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SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING

UNIT – I - Introduction – SCIA5301

INTRODUCTION

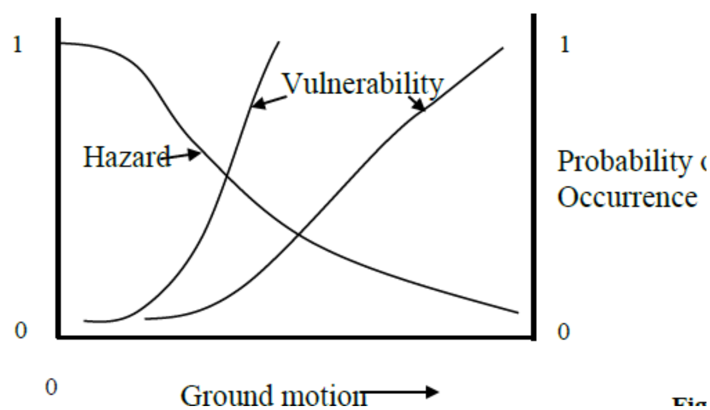
Engineering Seismology

Engineering Seismology is the study of Seismology as related to Engineering. This involves understanding the source, the size and the mechanisms of earthquakes, how the ground motion propagates from the source to the site of engineering importance, the characteristics of ground motion at the site and how the ground motion is evaluated for engineering design. This subject is therefore related to the hazard of earthquakes. The seismic hazard at a site cannot be controlled. It can only be assessed. In the same context, **Earthquake Engineering** is the subject of analysis and design of structures to resist stresses caused by the earthquake ground motion. Resisting the stresses imply either resisting without failure or yielding to the stresses gracefully without collapse. This subject is related to the vulnerability of built structures to seismic ground motion. The vulnerability is controlled by design. The decision to control the vulnerability of a structure is based on the economics of the situation and on the judgement about the acceptable risk to the community.

Therefore, the assessment of seismic risk is based on the seismic hazard, the vulnerability and the value of the loss. This is expressed by the relation:

Risk = Hazard * Vulnerability * Value.

In this context, "Seismic Hazard" is defined as the probability of occurrence of a ground motion of a given size within a given period of time at the site of interest. This will depend on the possible sources of earthquakes within a reasonable distance of the site and the seismic activity of these sources in relation to size and time. The "Vulnerability" is a measure of the probability of damage (loss) to the structure to a ground motion of a given size. Different structures have different vulnerability curves.



Fig

Seismicity and Plate tectonics

If we look at a map of seismic activity around the world, we notice that earthquakes predominantly happen along some belts. For example the Circum- Pacific belt (around the pacific ocean) or the Alpide belt starting at far east and follow all the way to Europe. However, we note that earthquakes do happen infrequently elsewhere. There is no place on the globe which can truly be said to be non-seismic. The fact of the seismic belts and other reasons led to the Plate Tectonic theory. [Continental Drift theory by Wegner in 1915] . According to this theory, the crust of the earth (more accurately the lithosphere which includes the crust and a small part of the upper mantle) is broken up into about 20 rigid plates. These plates are slowly moving relative to each other continuously. Sometimes, these movements are arrested by static friction and stresses build up. When the stress is sufficiently large it overcomes the friction and the plates suddenly move apart. This explains the reason behind the earthquakes along the boundaries of the plates. However, the plates are not entirely rigid and the boundary stresses may cause failure inside the plate as well. That is why, no part of the globe is safe against earthquakes. What causes the plates to move is only a hypothesis- the heat loss in the earth's interior causing convection current to develop in the mantle and the current carries the plates with it.

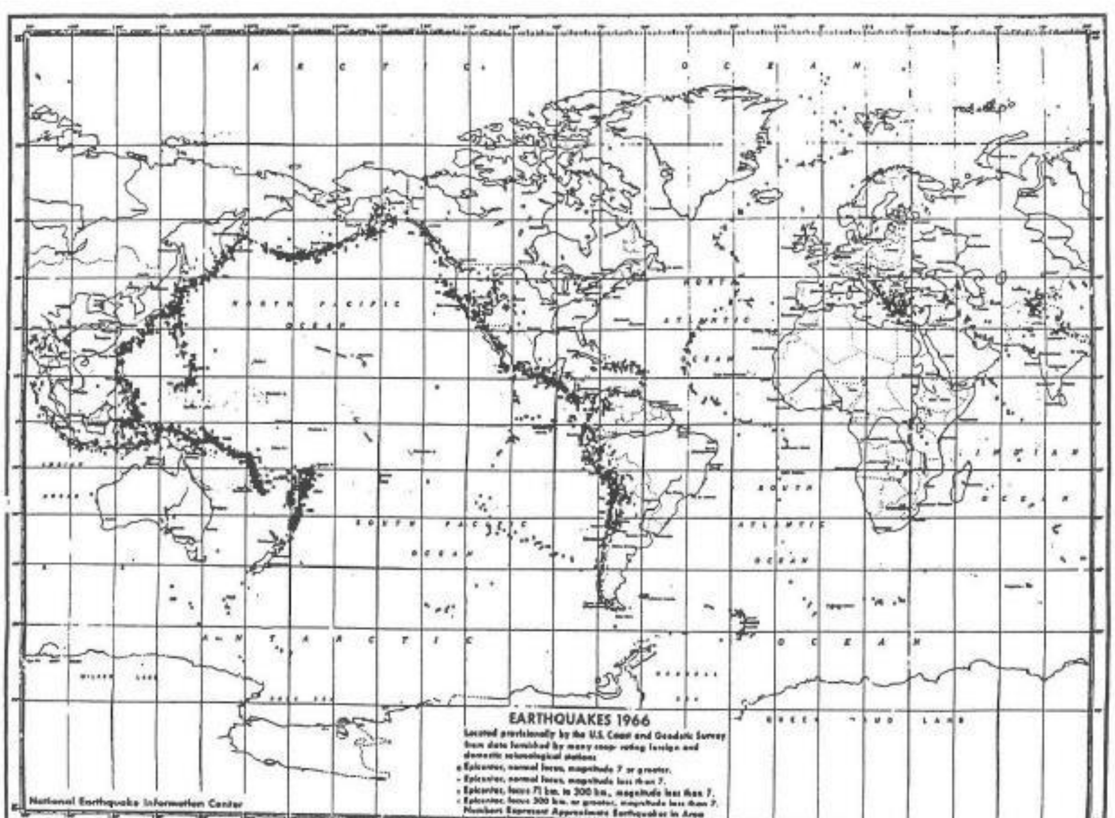


Fig. 12-2 Map showing global seismicity for the year 1966. The epicenters of the earthquakes were calculated by the U.S. Coast and Geodetic Survey from the observed travel times of seismic P-waves at the seismographic stations. The map can be considered almost complete for earthquakes with a magnitude above 4.[12-7]

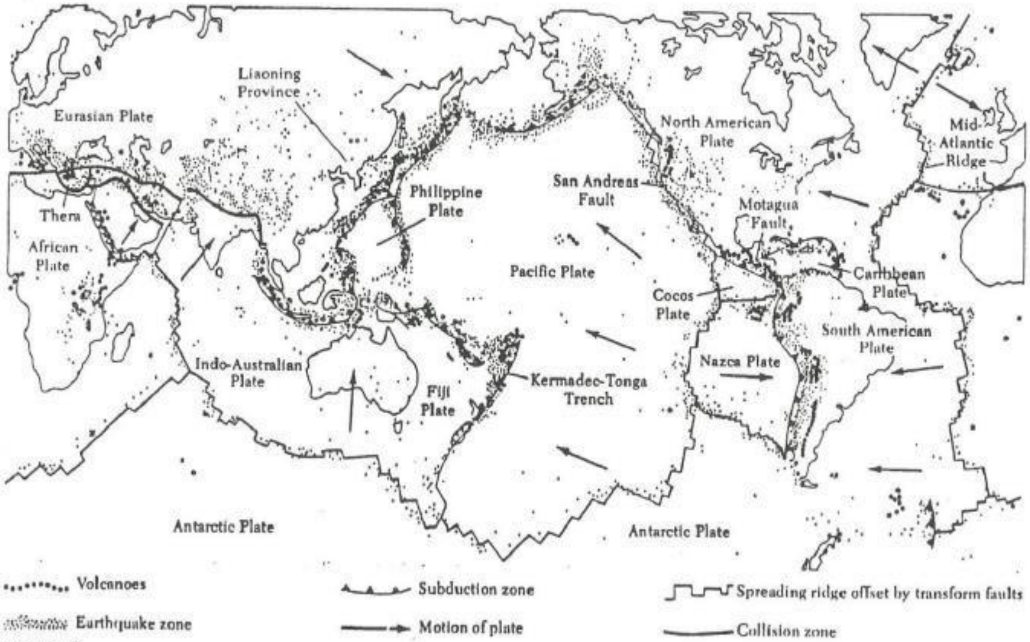
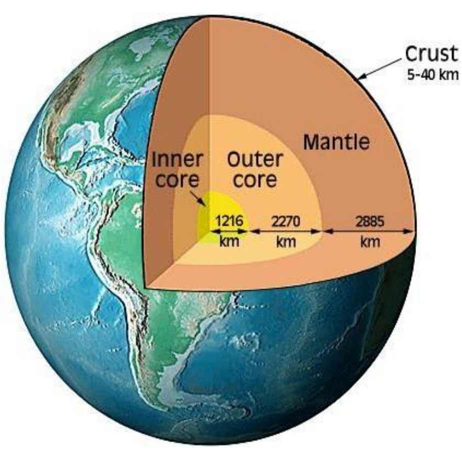


FIGURE 4
World map showing relation between the major tectonic plates and recent earthquakes and volcanoes. Earthquake epicenters are denoted by the small dots, and the volcanoes by large dots.

Layers of Earth

The earth is divided into four main layers: the inner core, outer core, mantle and crust

Earth's mass is in the mantle. It composed Fe, Mg, Al, Si, O_2 and other compounds. The temp is about 1000^{o} C, the mantle is solid but can deform slowly in a plastic manner . Crust -thin layers- composed of the least dense calcium (Ca) and sodium (Na) aluminum-silicate minerals. The crust is rocky and brittle, relatively cold so it can fracture in earthquakes. Earthquakes occur in the crust or upper mantle- ranges from earth's surface to 800 kilometers deep.



Hypocentre or Focus:

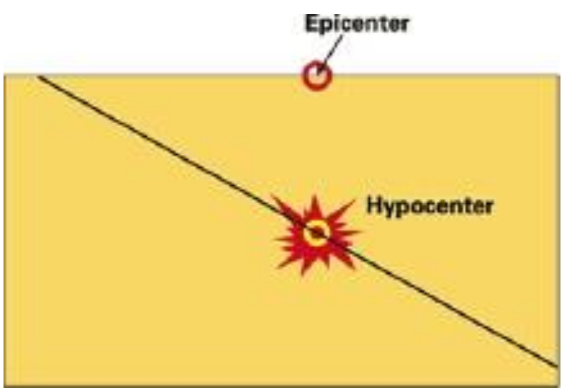
The point on the fault where slip is first originated. From this point, the slip propagates and spreads over the rupture surface (the fault) until the slip is stopped by either strong material or less stress. The hypocentre is represented by three coordinates: Latitude, Longitude and the depth from the earth's surface. Note that the whole fault does not move at the same instant.

Epicentre:

The point on the earth's surface immediately above the hypocentre. It is represented by the latitude and longitude of the point. The error in the determination of the epicentre is about 10km presently. But in the old days, this error could be very large. There are instances of the determination in the wrong hemisphere. It is therefore essential to correlate the instrumental determination of epicentre with the area of maximum damage.

