ast*d))/(b/2)
valuex1=(-factor1+sqrt(factor1^2-4*1*factor2))/2
valuex2=(-factor1-sqrt(factor1^2-4*1*factor2))/2
k=valuex1/d
xu=(d*0.87*fy*(p-pc))/(0.36*fck)
esc=0.0035
ey=fy/200000
mu=(((esc/ey)*(1-k))/(xu/d))*100
xax=(p-pc)/(1.0);
yax=mu
plot(xax,yax,'bo');
xlabel('p-pc');
ylabel('curvature ductility');
```

8.2 Problem: A Reinforced concrete frame consists of beams having spans of 6m c/c. A typical floor inner beam carries a negative bending moment of 450 KNm and a shear of 325 KN at the face of beam column joint due to gravity and earthquake loads. Design the beam solution for ductility.

Solution:

Let the cross section of beam = 350 mm x 650 mm

f_{ck} = 20 MPa

f_y = 415 MPa

Effective cover for tension steel = 75 mm

Factored bending moment = 450 x 1.2 = 540 KNm

$x_m =$ 0.48 d = 0.48 x 650 = 312 mm

$M_{ulim} = 0.36\, f_{ck}\, b\, x_m\, (d-0.42x_m) = 0.36\times20\times350\times312\times(650-0.42\times312) = 408.027$ KNm < 540 KNm

The section can be designed as a doubly reinforced section. For M20 concrete and Fe 415 grade steel (d'/d = 0.075) or d'=40 mm where d' = effective cover for compression steel.

$$M_{u\lim} = 0.87 f_y A_{st1}(d - 0.42 x_m)$$

$$408.027 \times 10^6 = 0.87 \times 415 \times A_{st1}(650 - 0.42 \times 312)$$

$$A_{st1} = \frac{1130 \times 112.479}{518.960} = 2177.648$$

$$\frac{A_{st1}}{bd} = \frac{2177.648}{350 \times 650} = 0.95\%$$

$$\sigma_{sc} = 0.87 f_y$$

$$M - M_{\lim} = \sigma_{sc} A_{sc}(d - d')$$

$$10^6(540 - 408.027) = 0.87 \times 415 \times A_{sc}(650 - 40)$$

$$A_{sc} = 599.222$$

$$\frac{A_{sc}}{bd} = \frac{599.222}{350 \times 650} = 0.263\%$$

$$\sigma_{sc} A_{sc} = 0.87 f_y A_{st2}$$

$$A_{st2} = 599.222$$

$$A_{st} = A_{st1} + A_{st2} = A_{st} = 2776.87 mm^2$$

$$p = \frac{100 A_{st}}{bd} = 1.22\%$$

Minimum tension steel $p_{\min} = \frac{0.24\sqrt{f_{ck}}}{f_y} = \frac{0.24\sqrt{20}}{415} = 0.2586\%$

Maximum tension steel $p_{\max} = 2.5\% > 1.22\%$

However the positive moment capacity is less than 50 % of the negative moment capacity at the same face. Thus it violates the ductility provision. Let us redesign the section as a doubly reinforced section such that the compression steel is at least 50 % of the tension steel at this section.

