EARTHQUAKE REISTANT DESIGN
UNIT – 1
EARTHQUAKE ENGINEERING

Seismology

Seismology is the study of the generation, propagation and measurement of seismic waves through earth and the sources that generate them. The word seismology originated fromGreek words, ‘seismos’ meaning earthquake and ‘logos’ meaning science. The study of seismic wave propagation through earth provides the maximum input to the understanding of internal structure of earth.

Earthquake phenomenon

Earthquake is the vibration of earth’s surface caused by waves coming from a source of disturbance inside the earth (refer Figure 1.1). Most earthquakes of engineering significance are of tectonic origin and is caused by slip along geological faults.

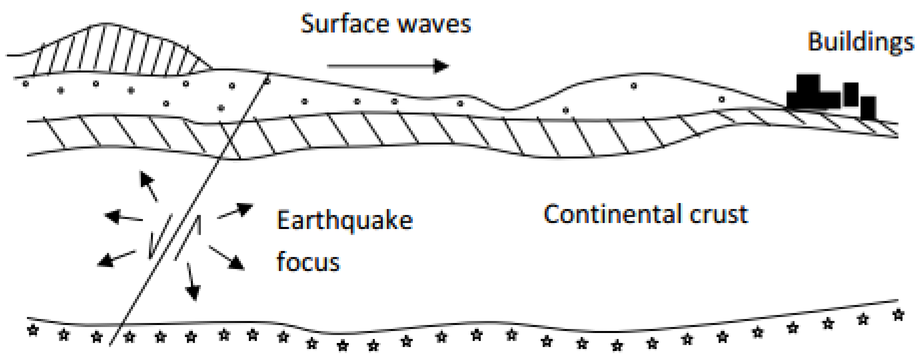


Figure 1.1 General depiction of an earthquake

An earthquake (also known as a quake, tremor or temblor) is the result of a sudden release of energy in the Earth’s crust that creates seismic waves. The seismicity or seismic activity of an area refers to the frequency, type and size of earthquakes experienced over a period of time. The word earthquake is used to describe any seismic event whether a natural phenomenon or an event caused by humans that generates seismic waves. Earthquakes are caused mostly by rupture of geological faults, but also by volcanic activity, landslides, mine blasts, and nuclear tests.

Causes and Effects of Earthquakes

The important causes of the earthquake are

Natural Causes of Earthquake:

1. Tectonic Movement: This particularly happens when the continental plate collides against the oceanic plate. The oceanic plate is overridden by the continental plate. By a process called subduction jerky movements are caused along the inclined surface. Tectonic earthquakes have occurred in Assam in 1950.
2. Volcanic Activity: Earthquakes may also be caused by the movement of lava beneath the surface of the earth during volcanic activity. The earthquakes due to Krakatoa volcanic eruption in 1883 is a good example of volcanic eruption.
3. Dislocation of the Earth’s crust: Earthquakes may be caused by the dislocation of the crust beneath the surface of the Earth.
4. Adjustment in inner Rock Beds: Earthquakes are also caused where is an adjustment between Sima [i.e., beneath the ocean is formed by Silica and Magnesium] and Sial (i.e., Continent is formed by Silica and Aluminum) in the interior of the Earth’s Crust. This Earthquake may be called as a Plutonic Earthquake.
5. Pressure of gases in the interior: The expansion and contraction of gases in the interior of the Earth sometimes cause a sudden shake on the Earth’s surface.
6. Other Causes:

1. Landslides and avalanches,
2. Denudation of the Landmasses and depositions of materials,
3. Faulting and folding in the rock beds are responsible for causing minor earthquakes.

Man-made Earthquakes:

1. The impounding of large quantities of water behind dams disturbs the crustal balance. This causes earthquakes such as the Koyna earthquake in Maharashtra.
2. The shock waves through rocks set up by the underground testing of Atom bombs or Hydrogen bombs may be severe to cause earthquake.

Effects of Earthquake

1. Earthquake causes dismantling of buildings, bridge and other structures at or near epicenter. Many men and animals are killed or buried under collapsed houses.
2. Rails are folded, underground wires broken. Fire breaks out inevitably in large towns.
3. Earthquakes originate sea waves called Tsunamis.
4. Earthquakes result in the formation of cracks and fissures on the ground formation.
5. The earthquakes cause landslides and disturb the isostatic equilibrium.
6. Landslide due to earthquake may block valleys to form lakes

Faults

The term fault is used to describe a discontinuity within rock mass, along which movement had happened in the past.Plate boundary is also a type of fault.Most faults produce repeated displacements over geologic time. Movement along a fault may be gradual or sometimes sudden thus, generating an earthquake.