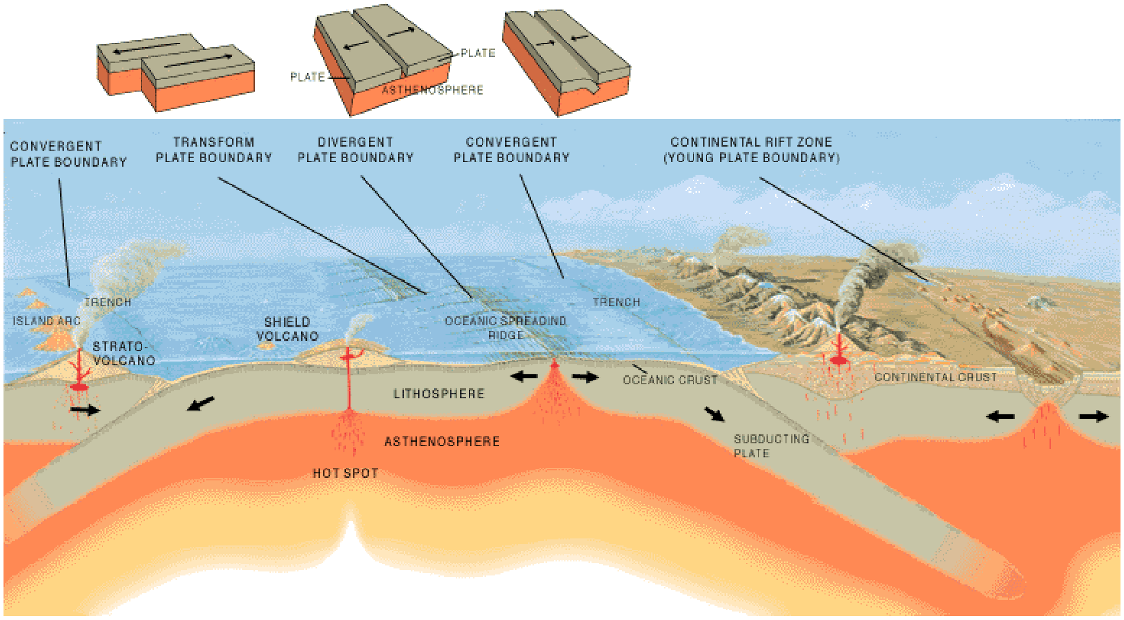
Focal depth:

This is the depth of focus below the epicentre. There are three grades of depth- Shallow, Intermediate and Deep. Most continental earthquakes are shallow and these are of engineering importance. Focal depth of an earthquake is the most difficult one to determine and should be treated with caution. In the bulletin of earthquakes, most earthquakes are given a focal depth of 33km which simply imply that these are of shallow depth but the depth was not possible to determine any more accurately.



<u>Types of Plate Boundries</u>

- Divergent (Tension)
- Convergent (Compression)
- Transform (shearing)



Types of waves

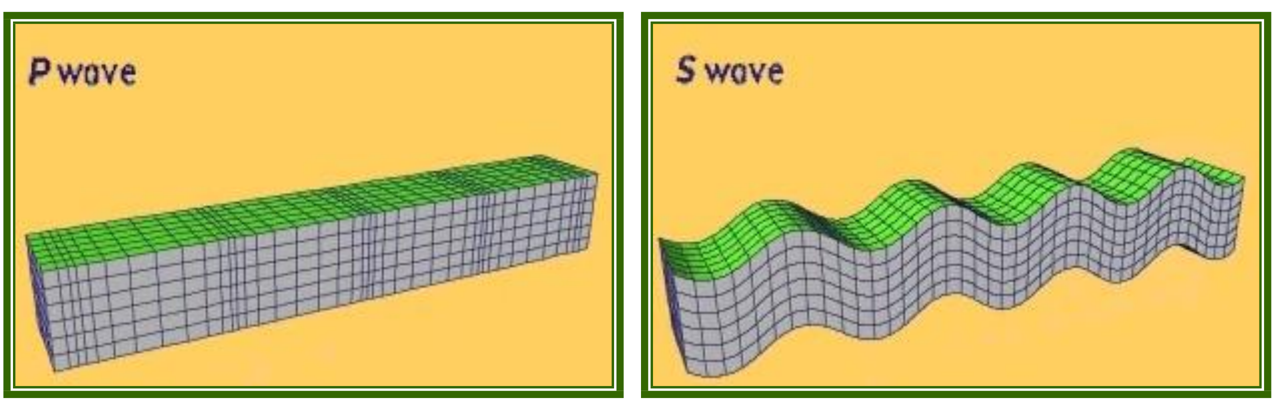
Earthquakes generate four principal types of waves; two, known as **body waves,** travel within the Earth, whereas the other two, called **surface waves**, travel along its surface

Primary, or P waves or compressional or longitudinal waves

P waves reach seismic recording station faster than the S waves. P waves push rock particles back-and-forth motion along the path of propagation, thus stretching or compressing the rock as the wave passes any one point; these waves are like sound waves in air.

Secondary, or S waves or transverse or shear waves

Rock particles to move back and forth perpendicular to the direction of propagation; as the wave passes, the rock is distorted first in one direction and then in another.

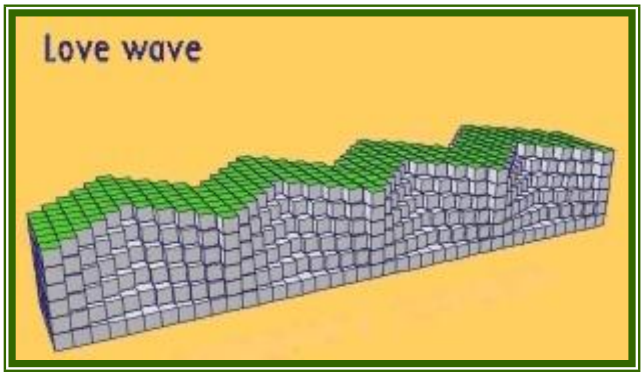


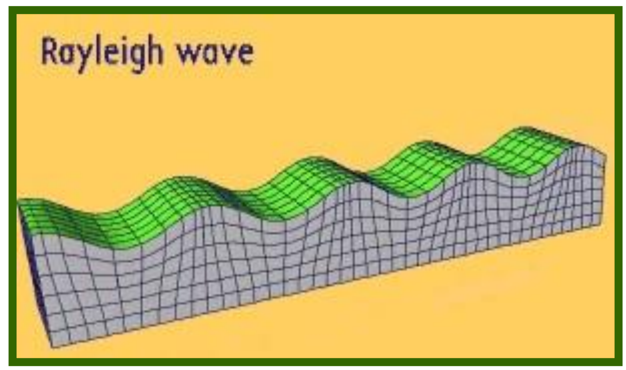
Love waves—named after A.E. Love, who first predicted their existence

Travel faster and are propagated in a surface layer that overlies a solid rock layer with different elastic properties. It is entirely perpendicular to the direction of propagation and has no vertical or longitudinal components. The energy of Love waves spreads from the source in two directions rather than in three, and so these waves produce a strong record at seismic stations

Rayleigh waves after Lord Rayleigh, who first predicted their existence.

Travel on the free surface of an elastic solid. It has longitudinal and vertical vibration that gives an elliptical motion to the rock particles. The displacements are greatest at the surface and decrease exponentially downward. It shows varying wavelength. This wave is similar to how ocean waves propagate.





<u>Other causes of earthquakes</u>

- Volcanic earthquakes
- Explosions (underground detonations of chemical or nuclear devices)
- Collapse earthquakes (roofs of mines and caverns)
- Reservoir induced earthquakes
- Impacts with extraterrestrial bodies (meteorites)

***Fault*:** sudden change in rock structure at contactbetween two tectonic blocks Cause relative slip between tectonic plates:

- slow slip, which produces no ground shaking
- sudden slip, that generates earthquakes

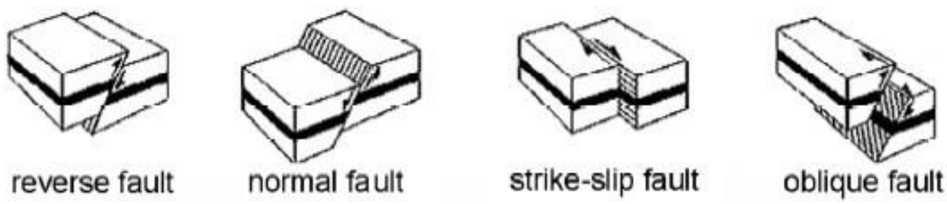
Fault types

Strike-slip fault: are vertical (or nearly vertical) fractureswhere the blocks have mostly moved horizontally

Normal fault: fractures where the blocks have mostly shifted vertically, while the rock mass above an inclinedfault moves down

Reverse fault: fractures where the blocks have mostly shifted vertically, while the rock above the fault moves up

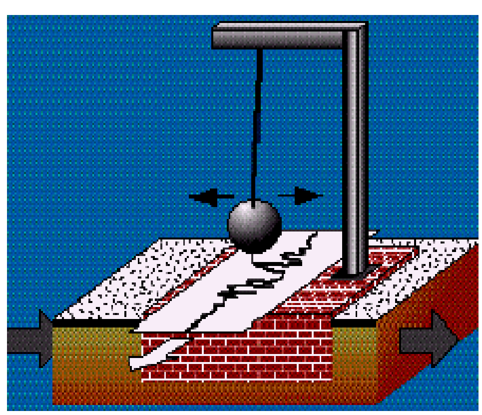
Oblique fault: the most general case, a combination of vertical and horizontal movement



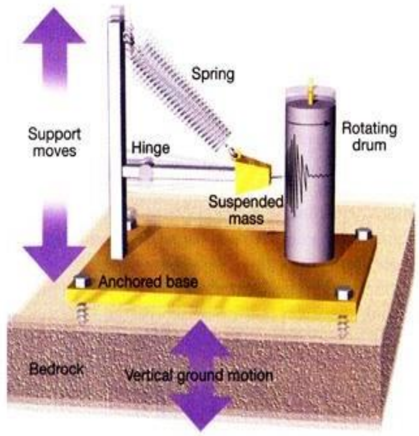
<u>Seismic Instrumentation</u>

The study of earthquake waves, Seismology, dates back almost 2000 years to the Chinese Seismographs, instruments that record seismic waves. The first seismograph called Di-Dong-Di was invented by Cheng Heng (132 A.D.).

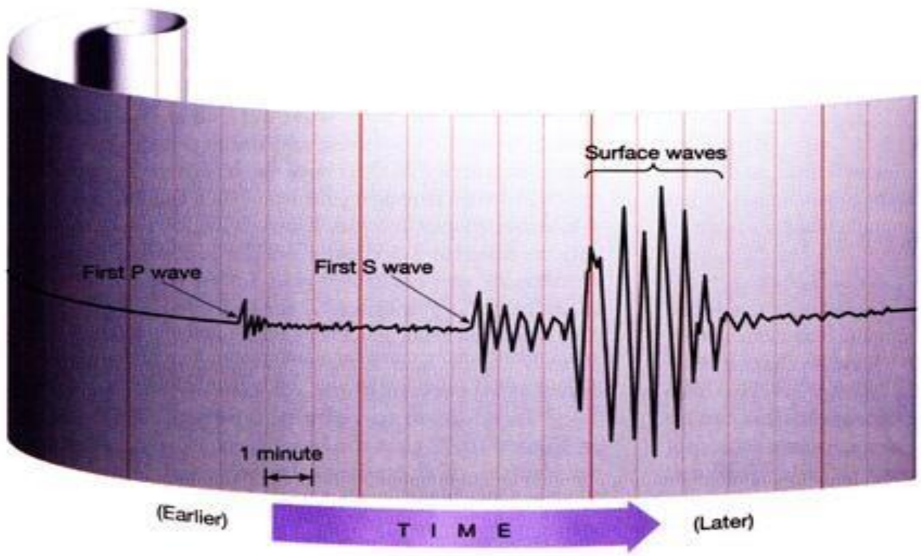
Pendulum-mounted: records horizontal motion.



Spring mounted: seismograph records vertical motion.



Modern seismogram



Magnitude:

Earthquake magnitude is measured in Richter scale magnitude, named after geophysicist Charles F. Richter of CIT, California, who developed in 1935. Magnitude = $Log_{10}(A_{max})$; Amax = max. amplitude in microns(10^{-3}m) recorded by a seismographThe magnitude number is assigned to an earthquake on the basis of amount of ground displacement or amount of strain energy released at the source, which is measured by a seismograph.It is a mathematical device to measure the size of earthquakes.i.e the logarithm of the amplitude of waves recorded on a seismogram at a certain period.The Richter Scale is not used to express damage.

Intensity:

It is a measure of the shaking and damage caused by the earthquake, and this value changes from location to location. Adjustments are included for the variation in the distance between the various seismographs and the epicenter of the earthquakes.

Measuring Earthquakes

Seismogram is visual record of arrival time and magnitude of shaking associated with seismic wave. Analysis of seismogram allows measurement of size of earthquake.