$$A_{st} = \frac{1.22 \times 350 \times 575}{100} = 2455 mm^2$$

$$A_{sc} = \frac{0.61 \times 350 \times 575}{100} = 1227 mm^2$$

Provide 5-28 diameter bars at the top face $A_{st} = 3078 mm^2$ and 3-28 diameter bars at the bottom face $A_{sc} = 1847 mm^2$

Shear force: Factored shear force = $V_u = 1.2 \times 325 KN = 390 KN$

Table 8.2.1 Shear strength of concrete for 1.22% tension steel

Sl No	p (%)	$\tau_c (MPa)$
1	1.0	0.62
2	1.22	0.664
3	1.25	0.67

Nominal shear stress $\tau_v = \frac{V_u}{bd} = \frac{390 \times 1000}{350 \times 575} = 1.9378 MPa < 2.8 MPa$

Let us choose 8 diameter – 4 legged stirrups. The spacing of stirrups is given by

Least of

i), $x = \frac{0.87 f_y A_{sv}}{(\tau_v - \tau_c) b} = \frac{0.87 \times 415 \times 4 \times 50}{(1.9378 - 0.664) 350} = 161.956 mm$

ii).. $\frac{d}{4} = \frac{575}{4} = 143.75 mm$

iii).. $8 \times 28\phi = 224 mm$

Let us provide 8mm-4 legged stirrups @ 125 mm c/c in a length equal to 2d or 1150 mm from the face of the beam column joint. The first stirrup can be provided at a distance of 25 mm from the face of the joint so that the remaining spacing is 1125 mm. It will require a total of 10 stirrups.

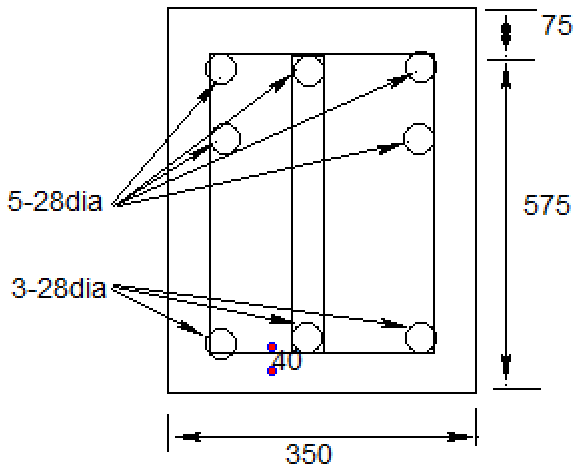


Figure 8.2.1 Details of doubly reinforced beam

Home Assignments

Q1: Design a rectangular RCC beam of 6m span supported on a RCC column to carry a point load of 100 KN in addition to its own weight. The moment due to seismic force is 5.01 KNm and shear force is 32 KN. Use M20 grade concrete and Fe 415 steel.

Q2 Compare the ductility with respect to curvature of the cross section of the beam of Figure 8.2.2 using a) M25 and Fe250 b) M25 and Fe415 c) M30 and Fe250 and d) M30 and Fe415

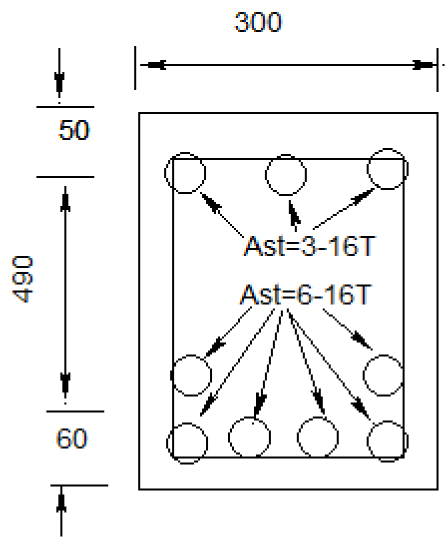


Figure 8.2.2 Details of doubly reinforced beam

Computation of earthquake forces on building frame using seismic coefficient method as per IS 1893-2002

In this lecture, the computation of earthquake forces on building frame using seismic coefficient method as per IS 1893-2002 has been discussed.

9.1 Seismic methods of design

After selecting the structural model, it is possible to perform analysis to determine the seismically induced forces in the structures. Linear static analysis or equivalent static analysis can be used for regular structures with limited height.