Richter Scale- (logarithmic scale)

- Magnitude- based on amplitude of the waves
- Related to earthquake total energy

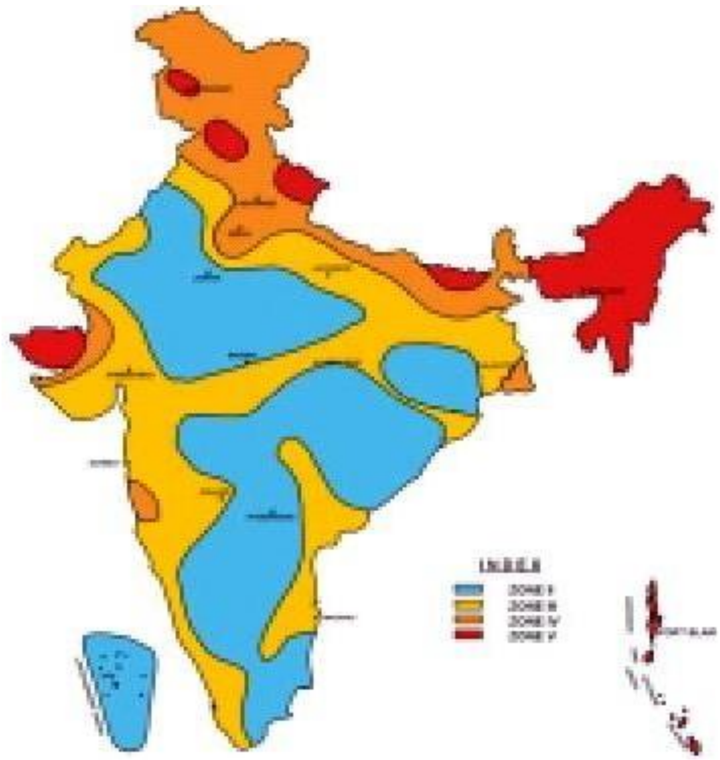
Mercalli Intensity scale

- Measured by the amount of damage caused in human terms- I (low) to XII (high); drawback: inefficient in uninhabited area
- Depends on distance to earthquake & strength of earthquake

Modified Mercalli Scale		Richter Magnitude Scale
I	Detected only by sensitive instruments	1.5
II	Felt by few persons at rest, especially on upper floors; delicately suspended objects may swing	2
III	Felt noticeably indoors, but not always recognized as earthquake; standing autos rock slightly, vibration like passing truck	2.5
IV	Felt indoors by many, outdoors by few, at night some may awaken; dishes, windows, doors disturbed; motor cars rock noticeably	3
V	Felt by most people; some breakage of dishes, windows, and plaster; disturbance of tall objects	3.5 4
VI	Felt by all, many frightened and run outdoors; falling plaster and chimneys, damage small	4.5
VII	Everybody runs outdoors; damage to buildings varies depending on quality of construction; noticed by drivers of automobiles	5
VIII	Panel walls thrown out of frames; fall of walls, monuments, chimneys; sand and mud ejected; drivers of autos disturbed	5.5
IX	Buildings shifted off foundations, cracked, thrown out of plumb; ground cracked; underground pipes broken	6 6.5
X	Most masonry and frame structures destroyed; ground cracked, rails bent, landslides	7
XI	Few structures remain standing; bridges destroyed, fissures in ground, pipes broken, landslides, rails bent	7.5
XII	Damage total; waves seen on ground surface, lines of sight and level distorted, objects thrown up into air	8

Seismic Zones of India:

Indian Sub-continent Four seismic zones with different level of ground shaking as per IS: 1893 – 2002



Zone II Zone III

Zone IV Zone V

RESPONSE SPECTRUM

A **response spectrum** is simply a plot of the peak or steady-state response (displacement, velocity or acceleration) of a series of oscillators of varying natural frequency, that are forced into motion by the same base vibration or shock. The resulting plot can then be used to pick off the response of any linear system, given its natural frequency of oscillation. One such use is in assessing the peak response of buildings to earthquakes. The science of strong ground motion may use some values from the ground response spectrum (calculated from recordings of surface ground motion from seismographs) for correlation with seismic damage.

If the input used in calculating a response spectrum is steady-state periodic, then the steady-state result is recorded. Damping must be present, or else the response will be infinite. For transient input (such as seismic ground motion), the peak response is reported. Some level of damping is generally assumed, but a value will be obtained even with no damping.

Response spectra can also be used in assessing the response of linear systems with multiple modes of oscillation (multi-degree of freedom systems), although they are only accurate for low levels of damping. Modal analysis is performed to identify the modes, and the response in that mode can be picked from the response spectrum. This peak response is then combined to estimate a total response. A typical combination method is the square root of the sum of the squares (SRSS) if the modal frequencies are not close. The result is typically different from that which would be calculated directly from an input, since phase information is lost in the process of generating the response spectrum.The main limitation of response spectra is that they are only universally applicable for linear systems. Response spectra can be generated for non-linear systems, but are only applicable to systems with the same non-linearity, although attempts have been made to develop non- linear seismic design spectra with wider structural application. It should be noted that the results cannot be directly combined for multi-mode response.

Response spectra are very useful tools of earthquake engineering for analyzing the performance of structures and equipment in earthquakes, since many behave principally as simple oscillators (also known as single degree of freedom systems. Thus, if you can find out the natural frequency of the structure, then the peak response of the building can be estimated by reading the value from the ground response spectrum for the appropriate frequency. In most building codes in seismic regions, this value forms the basis for calculating the forces that a structure must be designed to resist (seismic analysis).As mentioned earlier, the ground response spectrum is the response plot done at the free surface of the earth. Significant seismic damage may occur if the building response is 'in tune' with components of the ground motion (resonance), which may be identified from the response spectrum. This was observed in the 1985 Mexico City Earthquake [1] where the oscillation of the deep-soil lake bed was similar to the natural frequency of mid- rise concrete buildings, causing significant damage. Shorter (stiffer) and taller (more flexible) buildings suffered less damage.

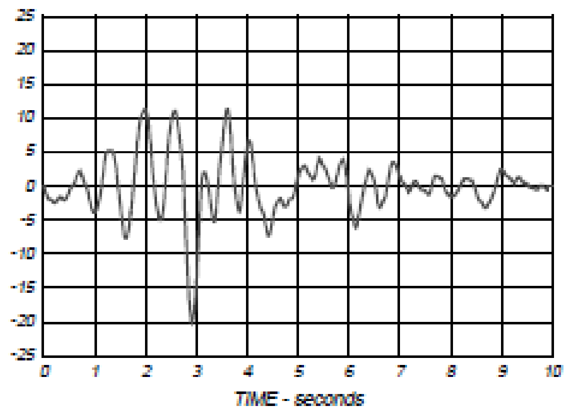
In 1941 at Caltech, George W. Housner began to publish calculations of response spectra from accelerographs[2]. In the 1982 EERI Monograph on "Earthquake Design and Spectra"[3], Newmark and Hall describe how they developed an "idealized" seismic response spectrum based on a range of response spectra generated for available earthquake records. This was then further developed into a design response spectrum for use in structural design, and this basic form (with some modifications) is now the basis for structural design in seismic regions throughout the world (typically plotted against structural "period", the inverse of frequency). A nominal level of damping is assumed (5% of critical damping).

For "regular" low-rise buildings, the structural response to earthquakes is characterized by the fundamental mode (a "waving" back-and-forth), and most building codes permit design forces to be calculated from the design spectrum on the basis of that frequency, but for more complex structures, combination of the results for many modes (calculated through modal analysis) is often required. In extreme cases, where structures are either too irregular, too tall or of significance to a community in disaster response, the response spectrum approach is no longer appropriate, and more complex analysis is required, such as non-linear static or dynamic analysis like in seismic performance analysis technique.

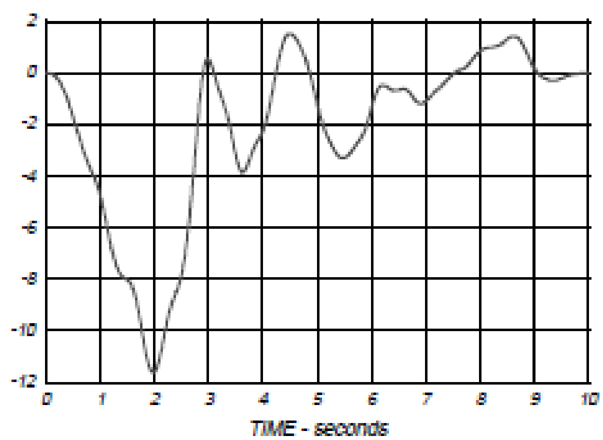
For three dimensional seismic motion, the typical modal Equation is rewrittenas

$$\ddot{y}(t)_n + 2\zeta_n\omega_n\dot{y}(t)_n + \omega_n^2 y(t)_n =$$
$$p_{nx}\ddot{u}(t)_{gx} + p_{ny}\ddot{u}(t)_{gy} + p_{nz}\ddot{u}(t)_{gz}$$

Typical Earthquake Ground Acceleration - Percent of Gravity



Typical Earthquake Ground Displacements - Inches



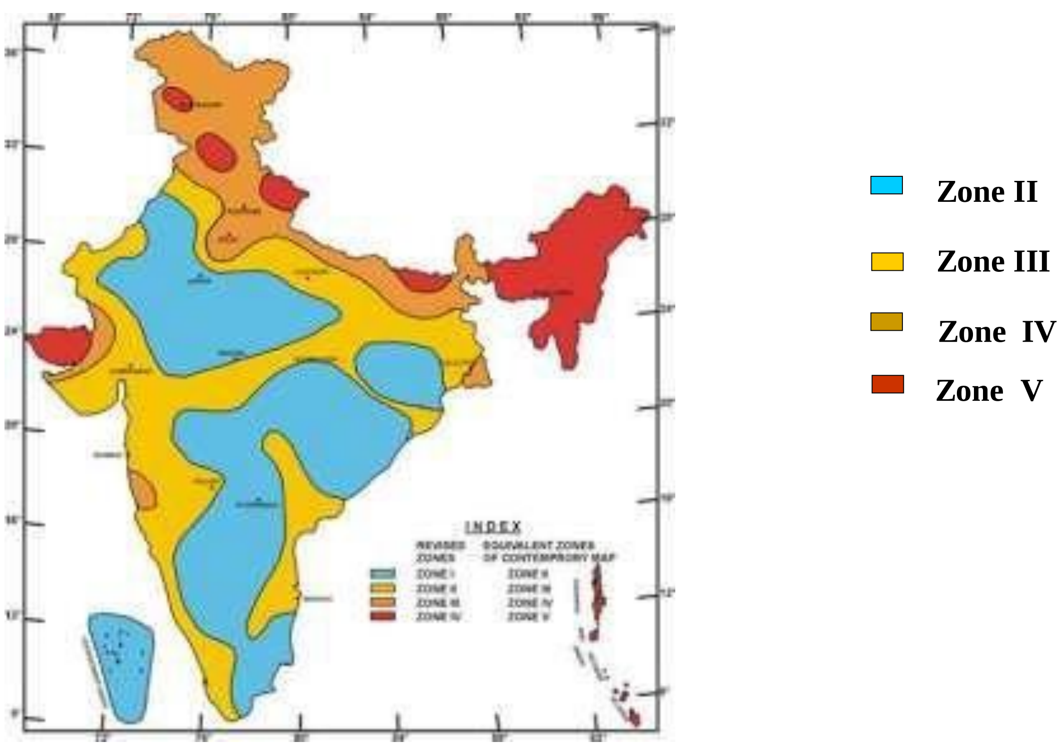
SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING

UNIT – II - Earthquake History– SCIA5301

EARTHQUAKE HISTORY

Indian Seismicity

Seismicity is defined as the distribution of seismic activity in time, location, magnitude & depth during the historical & recent instrumented period. Studies of seismicity are of great importance to understand the dynamic behaviour of the earth and is useful to determine the earthquake hazard in a specific region. Earthquakes are known to have occurred in the region of the Indian subcontinent from ancient times. Brief references are made to the seismic phenomena during the medieval period but it is not until the start of the colonial era from when clearer records of earthquakes emerge. Though incomplete in some respects, these provide a good overall summary of the distribution of earthquakes in the last 200 years. Most of the activity, including many "great" earthquakes have occurred in the northern subcontinent and in the Andaman & Nicobar Archipelago. The southern peninsula has suffered damaging earthquakes but less frequently as in the north. Indian Sub-continent Four seismic zones with different level of ground shaking as per IS: 1893 – 2002



Earthquakes In India

Earthquakes of Magnitude >8

– *1819 Cutch Earthquake (M8.3)*
– *1897 Assam Earthquake (M8.7)*
– *1905 Kangra Earthquake (M8.6)*
– *1934 Bihar-Nepal Earthquake (M8.4)*
– *1950 Assam Earthquake (M8.7)*

1819 Cutch Earthquake

- West coast of India
- Away from plate boundaries
- Perhaps strongest intra-plate event in world
- Motion perceived even at Calcutta
- First clear occurrence of faulting during
- earthquakes
- Allah Bund : Fault scarp ~100 km long ~3m high
- Low population area
- Death toll ~1500 lives

1897 Assam Earthquake

- Amongst greatest earthquakes of world
- Magnitude 8.7
- Mean radius of perception : 900 miles
- Mean radius of area of serious damage: 300 miles
- Longest dimension of meizoseismal area: 160 miles
- Chendarang fault
- 12 miles long, throws up to 35ft
- Surface distortion
- Upthrow of objects
- Liquefaction in alluvial plain of Brahmputra
- Effects in meizoseismal area provided model for
- Modified Mercalli Intensity XII

1905 Kangra Earthquake

- 4 April 1905
- Magnitude 8.6
- About 19,000 lives lost
- Very low population density

- Maximum Intensity X around Kangra
    - Intensity at Dehradun VIII
    - Intensity between Kangra and Dehradun up to VI/VII
    - Initially thought of as two different earthquakes

1934 Bihar-Nepal Earthquake

- 15 January 1934
    - Around 2:13pm
- Deaths
    - 7253 in India and 3400 in Nepal
- Magnitude 8.4
- Maximum intensity X in about 80 □20 miles
    - Intensity X also at Munger and in Kathmandu
- Valley (about 100 miles from main damage area)
- Slump Belt
    - 190 mile long, up to 40 miles wide
    - Excessive liquefaction
    - Buildings slumped into alluvium
    - Subsidence of embankments (roads/rails)
    - Uplift of bottoms in tanks
    - Fissures / emissions of sand and water
- one fissure : 15’ deep, 30’ wide, 900’ long!