9.1.1 Equivalent lateral force method (seismic coefficient method)

Seismic analysis of most structures is still carried out on the assumption that the lateral (horizontal force) is equivalent to the actual (dynamic loading). This method requires less effort because except for the fundamental period, the periods and shapes of higher natural modes of vibration are not required. The base shear which is the total horizontal force on the structure is calculated on the basis of the structure's mass, its fundamental period of vibration and corresponding shape. The base end shear is distributed along the height of the structure in terms of lateral forces, according to the code formula. Planar models appropriate for each of the two analyses and the various effects including those due to torsional motions of the structure are combined. This method is usually conservative for low to medium height buildings with a regular configuration.

The factors taken into account in assessing lateral design forces and the design response spectrum are as follows:

- **Zone factor**

Seismic zoning assesses the maximum severity of shaking that is anticipated in a particular region. The zone factor (z) thus is defined as a factor to obtain the design spectrum depending on the perceived seismic hazard in the zone in which the structure is located. The basic zone factors included in the code are reasonable estimate of the effective peak ground acceleration. Zone factors as per IS 1893(part 1):2002 are given in Table 9.1

Table 9.1 Zone factor (z)

Seismic zone	II	III	IV	V
Seismic intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

- **Importance factor**

The importance factor is a factor used to obtain the design seismic force depending upon the functional use of the structure.

It is customary to recognise that certain categories of building use should be designed for greater levels of safety than the others and this is achieved by specifying higher lateral design forces. Such categories are:

a) Buildings which are essential after an earthquake: Buildings and fire stations etc
b) Places of assembly: schools, theatres etc
c) Structures the collapse of which may endanger lives- nuclear plants, dams etc

Table 9.2 shows importance factors for varying structures.

Table 9.2 Importance Factor for Structures

Structure	Importance factor
Important services and community buildings such as hospitals, schools, monumental structures, emergency buildings like telephone exchanges, television stations, radio stations, railway stations, fire station buildings, large community halls like cinemas, assembly halls and subway stations, power stations	1.5
All other buildings	1.0

- **Response reduction factor**

The basic principle of designing a structure for strong ground motion is that the structure should not collapse but damage to the structural elements is permitted. Such a structure is allowed to be damaged in case of severe shaking. The structure should be designed for seismic forces much less than what is expected under strong shaking if the structures were to remain linearly elastic. Response reduction factor (R) is the factor by which the actual base shear force should be reduced, to obtain the design lateral force. Base shear force is the force that would be generated if the structure were to remain elastic during its response to the design base earthquake (DBE) shaking. Table 9.3 gives response reduction factor for building systems.

Table 9.3 Response reduction factor for building systems

Lateral load resisting system	Response reduction factor (R)
Building frame systems Ordinary RCC moment resisting frame (OMRF)	3.0
Special RCC Moment resisting frame (SMRF)	5.0
Steel frame with concentric braces	4.0
Steel frame with eccentric braces	5.0
Steel moment resisting frame designed as per SP6	5.0
Buildings with shear walls load bearing masonry wall buildings a) unreinforced	1.5
b). Reinforced with horizontal RCC bands	2.5
c). Reinforced with horizontal RCC bands and vertical bars at corners of rooms and joints of openings	3.0
Ordinary RCC shear walls	3.0
Ductile shear walls buildings with dual systems	3.0
Ordinary shear wall with OMRF	4.0
Ordinary shear wall with SMRF	4.0
Ductile shear wall with OMRF	4.5
Ductile shear wall with SMRF	5.0

- **Fundamental natural period**

The fundamental natural period is the first (longest) modal time period of vibration of the structure- Because the design loading depends on the building period and the period can not be calculated until a design has been prepared. IS 1893 (part 1): 2002 provides formulae from which T_a may be calculated.

From a moment-resisting frame building without brick infill panels, T_a may be estimated by the empirical expressions

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

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

For all other buildings including moment-resisting frame buildings with brick infill panels. T_a may be estimated by the empirical expression

$T_a = \dfrac{0.09h}{\sqrt{d}}$ where h is the height of building in metres (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) and d is the base dimension of the building at the plinth level, in metres along the considered direction of the lateral force.