1950 Assam-Tibet Earthquake

- Magnitude 8.7
- Epicenter near Rima (Tibet)
- Maximum intensity XII
- Aftershocks M 7.0
    - More property loss in Assam than in 1897 earthquake
- Massive landslides
    - Blockade of rivers
    - Later, led to floods as dams burst one by one

1988 Bihar-Nepal Earthquake

- 21 August 1988 at 4:39am
- Magnitude 6.6

- Maximum Intensity VIII
- Deaths: 1004; Injuries: 16000
- Summer time; Most people outdoors
- Same damage trend in Munger and
- Kathmandu as in 1934 earthquake
- Damage to buildings and bridges
- Shaking induced
- Liquefaction
- Nominal
- Damage to embankments
- Damage in Darjeeling and Sikkim

1991 Uttarkashi Earthquake

- 20 October 1991 at 2:52am
- Winter morning; most people indoors
- Magnitude 6.6
- Maximum intensity ~ VIII+
- 768 dead; 5,066 injured
- Very low population density in the area
- Excellent strong motion records
- Peak ground acceleration ~ 0.3g
- Collapse of a modern and vital bridge
- Stone walls and RC roof
- Total collapses
- Major loss of life

1993 Killari Earthquake

- Magnitude 6.4
- Maximum Intensity VIII-IX
- Death toll ~10,000
- Up to 35% in some villages
- Earlier estimates up to 30,000
- Surface rupture

- Intra-plate earthquake
- Located in Seismic Zone I of the prevalent
- zone map!
- Damage in a limited area 20 km □20 km
- Major cause of casualty in houses

2001 Bhuj Earthquake

- Magnitude 7.7
- Maximum MSK Intensity X
- Bhuj in Seismic Zone V of Indian seismic map
- 8.46 am on 26 January 2001
- ~13,805 dead; 1,67,000 injured
- 300,000 houses destroyed; 700,000 houses damaged
- Numerous multistorey RC buildingscollapsed
- 130 such buildings collapsed in Ahmedabad ~225km from epicenter

2004 Indian Ocean earthquake

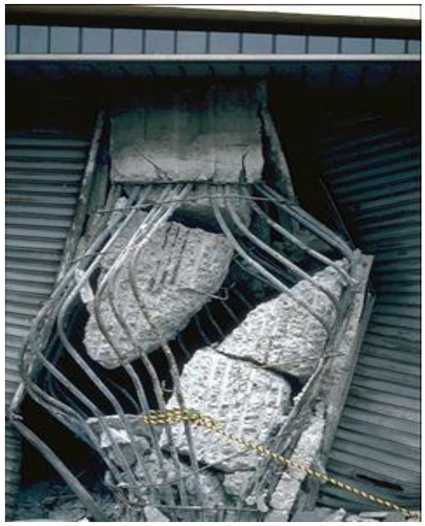
- Occurred on 26 December
- The epicentre off the west coast of Sumatra, Indonesia
- Magnitude of 9.1–9.3
- Maximum Mercalli intensity of IX (*Violent*)
- Killed 230,000 people in 14 countries
- One of the deadliest natural disasters in recorded history.
- Triggered a series of devastating tsunamis along the coasts of most landmasses

<u>Failures of RC structures During Earthquakes</u>

Column failure

- too few links in a load bearing column.
- Column failed in shear due to the shaking of an earthquake.
- Link at bottom and top have kept the reinforcement in shape at these points and so stopped failure.

- However where the links have failed, most likely due to lack of suitable anchorage, the vertical reinforcement bars have been able to twist and fail.
- In technical terms this is because the effective lengths of the bars has been increased past their design strengths.
- This failure is in general **inadequate** detailing in the design stage



Collapse of 'Soft' Storey

The above collapse is the result of a "soft" storey. Part of the left hand side of the building has collapsed, probably due to either a stiffness difference, which has resulted in the collapse of the whole floor and thus the failure of the building which will have to be demolished.

In the above, the left hand column on the bottom floor has failed. This has led to the collapse of the whole building, and the annex behind. The failure has most likely occurred this way because of different stiffness ratios between the bottom and next layers. These stiffness ratios differ because the restaurant had a glass front, to make it look attractive, which could not compete with the concrete in terms of strength properties. This then resulted in the formation of a stress concentration in the column which was above its capacity, thus the failure and subsequent collapse.

Stress Concentrations

The above building has failed because the stiffness factor has changed at the corner between the two parts of the building. This has caused stress concentrations at this point which when shaken has resulted in the failure.

From this initial point of failure the column and then the left hand side of the building have collapsed. The easiest way to combat this is to keep the design of the building simple, even box like. However this does not create the most visually impressive designs

Liquefaction

Bridge has failed because the right hand column has shifted causing the span to simply fall to the ground.

The right hand column has shifted because the ground got "liquified" during earthquake. This occurs because the soil vibrates to such an extent that the water in it is forced to the top causing the soil to act as a liquid.I.e sudden reduction in the shear strength of the soil. This liquid is then not stiff enough to withstand the forces on it from the columns foundations and so moves causing the failure.

<u>Lessons Learnt From Past Earthquakes</u>

1. Avoid unnecessary mass. Achieve a uniform distribution of mass.

2. Preserve symmetry and Avoid significant torsional motions.

3. Simple a structural system. Make sure there is a complete load path.

4. Use redundant structural system. Use additional structural system wherever possible.

5. Structure should be compact and regular in both plan and elevation. Avoid structures with elongated or irregular plans; having substantial setbacks in elevation (slenderness).

6. Use a uniform and continuous distribution of stiffness and strength. Avoid nonstructural components that unintentionally affect this distribution. Avoid sudden changes in member sizes or details.

7. Permit inelastic action (damage) only in inherently non-critical ductile elements (i.e., in beams rather than columns).

8. Detail the members to avoid premature (or) brittle failure modes. Utilise capacity design principles to avoid undesired shear, axial or joint failures and to foster ductile flexural failure

modes.

9. Avoid hammering (pounding) of adjacent structures.

10. Tie all structural components together. Anchor nonstructural components to structure to avoid falling hazards.

11. Avoid systems with low viscous damping. Absence of non-structural components tied to structure may be indication of low damping in steel structures

SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – III - DESIGN CONCEPTS AND BEHAVIOUR OF STRUCTURES – SCIA5301

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DESIGN CONCEPTS AND BEHAVIOUR OF STRUCTURES

Guidelines for earthquake resistant design

The following factors should be considered while designing a building against earthquake.

Drift:

It is the maximum lateral displacement of the structure with respect to total height or relative inter-storey displacement. The overall drifts index is the ratio of maximum roof displacement to the height of the structure and inter-storey drift is the ratio of maximum difference of lateral displacement at top and bottom of the storey divided by the storey height.

Non structural elements and structural non seismic members primarily get damaged due to drift. Higher the lateral stiffness lesser is the likely damage. The storey drift in any storey due to minimum specified design lateral force with partial safety factor of unity shall not exceed 0.004 times the storey height.

Separation between adjacent units or buildings:

Two adjacent buildings or two adjacent units of the same building with separation joint in between shall be separated by distance equal to the amount R times the sum of the calculated storey displacements as specified above of each of them to avoid damaging contact when the two units deflect towards each other.

Soft storey:

Soft storey or flexible storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three storeys above. In case of buildings with a flexible storey such as ground storey consisting of open spaces for parking

i.e. stilt buildings, special arrangements are need to be made to increase the lateral strength and stiffness of the soft storey.

For such buildings, dynamic analysis is carried out including the strength and stiffness effects of infills and inelastic deformations in the members particularly those in the soft storey and members designed accordingly. Alternatively, the following design criteria are to be adopted after carrying the earthquake analysis neglecting the effect of infill walls in other storeys.

When the floor levels of two similar adjacent buildings are at the same elevation levels, factor R can be taken as R/2.

a) The columns and beams of the soft storey are to be designed for 2.5 times the storey shear and moments calculated under seismic loads specified.

b) Besides the columns designed and detailed for calculated storey shears and moments, shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible to be designed exclusively for 1.25 times the lateral storey shear calculated.

Foundation:

The use of foundations vulnerable to significant differential settlement due to ground shaking shall be avoided for structures in seismic zones-III, IV & V. individual spread footings or pile caps shall be interconnected with ties except when individual spread footings are directly supported on rock. All ties shall be capable of carrying in tension and in compression an axial force equal to Ah/A times the larger of the column or pile cap load in addition to the otherwise computed forces where Ah is the design horizontal spectrum value.

Projections:

a) Vertical projections:

Tanks, towers parapets, chimneys and other vertical cantilever projections attached to buildings and projecting the above roof shall be designed and checked for stability for 5 times the design horizontal seismic co-efficient Ah. In the analysis of the building, the weight of these projecting elements will be lumped with the roof weight.

b) Horizontal projections:

All horizontal projections like cornices and balconies shall be designed and checked for stability for 5 times the design vertical co- efficient equal to 10/3 Ah. These increased design forces either for vertical projection or horizontal projection are only for designing the projecting parts and their connection with the main structures.

This means that for the design of main structure such increase need not to be considered.

Shape of the building:

Very slender buildings should be avoided. Large overhangs and projections attract large earthquake forces. Heavy masses like large water tanks, etc., at the top shall be avoided. Small water tanks, if provided, should be properly connected with the framing system. Building should be sufficiently be away from steep slopes. It should be built on filled up soil.

Asymmetry should be avoided as they undergo torsion and extreme corners are subjected to very large earthquake forces.

Damping:

Damping is the removal of kinetic energy and potential energy from a vibrating structure and by virtue of which the amplitude of vibration diminishes steadily. Some vibrations are due to initial displacement or initial velocity. Due to damping, these vibrations decay in amplitude.

1. When there is harmonic applied force and its period is nearly equal to the natural period of the structure. The vibration will grow from zero displacement and velocity. Damping limits the vibration maximum amplitude.
2. More damping less is the amplitude.
3. Negative damping may arise while the vibration is small, followed by positive damping at large amplitude vibrations. The code adopted for design of multistoried buildings considering seismic forces is IS 1893 (part I) – 2002. more than 60% area of India is earthquake prone. According to IS 1893 (part I) – 2002, India is divided into several zones to their magnitude of intensities.

<u>Philosophy and principle of earthquake resistant design</u>

It should be clear that earthquakes result in very high lateral forces on structures. it will be uneconomical to design all the buildings for such high earthquake forces. As earthquakes are rare events, the IS: 1893 outlines the philosophy of earthquake resistant design that the building should not have any significant structural damage under moderate earthquakes, which are relatively frequent. On the other hand, under a major earthquake, which is rare (average return period being 1000 years or more), the building may undergo severe damage, but it should not collapse in any case, as collapse results in large scale loss of life.

To avoid collapse of buildings during earthquakes there are four basic principles: (i) Reduced mass; (ii) Symmetry and Continuity of construction, (iii) Strength and overstrength and (iv) Ductility of structure. There are functional limits on reduction of mass, but it is obvious that a light weight structure will attract less force compared to a heavy structure. Seismic performance of a symmetric and regular structure has been observed to be much better than the asymmetric and irregular structures. The common types of irregularities found in buildings are described in the following sections. Role of strength and overstrength in resisting the inertia forces due to earthquakes is obvious. Overstrength is that part of the strength of the structure, which is not explicitly estimated in the design process and considered as a reserve strength. It arises due to higher material strength, strain hardening, strength increase due to strain rate effect, member oversize, provided reinforcement more than required, codal minimum requirements, effect of non-structural elements and redundancy etc. The role of ductility in resisting the earthquakes is not that obvious to common sense. It can be visualized by considering the earthquake ground motion as an energy imparted to the structure, which is to be dissipated by the structure. Ductility is the property of the structure which helps in energy dissipation without excessive damage or collapse of the structure. This is being dealt with in detail in the following sections.

PRINCIPLES

- P1 - Design codes should not hinder progress in design and construction methods.
- P2 - Design codes should reflect present knowledge. Code s should include a variety of design procedures making it possible to adjust the accuracy and difficulty of the procedures to the importance of the prob lems and the technical development of the societies concern. (7).
- P3 - Structural design should be viewed as a part of the broader activity of planning. Interdependence of general planning, structural design, construction methods and use should be taken into account (8).
- P4 - Structural reliability should be discussed as a socio-economic problem. Points of view of authorities, owner, designer, builder, and user should be considered and harmonized.