- **Design Response Spectrum**

The design response spectrum is a smooth response spectrum specifying the level of seismic resistance required for a design. Seismic analysis requires that the design spectrum be specified. IS 1893 (Part 1): 2002 stipulates a design acceleration spectrum or base shear coefficients as a function of natural period. These coefficients are ordinates of the acceleration spectrum divided by acceleration due to gravity. This relationship works well in SDOF systems. The spectral ordinates are used for the computation of inertia forces. Figure relates to the proposed 5 percent damping for rocky or hard soils sites and the following Table gives the multiplying factors for obtaining spectral values for various other damping (note that the multiplication is not to be done for zero period acceleration). The design spectrum ordinates are independent of the amounts of damping (multiplication factor of 1.0) and their variations from one material or one structural solution to another.

The static method is the simplest one. It requires less computational effort. First, the design base shear is computed for the whole building and it is then distributed along the height of the building. The lateral forces at each floor level thus obtained are distributed to individual lateral load resisting elements.

- **Seismic base shear**

The total design lateral force or design seismic base shear (V_b) along any principal direction is determined by

$V_B = A_h W$

Where A_h is the design horizontal acceleration spectrum value using the fundamental natural period, T, in the considered direction of vibration and W is the seismic weight of the building. The design horizontal seismic coefficient A_h for a structure is determined by the expression

$$A_h = \frac{ZIS_a}{2Rg}$$

For any structure with $T \leq 0.1s$, the value of A_h will not be taken less than Z/2 whatever be the value of I/R. In Equation (2), z is the zone factor as discussed previously, for the maximum considered earthquake (MCE). The factor 2 in the denominator is used so as to reduce the maximum considered earthquake (MCE) zone factor to the factor for design basis earthquake (DBE). I is the importance factor as discussed previously and depends upon the functional use of the structure, the hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance. R is the response reduction factor as discussed previously and depends on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. This factor is used to decide what other building materials are used, the type of construction and the type of lateral bracing system. $\frac{S_a}{g}$ is the response acceleration as defined previously for 5% damping based on appropriate natural periods. The curves of represent free-fluid ground motion. For other damping values of the structure, multiplying factors should be used. Table 9.4 shows response acceleration coefficient.

Table 9.4 Response acceleration coefficient $\frac{S_a}{g}$

For rocky or hard soil sites		
$\frac{S_a}{g}$	1+15T	$0.0 \le T \le 0.10$
	2.5	$0.10 \le T \le 0.40$
	1.0/T	$0.40 \le T \le 4.0$
For medium soil sites		
$\frac{S_a}{g}$	1+15T	$0.0 \le T \le 0.10$
	2.5	$0.1 \le T \le 0.50$
	1.36/T	$0.55 \le T \le 4.0$
For soft soil sites		
$\frac{S_a}{g}$	1+15T	$0.0 \le T \le 0.10$
	2.5	$0.1 \le T \le 0.67$
	1.67/T	$0.67 \le T \le 4.0$

- **Seismic weight**

The seismic weight of the whole building is the sum of the seismic weights of all the floors. The seismic weight of each floor is its full dead load plus the appropriate amount of imposed dead, the latter being that part of the imposed loads that may reasonably be expected to be attached to the structure at the time of earthquake shaking. It includes the weight of permanent and movable partitions, permanent equipment, a part of the live load etc. While computing the seismic weight of each floor, the weight of columns and walls in any storey should be equally distributed to the floors above and below the storey. Any weight supported in between storeys should be distributed to the floors above and below in inverse proportion to its distance from the floors.

As per IS1893(Part 1) the percentage of imposed load as given in the table below should be used. For calculating the design seismic forces of the structure, the imposed load on the roof need not be considered. Table 9.5 gives percentage of imposed load to be considered in seismic weight calculation.