- P5 - Owing to the difficulty and shortage of the information required to base design on optimal decision rules minimizing generalized costs (or maximizing utilities), it is accepted that safety be checked with respect to a set of limit states. These should be defined and classified according to the types of the structural behaviour of and resulting damage in the structure ((1) and (2)). Classification based on the intensity of the actions is unsuitable (5).

- P6 - Design should be based on limiting the probability of different limit states (ultimate and serviceability) being surpassed. This prob-ability should refer: for a ultimate limit state, to a single occurrence of this; for serviceability limit states,to the limit state being surpassed during given intervals of time (or fractions of the reference period). In particular cases serviceability limit states may refer to very short durations, which can be assimilated to single occurrences (1).
- P7 - Defining the safety of a structure in a given site should be viewed as a single problem, not to be separately dealt with for each type of action and each type of member ((1), (4) and (5)).
- P8 - The range of applicability of safety parameters indicated in codes (partial factors, /' ; reliability indices, A ; probabilities of failures, Pf) should be clearly stated and should cover usual situations. Refinements of these parameters to take into account: cost of failure, attenuation cost, material and workmanship control, loads control, design accuracy, etc., should make possible an undistorted consideration of these different aspects without disregarding the most important ones (3).

Cyclic Load Behaviour Of RCC Element

The assessment of the behaviour of engineering structures under complex loading requires the utilisation of both experimental and analytical investigation tools. Whilst final c o n f i r m a t i o n o f ne w concept s and approaches to design require experimental verification, the role of analysis is central to the process of reaching the point of confirmation. This is mainly due to the impracticality of investigating structural performance in a parametric fashion using labour and capital-intensive testing procedures, especially under dynamic loading. Therefore, development of advanced analytical model-ling techniques and highly controlled testing methods should go in tandem. Whereas analytical procedures for the study of structures and components in the vicinity of their yield limit state have reached a satisfactory level of development, accurate analysis up to structural collapse is still an active research area, particularly in earthquake engineering, as discussed further herein.

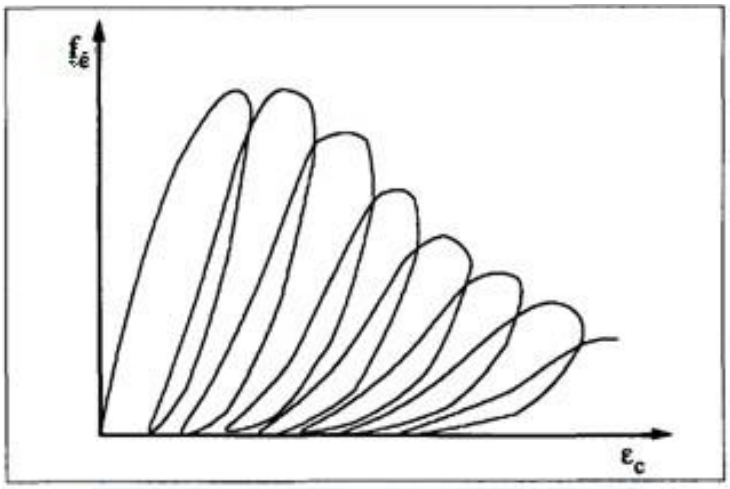
To enable the evaluation of ductility demand, in order to provide the necessary supply, accurate model ling of critical (plastic hinge) zones up to very large deformations is essential. This imposes extremely taxing requirements on analytical investigation tools, both on the local and the global structural levels. In reinforced concrete analysis, the subject of this paper, the fundamental requirements for use of an analysis model in earthquake applications are as follows:

Provide accurate estimates of stiffness and strength under cyclic loading at aW strain level. This is necessary for accurate evaluation of lateral response and damage accumulation modelling.

- Exhibit stability and accuracy at very high strain levels. This is required for the calculation of the collapse load of the overall structure, which may correspond to one or more zones reaching very high levels of deformation.
- Account accurately for the effect of confinement. This is required for the distinction between detailing of critical and non-critical zones.

The above characteristics have been the subject of extensive research in the past two decades or more, as reflected by the number of research publications on constitutive relationships for concrete [1]. However, models applicable to variable amplitude cyclic loading, hence dynamic loading, are rather scarce.

Uniaxial tests carried out on concrete short columns have allowed the observation of strength and stiffness degradation n o f concrete e due to *cyclic* loading. Fig shows a typical stress-strain curve for concrete under cyclic loading.



Typical stress-strain curve for concrete under cyclic loading.

<u>Cyclic Load Behaviour Of Steel Element</u>

The elastic and inelastic behavior of steel frames under the action of constant gravity loads and cyclic horizontal loads. An understanding of the inelastic behavior of frames under these loads is necessary to correlate recent research in earth-quake engineering which has centered around full-scale dynamic testing of buildings and computer studies of simple systems under recorded earthquake motions. And, in recent tests at Berkeley, cantilever beams were tested under cyclic loads to study the connection behavior under earthquake conditions. The earthquake problem is essentially one of constant gravity loads and a variable horizontal acceleration (or displacement) of the base of the frame. The inertia forces are often replaced by horizontal forces applied at the floor levels in studying the dynamic effects of earthquakes. Currently available methods of frame analysis are adequate to predict the static behavior of frames under the combined effect of gravity and monotonically increasing horizontal loads However, these methods were not adequate to describe the behavior of frames under reversed loading.

The initial theoretical investigations will be concerned with the analysis for static cyclic lateral loading of multi-story frames. The static approach will be used as a starting point and is almost necessary to gain a close account of the behavior of a frame under cyclic loading. Since the frame is designed for instability effects, the frame will act as a sub assemblage of a multi-story frame and therefore the columns will have a reduced plastic moment capacity due to the high axial load. The static analysis will be basically second-order elastic-plastic with modifications\ to account for the effect of strain hardening and the true position of the plastic hinges.

The analysis for dynamic cyclic lateral loading of multi-story frames may require merely the extension of the statical method or a present dynamic method, or a completely new approach. After a close study of the variables involved in the dynamic loading and based on the static analysis, a prediction of frame behavior under dynamic conditions may be possible. However, without a dynamic test of a frame, this extension will depend on the use of data obtained in the cyclic coupon tests which are to be conducted as part of the material properties tests and other references. One simple approach has been to use a constant dynamic and plastic moment capacity as some ratio of the static moment capacity Perhaps the non-linear dynamic analysis under study at Michigan which uses the Ramberg-Osgood function to represent the force displacement characteristics of a single degree of freedom system should be correlated to the test frames. And since the extension of this method to multi-story frames under the assumptions that the beams follow the R-O relationship and the columns remain linearly elastic, there will be limitations on such correlations which could be made. Note also that all secondary effects are excluded since only flexural resistance of the members is considered in the above analysis. The spectral response curve and a determination of the damping parameters of the structures tested can be made for correlation purposes with other frame tests. In the latter case, a mechanical system consisting of a mass, a linear spring, and a viscous damper which has its force proportional to the relative velocity to the first power is used to calculate various conventional damping parameters.

The seismic behavior of CFRT column under cyclic loading is good. Compared to steel tubular columns, the CFRT columns exhibited better behavior of resisting local buckling; Compared to conventionally reinforced concrete columns the CFRT columns exhibited greater energy-dissipation characteristics. The test results show that the use of a thicker steel tube can increase the strength. Higher concrete strength will result in higher ultimate strength but larger strength degradation and lower energy dissipation compared to columns with normal strength concrete. It was also found that higher axial compression force resulted in low energy dissipation and larger strength degradation. The numerical method proposed in this paper can not only simulate the load-displacement relationship but also calculate the ultimate strength of CFRT columns, and it can replace the experiment to a certain degree.

SCHOOL OF BUILDING AND ENVIRONMENT
DEPARTMENT OF CIVIL ENGINEERING

UNIT – IV - Frames and Shear Wall– SCIA5301

FRAMES AND SHEAR WALL

SEISMIC CODE (IS 1893)

The Indian seismic code IS 1893 has now been split into a number of parts.

Part 1: General provisions and buildings

Part 2: Liquid retaining tanks elevated and ground supported Part 3:

Bridges and retaining walls

Part 4: Industrial structures including stack like structures Part 5:

Dams and embankments

The first part containing general provisions and those pertaining to buildings has been released in 2002. There has been a gap of 18 years since the previous edition in 1984. Considering the advancements in understanding of earthquake- resistant design during these years, the new edition is a major upgradation of the previous version. Some of these new developments have been incorporated in the 2002 version of the code, while many others have been left out so that the implementation of the code does not become too tedious for Indian professional engineers. For example, in the United States, the codes are revised every three years, and hence, a typical building code in the United States has acquired sophistication gradually over about six revisions during these 18 years. Since the Indian code has had to make a quantum jump with respect to many of the provisions, it still requires considerable effort for an average professional engineer to fully appreciate the new code and to be able to implement it correctly.

In the above scenario, the following steps are urgently needed:

(i) careful review of the new code to remove any deficiencies, errors, or scope for misinterpretation

(ii) development of explanatory handbook on the code to explain the new code with solved examples

PHILOSOPHICAL CHANGES IN THE NEW CODE

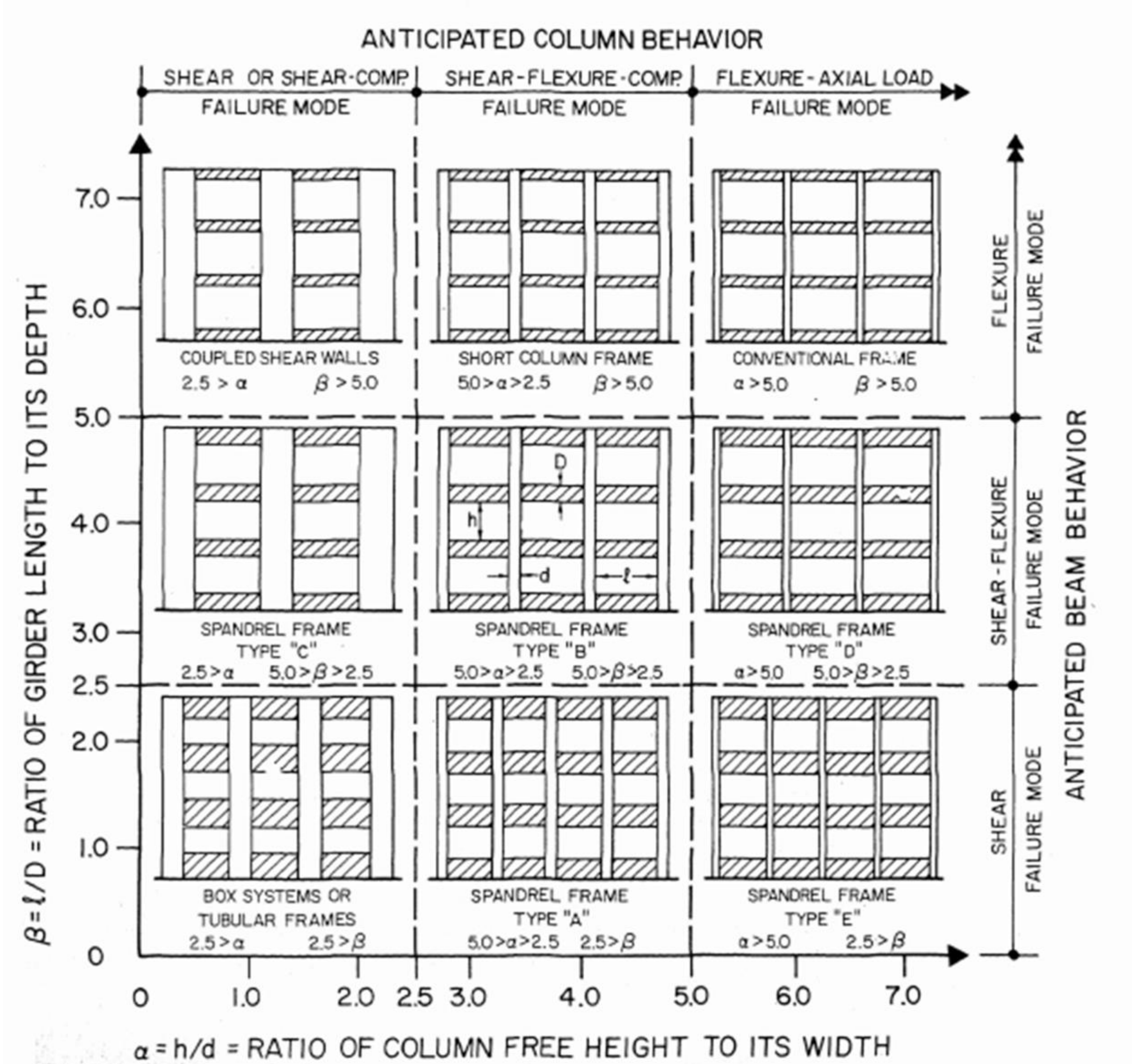
- The seismic zone map now contains only four zones as compared to the five zones earlier, and the relative values of zone factors are now different.
- The code now provides realistic values of acceleration from which the design forces are obtained by dividing the elastic forces by a response reduction factor; this enables a clear statement of intent to the designer that the design seismic force is much lower than what can be expected in the event of a strong shaking.
- The design spectrum shape now depends on the type of soil and the foundation-soil factor (β) has been dropped.
- The code now requires that there be a minimum design force based on empirical fundamental period of the building even if the dynamic analysis gives a very high value of natural period and thus low seismic force.