Table 9.5 Percentage of imposed load to be considered in seismic weight calculation

Imposed uniformly distributed floor load (KN/m^2)	Percentage of imposed load
Upto and including 3.0	25
Above 3.0	50

- **Distribution of design force**

Buildings and their elements should be designed and constructed to resist the effects of design lateral force. The design lateral force is first computed for the building as a whole and then distributed to the various floor levels. The overall design seismic force thus obtained at each floor level is then distributed to individual lateral load resisting elements depending on the floor diaphragm action.

Vertical distribution of base shear to different floor levels. The design base shear (V_b) is 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 is the design lateral force at floor i, W_i is the seismic weight of the floor i, h is the height of floor i measured from the base and n is the number of storeys in the building i.e the number of levels at which the masses are located.

- Distribution of horizontal design lateral force to different lateral force resisting element

In the case of buildings in which floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane is distributed to the various vertical elements of the lateral force-resisting system, assuming the floors to be infinitely rigid in the horizontal plane. For buildings in which floor diaphragms can not be treated as infinitely rigid in their own plane, the lateral shear at each floor is distributed to the vertical elements resisting the lateral forces, accounting for the in-plane flexibility of the diaphragms.

Home Assignments

Q1: Discuss the factors required for assessing the lateral design forces

Q2: Write short notes on zone, response reduction and importance factors.

Computation of earthquake forces on building frame using seismic coefficient method as per IS 1893-2002: Numerical Problems

In this lecture, the seismic analysis of multi-storeyed RC buildings are carried out as per IS 1893 (Part 1): 2002

10.1 Problem: A four storeyed building as shown in Figure 10.1 is to be analyzed by the equivalent static method.

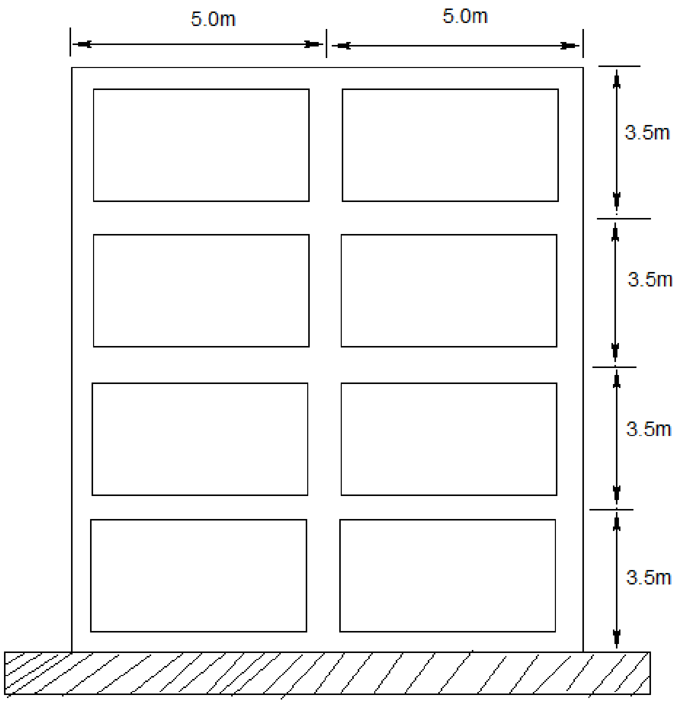


Figure 10.1.1 a: Plane frame structure (Four storeyed building)

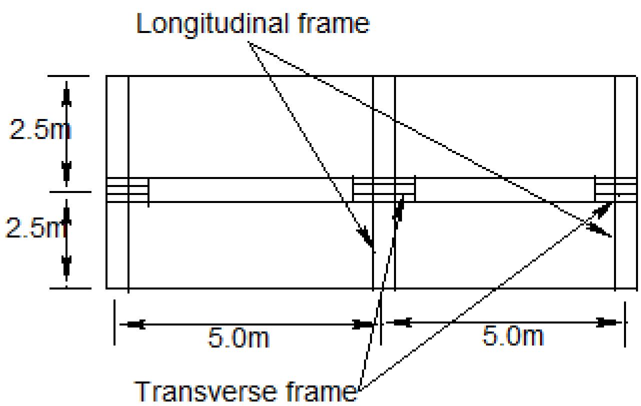


Figure 10.1.1 b: Plan showing the column and beams at floor levels of the plane frame

The following preliminary building data required for analysis is given

Type of structure : Multi-storey rigid jointed plane frame (Special RC moment resisting frame)

Seismic zone= IV (Clause 6.4.2, Table 2, IS 1893 (Part I): 2002)

Number of stories = Four, (G+3)

Floor height = 3.5 m

Infill wall = 250 mm thick including plaster in longitudinal and 150 mm in transverse direction

Imposed load = 3.5 KN/m^2

Materials = Concrete (M20) and Reinforcement (Fe415)

Size of columns = 250 mm x 450 mm

Size of beams= 250 mm x 400 mm in longitudinal and 250 mm x 350 mm in transverse direction

Depth of slab= 100 mm thick

Specific weight of RCC = 25 KN/m^3 (IS 875 (Part I)- 1987)

Specific weight of infill = 20 KN/m^3 (IS 875 (Part I)- 1987)

Type of soil = Rock

Response spectra = as per IS 1893 (Part I): 2002

Time history = Compatible to IS 1893 (Part I): 2002 spectra at rocky site for 5 % damping

Solution:

The earthquake forces are to be calculated as per IS 1893 (Part I): 2002. The imposed load on roof is nil.

Seismic weight of floors (Clause 7.4.1)

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

For Roof:

Mass of infill = (0.25mx10mx(3.5m/2)x20 KN/m^3+(0.15mx15mx(3.5m/2)x20 KN/m^3=87.5 KN+78.75 KN=166.25 KN

Mass of columns = 0.25mx0.45mx(3.5m/2)x3x25 KN/m^3=14.7656 KN

Mass of beams in longitudinal and transverse direction of that floor = 0.25mx0.40mx10mx25 KN/m^3+0.25mx0.35mx5mx3x25 KN/m^3=25 KN+32.8125 KN=57.8125 KN

Mass of slab = 0.10mx5mx10mx25 KN/m^3=125 KN

Imposed load of roof = 0 KN

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

Total load on the roof W4= 166.25 KN+14.7656 KN+57.8125 KN+ 125 KN + 0 KN=363.828 KN

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

For 1st Floor:

Mass of infill = (0.25mx10mx(3.5m/2)x2x20 KN/m^3+(0.15mx15mx(3.5m/2)x2x20 KN/m^3=2x87.5 KN+2x78.75 KN=2x166.25 KN=332.5 KN

Mass of columns = 0.25mx0.45mx(3.5m/2)x2x3x25 KN/m^3=2x14.7656 KN=29.5312 KN

Mass of beams in longitudinal and transverse direction of that floor = 0.25mx0.40mx10mx25 KN/m^3+0.25mx0.35mx5mx3x25 KN/m^3=25 KN+32.8125 KN=57.8125 KN

Mass of slab = 0.10mx5mx10mx25 KN/m^3=125 KN

Imposed load of roof = 5mx10 m x3.5 KN/m^2 x0.5 = 87.5 KN

Total load on the floor W1 = 332.5 KN+29.5312 KN+57.8125 KN+ 125 KN + 87.5 KN= 632.3437 KN

For 2nd Floor:

Load on 2nd Floor W2 = Load on 1st Floor W1 = 632.3437 KN

For 3rd Floor:

Load on 3rd Floor W3 = Load on 1st Floor W1 = 632.3437 KN

Seismic weight of building (Clause 7.4.2)

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

Seismic weight of all floors = W1 + W2 + W3 + W4 = 632.3437 KN + 632.3437 KN + 632.3437 KN + 363.828 KN = 2260.8591 KN

Clause 7.6.1 The approximate fundamental natural period of a vibration T_a in seconds of a moment resisting frame building without brick infill panels may be estimated by the empirical expression as

For Reinforced Concrete Frame building, $T_a = 0.075xh^{0.75} = 0.075x14^{0.75} = 0.5423\sec ond$ where h is the height of the building in metres.