PROVISION AND PRINCIPLES OF IS 1893(1)-2002

i. General principles and design criteria
ii. Assumptions
iii. Load combination and increase in permissible stresses
iv. Design spectrum
v. Regular and irregular configuration
vi. Importance factor zand response reduction factor
vii. Design imposed loads for earthquakes force calculation
viii. Seismic weight
ix. Design lateral force
x. Fundamental natural period
xi. Distribution of design force
xii. Dynamic analysis
xiii. Torsion
xiv. Buildings with soft storey
xv. Deformations
xvi. Miscellaneous
xvii. Indian map showing epicentres, principle tectonic features, principal lithological groups, seismic zones of india.

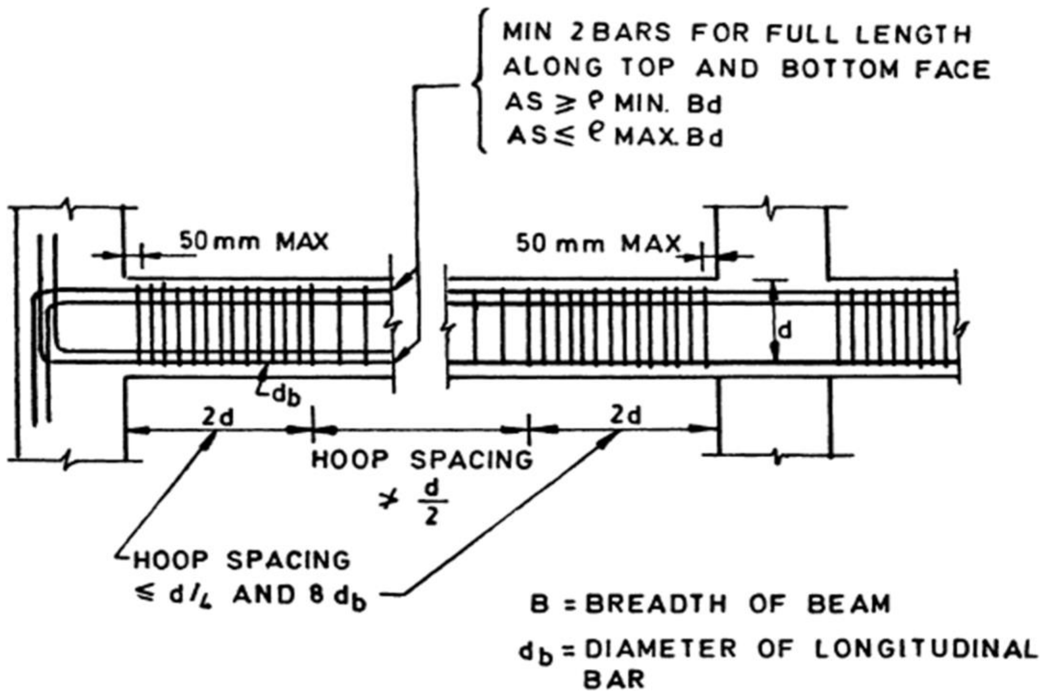
FRAMES AND SHEAR WALLS

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



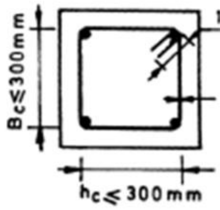
Longitudinal Reinforcement

Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall be spliced at one section. Any area of a column that extends more than 100 mm beyond the confined core due to architectural requirements, shall be detailed in the following manner. In case the contribution of this area to strength has been considered, then it will have the minimum longitudinal and transverse reinforcement as per this code. However, if this area has been treated as non-structural, the minimum reinforcement requirements shall be governed by IS 456 : 1978 provisions minimum longitudinal and transverse reinforcement, as per IS 456 : 1978



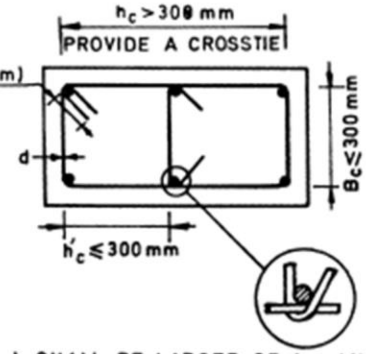
Transverse Reinforcement

Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135" hook _with a 10 diamee;; extension (but not < 75 mm) at each that IS embedded in the confined core .The parallel legs of rectangular hoops shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided Alternatively, a pair of overlapping hoops may be provided within the columm The hooks shall engage peripheral longitudinal bars. The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided, The design shear force for columns shall be the maximum of: a) calculated factored shear force as per analysis, and b) a factored shear force given as per code



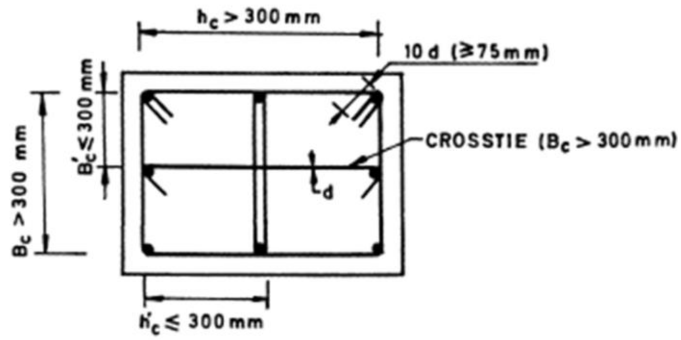
h SHALL BE LARGER OF h_c AND B_c

7A SINGLE HOOP



h SHALL BE LARGER OF h_c AND B_c

7B SINGLE HOOP WITH A CROSSTIE



h SHALL BE LARGER OF h'_c AND B_c

7C OVERLAPPING HOOPS WITH A CROSSTIE

Special Confining Reinforcement

This requirement shall be met with, unless a larger amount of transverse reinforcement is required from shear strength considerations. Special confining reinforcement shall be provided over a length I, from each joint face, towards midspan, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces The length (lo* shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) l/6 of clear span of the member, and (c) 450 mm. When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat When the calculated point of contraflexure, under the effect of gravity and earthquake loads, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height). This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length. Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result due to the presence of bracing, a mezzanine floor or a R.C.C. wall on either side of the column height

SHEAR WALLS

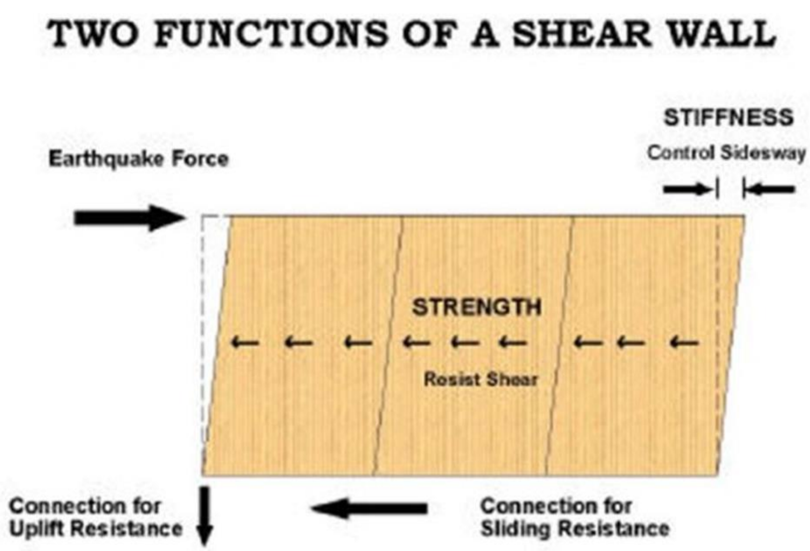
In structural engineering, a **shear wall** is a structural system composed of braced panels (also known as shear panels) to counter the effects of lateral load acting on a structure.Wind and seismic[1] loads are the most common loads that shear walls are designed to carry. Under several building codes, including the International Building Code (where it is called a **braced wall line**) and Uniform Building Code, all exterior wall lines in wood or steel frame construction must be braced. Depending on the size of the building some interior walls must be braced as well.

A structure of shear walls in the center of a large building — often encasing an elevator shaft or stairwell — form a **shear core**.

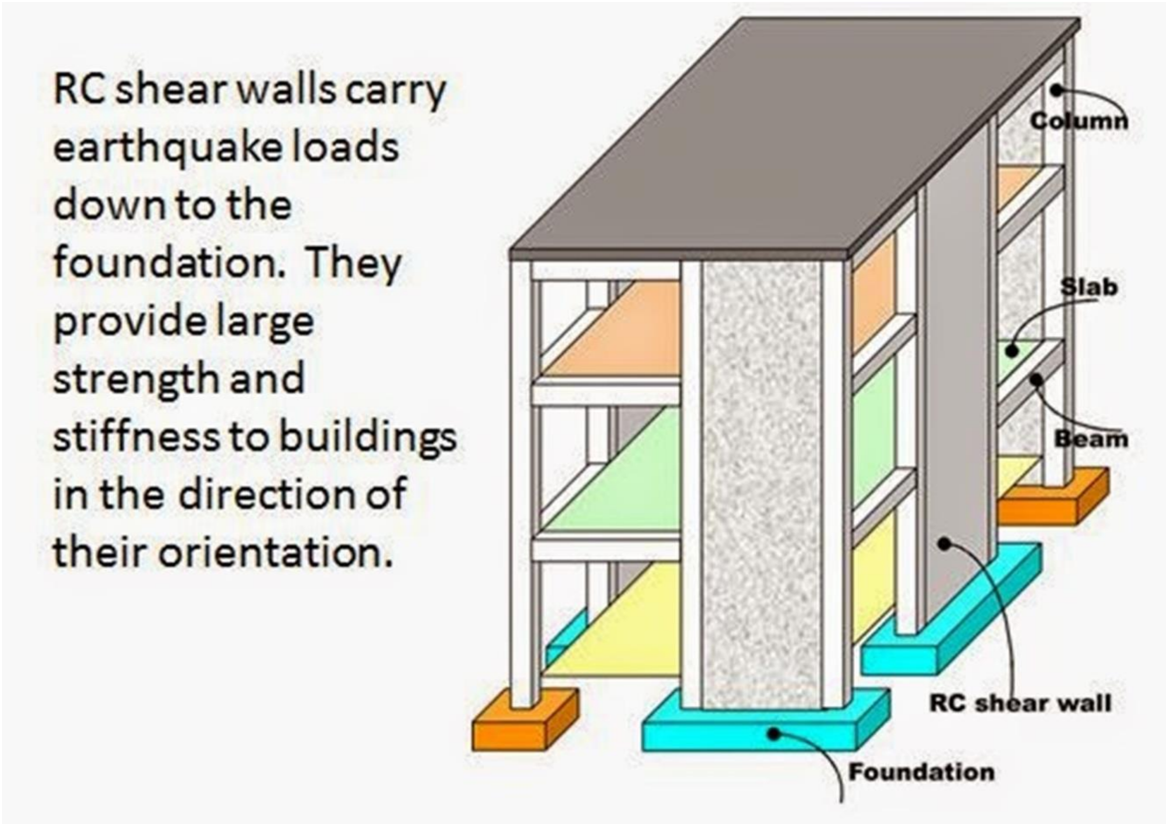
Plywood is the conventional material used in shear walls, but with advances in technology

Shear walls resist in-plane loads that are applied along its height. The applied load is generally transferred to the wall by a diaphragm or collector or drag member. They are built in wood, concrete,and CMU (masonry).

and modern building methods, other prefabricated options have made it possible to inject shear assemblies into narrow walls that fall at either side of an opening. Sheet steel and steel-backed shear panels in the place of structural plywood in shear walls has proved to provide stronger seismic resistance.



The requirements of this section apply to the shear walls, which are part of the lateral force resisting system of the structure. The thickness of any part of the wall shall preferably, not be less than 150 mm. The effective flange width, to be used in the design of flanged wall sections, shall be assumed to extend beyond the face of the web for a distance which shall be the smaller of (a) half the distance to an adjacent shear wall web, and (b) l/IO th of the total wall height. Shear walls shall be provided with reinforcement in the longitudinal and transverse -directions in the plane of the wall. The minimum reinforcement ratio shall be 0.002 5 of the gross area in each direction. This reinforcement shall be distributed uniformly across the cross section of the wall. If the factored shear stress in the wall exceeds 0.25 dfz or if the wall thickness exceeds 200 mm, reinforcement shall be provided in two curtains, each having bars running in the longitudinal and transverse directions in the plane of the wall. The diameter of the bars to be used in any part of the wall shall not exceed l/lOth of the thickness of that part. The maximum spacing of reinforcement in either direction shall not exceed the smaller of I&, 3 tw, and 450 mm; where Zw is the horizontal length of the wall, and tw is the thickness of the wall web. The nominal shear stress, r,,, shall be calculated as: Vll *v = tw where VU = factored shear force, tw = thickness of the web, and d w= effective depth of wall section. This may by taken as 0.8 I, for rectangular sections. The design shear strength of concrete, Q, shall be calculated as per Table 13 of IS 456 : 1978. The nominal shear stress in the wall, rv, shall not exceed Q, maX, as per Table 14 of IS 456 : 1978. When Tv is less than 7Fc shear reinforcement shall be provided When Tv is greater than Q, the area of horizontal shear reinforcement, At,, to be provided within a vertical spacing. S,, is given by V US = @87fy A, 4v & where Vus = (Vu - 7c tw dw), is the shear force to be resisted by the horizontal reinforcement. However, the amount of horizontal reinforcement provided shall not be less than the minimum, The vertical reinforcement, that is uniformly distributed in the wall, shall not be less than the horizontal reinforcement calculated



Flexural Strength

The moment of resistance, MUv, of the wall section may be calculated as for columns subjected to combined bending and axial load as per IS 456 : 1978. The moment of resistance of slender ,rectangular shear wa!l section with uniformly distributed vertical reinforcement The cracked flexural strength of the wall section should be greater than its untracked flexural strength. In walls that do not have boundary elements, vertical reieforcement shall be concentrated at the ends of the wall. Each concentration shall consist of a minimum of 4 bars of 12 mm diameter arranged in at least 2 layers. Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. Though they may have the same thickness as that of the wall web it is advantageous to provide them with greater thickness. Where the extreme fibre compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds 0*2f,k, boundaty elements shall be provided along the vertical boundaries of walls. The boundary ,elements may be discontinued where the calculated compressive stress becomes less than 0. I sfck. The compressive stress shall be calculated using a linearly elastic model and gross section properties. A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry an axial compression equal to the sum of factored gravity load on it and the additional compressive load induced by the seismic force.

Advantages of Shear Walls in RC Buildings

Properly designed and detailed buildings with shear walls have shown very good performance in past earthquakes. The overwhelming success of buildings with shear walls in resisting strong earthquakes is summarised in the quote: “We cannot afford to build concrete buildings meant to resist severe earthquakes without shear walls.” :: Mark Fintel, a noted consulting engineer in USA Shear walls in high seismic regions require special detailing. However, in past earthquakes, even buildings with sufficient amount of walls that were not specially detailed for seismic performance (but had enough well-distributed reinforcement) were saved from collapse. Shear wall buildings are a popular choice in many earthquake prone countries, like Chile, New Zealand and USA. Shear walls are easy to construct, because reinforcement detailing of walls is relatively straight- forward and therefore easily implemented at site. Shear walls are efficient, both in terms of construction cost and effectiveness in minimizing earthquake damage in structural and nonstructural elements (like glass windows and building contents)

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SCHOOL OF BUILDING AND ENVIRONMENT

DEPARTMENT OF CIVIL ENGINEERING

UNIT – V - Computation Analysis, Modern Concepts– SCIA5301

COMPUTATION ANALYSIS, MODERN CONCEPTS

Performance of regular buildings Masonry Building

Masonry buildings of brick and stone are superior with respect to durability, fire resistance, heat resistance and formative effects. Masonry buildings consist of various material and sizes (i) Large block (block size >50 cm)-concrete blocks, rock blocks or lime stones;(ii) concrete brick-solid and hollow; (iii) Natural stone masonry. Because of its easy availability, economic reasons and the merits mentioned above this type of construction are widely used. In very remote areas in Himalayas buildings are constructed of stacks of random rock pieces without any mortar. The majority of new construction use mud mortar, however, few use cement mortar also.

Causes of failure of masonry buildings:

These buildings are very heavy and attract large inertia forces. Unreinforced masonry walls are weak against tension (Horizontal forces) and shear, and therefore, perform rather poor during earthquakes. These buildings have large in plane rigidity and therefore have low time periods of vibration, which results in large seismic force. These buildings fall apart and collapsed because of lack of integrity. The lack of structural integrity could be due to lack of 'through' stones, absence of bonding between cross walls, absence of diaphragm action of roofs and lack of box light action.

Reinforced Masonry Buildings:

Reinforced masonry buildings have withstood earthquakes well, without appreciable damage. For horizontal bending, a tough member capable of taking bending if found to perform better during earthquakes. If the corner sections or opening are reinforced with steel bars even greater strength is attained. Even dry packed stone masonry wall with continuous lintel band over openings and cross walls did not undergo any damage.

Brick-R.C. Frame Buildings:

This type of building consists of RC frame structures and brick lay in cement mortar as infill. This type of construction is suitable in seismic areas.

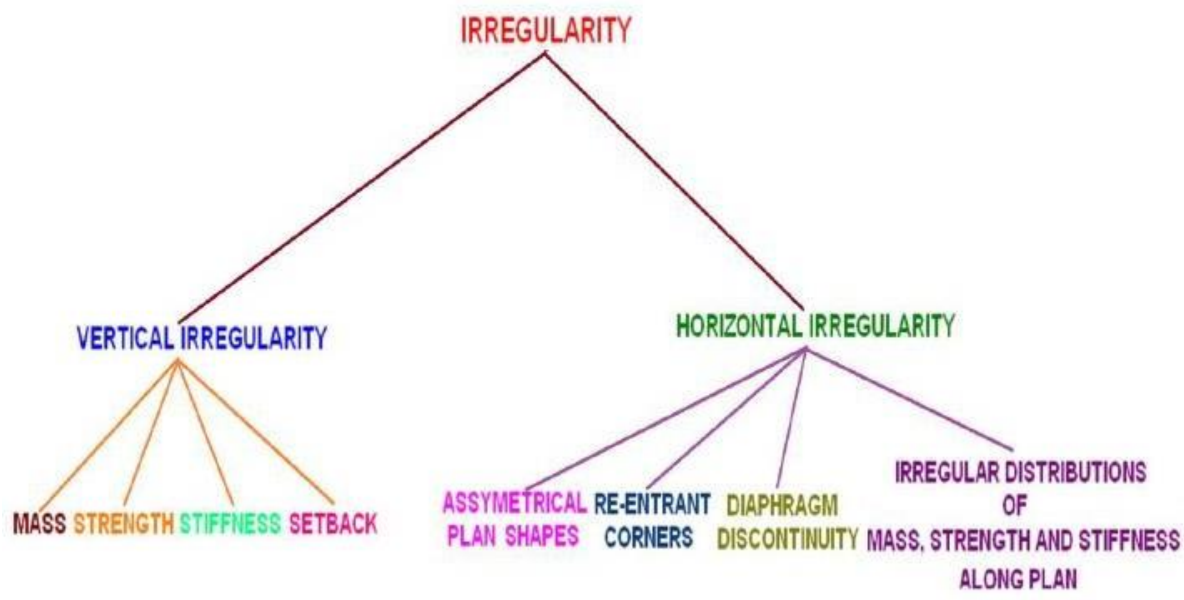
Causes of failure of RC frame buildings:

The failures are due to mainly lack of good design of beams /columns frame action and foundation. Poor quality of construction inadequate detailing or laying of reinforcement in various components particularly at joints and in columns /beams for ductility. Inadequate diaphragm action of roof and floors. Inadequate treatment of masonry walls.

Common type of damage in RC frame buildings:

The damage is mostly due to failure of infill, or failure of columns or beams. Spalling of concrete in columns. Cracking or buckling due to excessive bending combined with dead load may damage the column. The buckling of columns are significant when the columns are slender and the spacing of stirrup in the column is large.

Severe crack occurs near rigid joints of frame due to shearing action, which may lead to complete collapse. The differential settlement also causes excessive moments in the frame and may lead to failure. Design of frame should be such that the plastic hinge is confined to beam only, because beam failure is less damaging than the common failure**.**



IRREGULAR BUILDINGS AND THEIR SEISMIC PERFORMANCE

Buidings, which have more length compared to its width, fare better in terms of seismic performance. Slender buildings, which have less width but more height is found to sustain more damage during an earth quake than buildings which have more width. Buildings with reentrant corners, and also those, which lack in symmetry in the plan shape also, fare badly during earth quakes2 . Buildings, which have different loads in different storeys, will shake at different amplitudes at different levels, and this may result in structural damage in the building, if the vibrations exceed the natural frequency of the building components. Due to the variations in the terrain level, height of columns vary at the ground floor level for certain buildings. Such buildings have a tendency to buckle and fail during earthquake intensity. The difference in height of several storeys leads to great difference in stiffness at each level and may induce seismic damage. The effects of seismic forces on the irregular buildings are twisting, overturning, damage to nonstructural elements, and also damage to structural elements etc.

Some of the considerations for better seismic performance of buildings1 are:

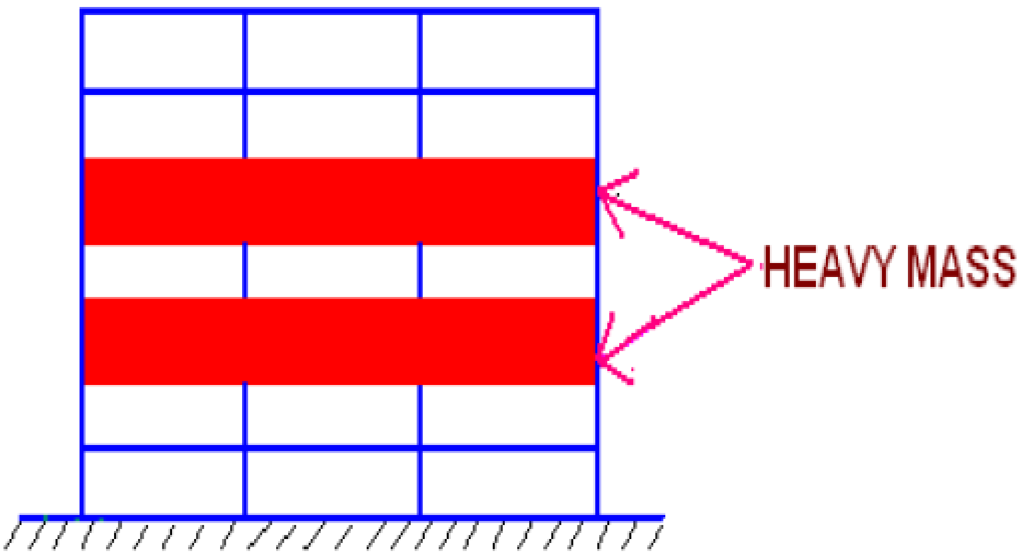
1. Having better width to height ratio for the buildings

2. Having symmetrical plan

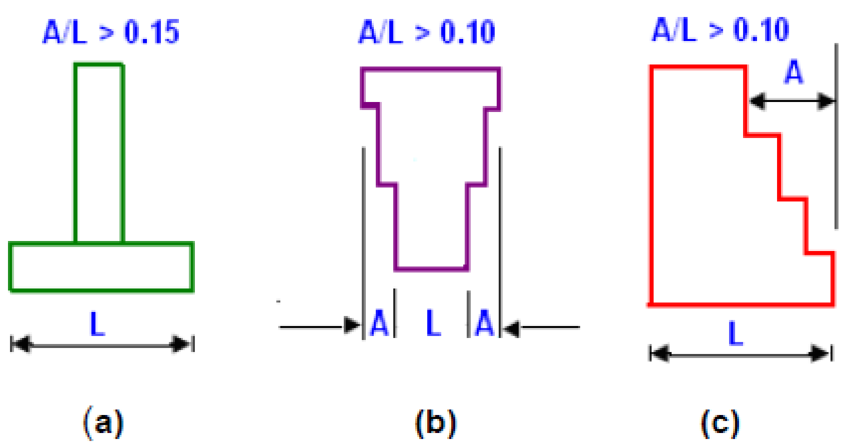
3. Introduction of shear walls, wherever needed

4. Improving ductility at places where stiffness is more

5. Adopting same height in all storey levels 6. Having better distribution in mass at all floor levels



Mass irregularity



Vertical setback irregularity

Failure Due To Vertical Irregularities

The vertical irregularities can be sub - classified into mass, stiffness, strength and setback irregularity. The Olive medical center was one of the building which failed during San Fernado earthquake in 1971. It was a six-storeyed building with mass

irregularity in the form of excess earth fill at the first storey. Furthermore, structural walls were present at the second floor level which resulted in stiffness and strength irregularities in the second storey. In addition several columns in the ground storey contained inadequate lateral confinement. Therefore, the first two storeys of the building which were critical for the building stability contained irregularities of mass, strength and stiffness. After occurrence of earthquake it was observed that the first two storeys which supported the whole building incurred heavy damage, and in contrary the upper four floors sustained a very less damage . A similar collapse of a four storey building was observed due to open first storey during Northridge earthquake (1994) .The soft-storey effect was the main reason for collapse of many multi-storey R/C buildings during the earthquakes that occurred in Turkey during the last decade (Adalier and Aydingun 1998; Durumus et al. 1999; Huang and Skokan 2002; Sezen et al. 2003; Eyidogan et al. 2003; http:// bingol.meb.gov.tr/index.html).. The majority of the residential and commercial buildings built in Turkey had soft storeys at the first-floor level which were often used for commercial purposes. These storeys were generally enclosed with glass windows instead of brick infill walls so as to be used as showrooms. The heavy masonry infills starting immediately above the soft storey which created a large variation of mass, stiffness and strength in the bottom storeys. The previous earthquake damages and results of analytical studies showed that the structural systems with a soft storey led to serious problems during severe earthquake ground shaking. During the occurrence of an earthquake, the presence of a soft storey increased the deformation demands significantly and the first-storey columns were expected to dissipate the whole seismic energy. Many failures and collapses can be attributed to the increased deformation demands in conjunction with poorly designed columns. The soft storey has been one of the major reasons of damage throughout the world during earthquakes as evident from seismic reports (EQE,1995; Youd et al. 2000; Yoshimura 2003). Therefore, it is prescribed to avoid sudden change of mass, stiffness and strength along the building height especially at the bottom storey.