Clause 7.5.3 The total design lateral force or design seismic base shear (V_B) along any principal direction shall be determined by the following expression:

Design seismic base shear is given by

$V_B = A_h W$ (W is the seismic weight of the building as per 7.4.2) (10.1.1)

$A_h = \frac{ZIS_a}{2Rg} = \frac{0.24x1x1.842}{2x5} = 0.0443$ (A_h is the design acceleration spectrum value as per Clause 6.4.2, IS 1893 (Part I): 2002 using the fundamental natural period T_a as per 7.6 in the considered direction of vibration) (10.1.2)

For $T_a = 0.5423\sec ond$, $\frac{S_a}{g} = \frac{1}{T_a} = \frac{1}{0.5423} = 1.842$ (10.1.3)

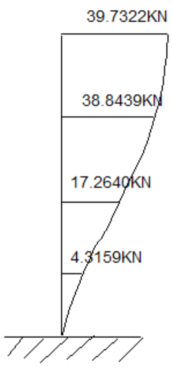
Hence Design base shear $V_B = 0.0443 \times 2260.8591KN = 100.156KN$ (10.1.4)

Table 10.1.1 Lateral load distribution with height

Storey level	$W_i(KN)$	$h_i(m)$	$W_i h_i^2$	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Lateral force at storey level Q_i
Roof	363.828	14	71310.288	0.396703	39.7322
3	632.3437	10.5	69715.89293	0.387834	38.8439
2	632.3437	7	30984.8413	0.172370	17.2640
1	632.3437	3.5	7746.210325	0.043093	4.315985

$\sum W_i h_i^2 = 179757.2326$ Checks: $\frac{W_i h_i^2}{\sum W_i h_i^2} = 1.0$

The loading and shear diagrams are as shown below



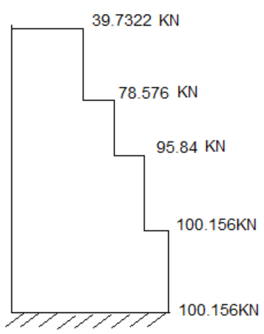


Figure 10.1.2 Loading diagram and Shear diagram

10.2 Problem: The plan and elevation of a three storey RCC school building is shown in Figure 10.2.2 below. The building is located in seismic zone V. The type of soil encountered is medium stiff and it is proposed to design the building with a special moment resisting frame. The intensity of dead load is 10 KN/m^2 and the floors are to cater to an imposed load of 3 KN/m^2. Determine the design seismic loads on the structure by static analysis.