Failure of Plan Irregular Buildings

Damage to irregular structures caused by asymmetry in plan has been observed during many major and minor earthquakes during the past. The non-coincident centers of mass and stiffness in a structure generate plan asymmetry which causes torsional vibration resulting in severe damage to structural components in the more laterally flexible regions of the structure. Due to presense of a stiff wall, the center of stiffness shifted towards the wall. This resulted in twisting of building with respect to the center of stiffness. This was due to occurrence of torsion generated by the eccentricity between the centers of mass and stiffness. The torsion resulted in severe damage of columns along the periphery away from the wall. the Ministry of Culture building which was damaged due to torsion during the Haiti earthquake in 2010. The presence of stiff core area on one side of the building resulted in torsion which led to damage of lateral load-resisting members away from the center of stiffness. Due to failure of these members, the whole storey experienced a downward pull which led to the total collapse of the building. Likewise, the six-storey reinforced concrete hotel in Guatemala City had torsional irregularity due to the eccentric location of a rigid service core area. This building experienced a severe damage during the Guatemala earthquake in 1976. The columns of the building located on the flexible side failed due to their incapability to resist the shear force increase due to the torsion. This resulted in second storey collapse in the building. Moreover, severe damage and collapse of the structures due to symmetric layouts of lateral load-resisting members because of torsional effects were reported in previous literature works (Wyllie et al. 1986; Anderson 1987; Elnashai at el. 2010). As discussed, the irregular structures rendered poor seismic performance, and this is mainly due to ignorance of the irregularity aspect in formulating the seismic design methodologies by the seismic codes (IS 1893:2002, EC8:2004, UBC 1997, NBCC 1995, NBCC
2005 etc.). Nevertheless, ignoring the irregularity aspect may induce noticeable errors in

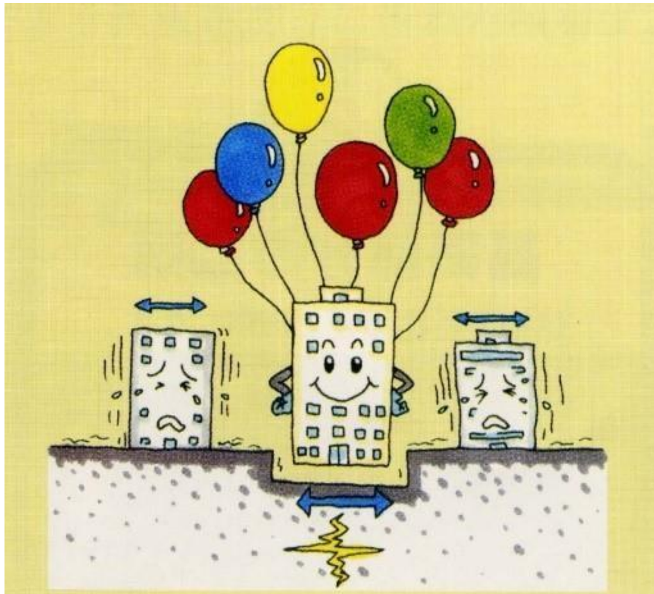
estimation of the seismic response parameters. Moreover, a structural engineer requires a good understanding regarding behavior of these irregular systems. However, most of the past research works mainly focus on the regular systems. Therefore, there is a need of a comprehensive evaluation of effects of different types of irregularity on the seismic response parameters to formulate improved design philosophy for these structures.

Base Isolation System

Base isolation is a state-of-the-art method in which the structure (superstructure) is separated from the base (foundation or substructure) by introducing a suspension system between the base and the main structure. n context of seismic design of structures, base isolation can be replaced with seismic isolation i.e., the structure above the ground, which is most affected during earthquake is separated from the effects of earthquake forces by introducing a mechanism that will help the structure to hover. The concept of base isolation is quite easy to grasp. It can be explained as a bird flying during an earthquake is not affected. In simple words if structure is floating on its base, the movement of ground will have no effect on the structure.

Purpose of Base Isolation

Wind and Earthquake are the most predominant loads that demands lateral design of a structure. Again, earthquake load is not controllable and it is not practical to design a structure for an indefinite seismic demand. Only practical approach left is to accept a demand and make sure the capacity is more than the demand. The inertial forces caused due to earthquake is directly proportional to the mass of structure and the ground acceleration. Increasing ductility of the building or increasing the elastic strength of the structure is the most conventional method of handling seismic demand. Engineer has to increase the capacity exceed the demand. Base isolation takes an opposite approach, i.e. to reduce the seismic demand instead of increasing the capacity. Controlling ground motion is impossible, but we can modify the demand on structure by preventing/reducing the motions being transferred to the structure from foundations.



Logic of Ideal Base Isolation

Principle of Base Isolation

The basic principle behind base isolation is that the response of the structure or a building is modified such that the ground below is capable of moving without transmitting minimal or no motion to the structure above. A complete separation is possible only in an ideal system. In a real world scenario, it is necessary to have a vertical support to transfer the vertical loads to the base. The relative displacement of ground and the structure is zero for a perfectly rigid, zero period structure, since the acceleration induced in the structure is same as that of ground motion. Whereas in an ideal flexible structure, there is no acceleration induced in the structure, thus relative displacement of the structure will be equal to the ground displacement. No Structure is perfectly rigid or flexible, therefore, the response of the structure will be between the two explained above. Maximum acceleration and displacements are a function of earthquake for periods between zero to infinity. During earthquakes there will be a range of periods at which acceleration in the building will be amplified beyond maximum ground acceleration, though relative displacements may not exceed peak ground displacements. Base isolation is the ideal method to cater this, by reducing the transfer of motion, the displacement of building is controlled.

Displacement occurs at CG of the structures for fixed base structures, which will be approx. two-third height for buildings and at isolation plane for base isolated structures with lesser displacement within the structure. The response of a base isolated structure and a structure without base isolation can be illustrated as shown in the figure below. The displacement and acceleration is controlled by base isolation.

Basic requirements of an isolation system are

1. Flexibility
2. Damping
3. Resistance to Vertical or other service loads.

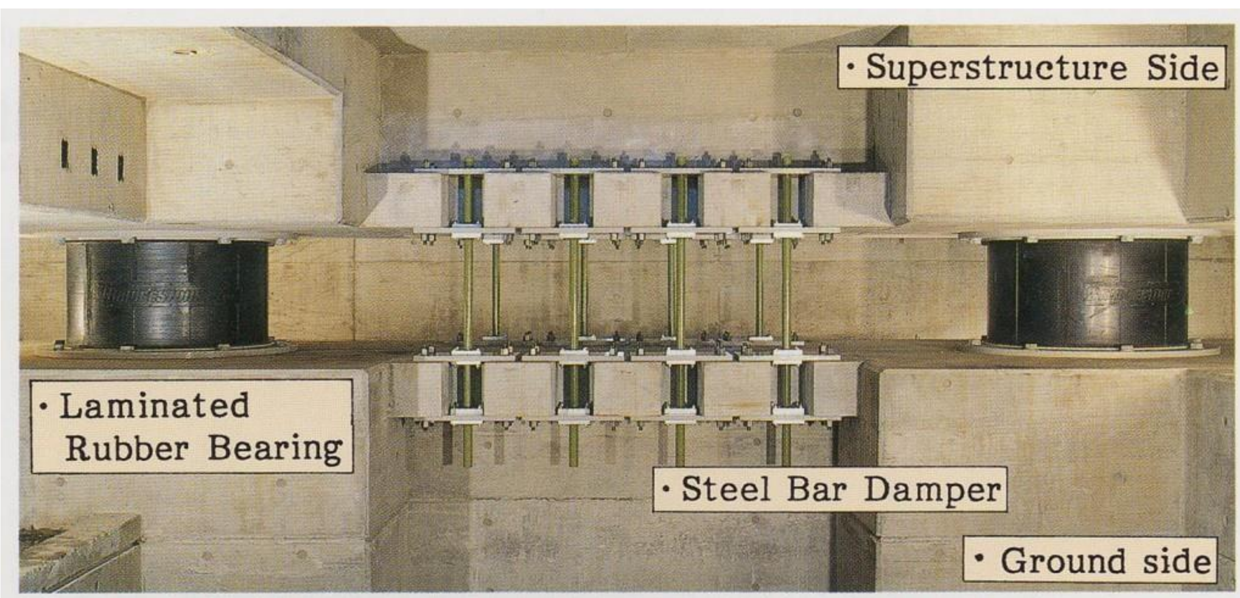
Type of Base Isolation Systems

Well, what kind of mechanism can achieve this, resisting the gravitational pull of earth? A lubricated sliding surface? Or a strong magnetic levitation? These might sound right but are not the right engineering solution. It should be a system which is capable of restraining the structure under strong gust of winds and gravitational pull. Though an ideal solution is yet to be discovered or invented, there are a few practical isolation mechanisms which are widely used in the field of earthquake engineering. Which means that these systems are capable of reducing the seismic demand of the structure.

Six major types of Isolators are

Elastomeric Rubber Bearings

Bearings formed of horizontal layers of synthetic or natural rubber in thin layers bound between steel plates. These bearings are capable of supporting high vertical loads with very small deformations. These bearings are flexible under lateral loads. Steel plates prevent the rubber layers from bulging. Lead cores are provided to increase damping capacity as plain elastomeric bearings does not provide significant damping. They are usually soft in horizontal direction and hard in vertical direction.



Roller and Ball Bearings

For isolation applications in machinery isolation, roller and ball bearing are used. It includes cylindrical rollers and balls. It is sufficient to resist service movements and damping depending on the material used.

Springs

Steel springs are most likely used in mechanical applications as in roller bearings. It is not adopted in structural applications because it is flexible in both vertical and horizontal directions. This will increase service deflections.

Sliding Bearing

Sliding systems with a predefined coefficient of friction can provide isolation by limiting acceleration and forces that are transferred. Sliders are capable of providing resistance under service conditions, flexibility and force-displacements

by sliding movement. Shaped or spherical sliders are often preferred over flat sliding systems because of their restoring effect. Flat sliders provide no restoring force and there are possibilities of displacement with aftershocks.

Soft-story and Sleeved Piles and Rocking Isolation systems are other major systems.

Type of Base Isolation Devices

There are Six major types of base isolation devices which are widely adopted for seismic base isolation.

1. Elastomeric Bearings
2. High Damping Bearings
3. Lead Rubber Bearings
4. Flat Slider Bearings
5. Curved Slider Bearings or Pendulum Bearings
6. Ball & Roller Bearings

Flexibility and Damping are the two major components of base isolation system. Flexibility of the isolation has predominant effect in response modification. Viscous dampers or Hysteretic dampers are often provided to enhance isolation. Response reduction using dampers is independent of the structure stiffness.

Types of Base Isolation System Dampers

1. Steel Dampers
2. Oil Dampers

3. Lead Dampers
4. Friction Dampers with disc springs

Advantages of Base isolation

1. Reduced the seismic demand of structure, thereby reducing the cost of structure.
2. Lesser displacements during an earthquake.
3. Improves safety of Structures
4. Reduced the damages caused during an earthquake. This helps in maintaining the performance of structure after event.
5. Enhances the performance of structure under seismic loads.
6. Preservation of property

Applications of Base isolation

1. Base isolation of bridges
2. Base isolation of important buildings
3. Enhancing response of historic structures
4. Isolation in Machinery Field