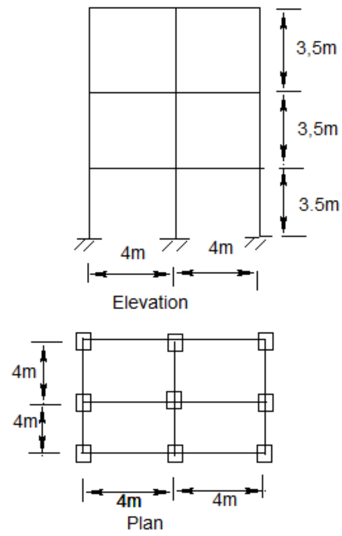


Figure 10.2.1 Plan and elevation of multi-storey building

Solution:

Design parameters:

For seismic zone V, zone factor z = 0.36 (10.2.1)

Importance factor I=1.5 (10.2.2)

Response reduction factor R=5 (10.2.3)

Seismic weight:

Floor area = 8 x 8 = 64 m^2

For live load upto and including 3 KN/m^2

Percentage of live load to be considered = 25 %

The total seismic weight on the floor is $W = \sum W_i$ where W_i is sum of loads from all the floors which includes dead loads and appropriate percentage of live loads

Seismic weight contribution from one floor = 64 x (10+0.25x3)= 688 KN

Load from roof = 64 x 10 = 640 KN

Hence, the total seismic weight of the structure = 2 x 688 +640 = 2016 KN

Fundamental natural period of vibration, T_a is given as

$$T_a = \frac{0.9h}{\sqrt{d}} \tag{10.2.4}$$

Where h is the height of the building in metres and d is the base dimension in metres at plinth level along the direction of the lateral loads

$$T_a = \frac{0.09 \times 10.5}{\sqrt{8}} = 0.334\,\mathrm{sec}ond$$

Since the building is symmetrical in plan, the fundamental natural period of vibration will be the same in both the directions

For medium stiff soil and $T_a = 0.334\,\mathrm{sec}ond \quad \frac{S_a}{g} = 2.5$

$$A_h = \frac{ZIS_a}{2Rg} = \frac{0.36 \times 1.5 \times 2.5}{2 \times 5} = 0.135 \tag{10.2.5}$$

Design base shear V_B is given as

$$V_B = A_h W = 0.135 \times 2016 = 272.16 KN \tag{10.2.6}$$

Table 10.2.1 Lateral load distribution with height

Storey level	$W_i(KN)$	$h_i(m)$	$W_i h_i^2$	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Lateral force at storey level Q_i
Roof	640	10.5	70560	0.626	170.37
2	688	7	33712	0.299	81.38
1	688	3.5	8428	0.0748	20.41

$\sum W_i h_i^2 = 112700$ Checks: $\frac{W_i h_i^2}{\sum W_i h_i^2} = 1.0$

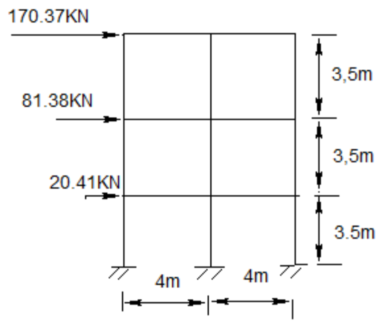


Figure 10.2.2 Design seismic forces by static analysis

Home Assignments

Q1: Plan of a five storey building is shown as below. Dead load including self weight of slab, finishes, partitions, etc can be assumed as 5 KN/m^2 and live load as 4 KN/m^2 on each floor and as 1.5 KN/m^2 on the roof. Determine the lateral forces and shears at different storey levels. Assume z=0.24, I=1, R=5, Soil type = 2, storey height = 3.5 m.

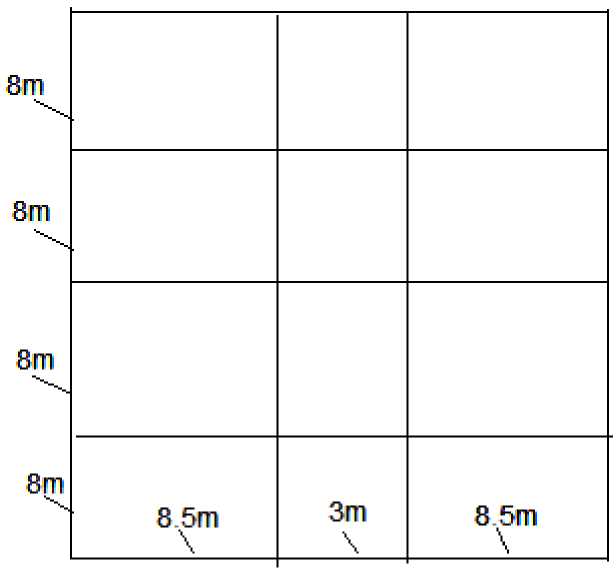


Figure Q1: Plan