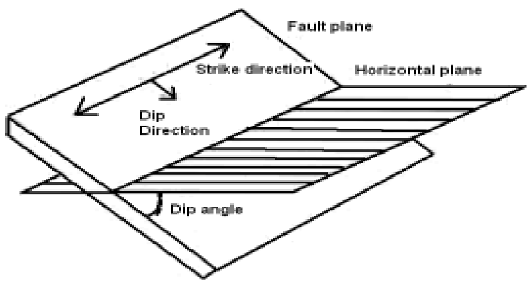


Figure 1.2 Various terminologies associated with the rupture plane of a fault

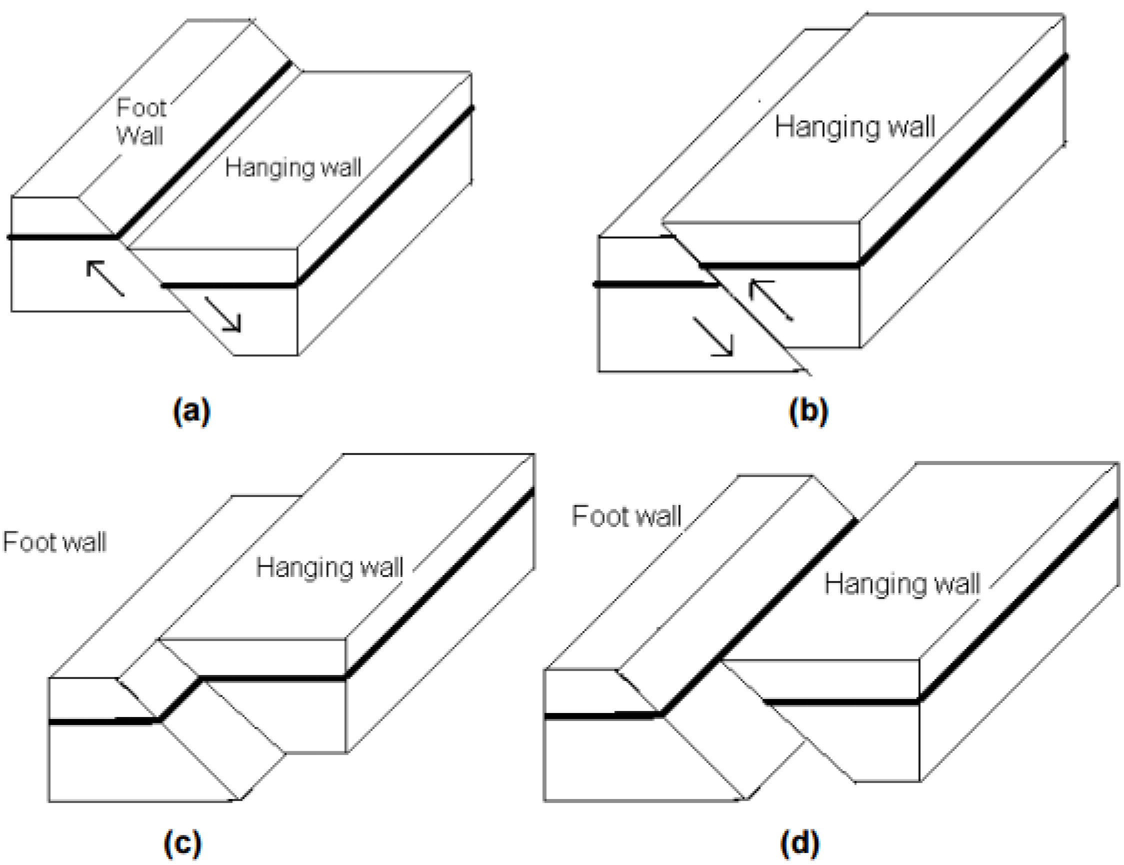


Figure 1.3 Types of faults (Arrow shows direction of relative displacement) (a) Normal fault; (b) Reverse fault; (c) Strike-slip fault; (d) Oblique fault

There are two important parameters associated with describing faults, namely, dip and strike, Figure 1.2. The strike is the direction of a horizontal line on the surface of the fault. The dip, measured in a vertical plane at right angles to the strike of the fault, is the angle of fault plane with horizontal. The hanging wall of a fault refers to the upper rock surface along which displacement has occurred, whereas the foot wall is the term given to that below. The vertical shift along a fault plane is called the throw, and the horizontal displacement is termed as heave

Faults are classified in to dip-slip faults, strike-slip faults and oblique-slip faults based on the direction of slippage along the fault plane, Figure 1.12. In a dip-slip fault, the slippage occurred along the dip of the fault, Figure – 1.3(a) and (b). In case of a strike-slip fault, the movement has taken place along the strike, Figure 1.3(c). The movement occurs diagonally across the fault plane in case of an oblique slip fault, Figure 1.3(d). Based on relative movement of the hanging and foot walls faults are classified into normal, reverse and wrench faults. In a normal fault, the hanging wall has been displaced downward relative to the footwall, Figure 1.3(a). In a reverse fault, the hanging wall has been displaced upward relative to the footwall, Figure 1.3 (b). In a wrench fault, the foot or the hanging wall do not move up or down in 9 relation to one another, Figure 1.3 (c). Thrust faults, which are a subdivision of reverse faults, tend to cause severe earthquakes. Faults are nucleating surfaces for seismic activity. The stresses accumulated due to plate movement produces strain mostly along the boundary of the plates. This accumulated strain causes rupture of rocks along the fault plane.

Internal Structure of Earth

The earth's shape is an oblate spheroid with a diameter along the equator of about 12740 km with the polar diameter as 12700km. The higher diameter along equator is caused by the higher centrifugal forces generated along the equator due to rotation of earth. Though the specific gravity of materials that constitute the surface of earth is only about 2.8, the average specific gravity of earth is about 5.5 indicating presence of very heavy materials towards interior of earth. The interior of the earth can be classified into three major categories as Crust, Mantle and Core

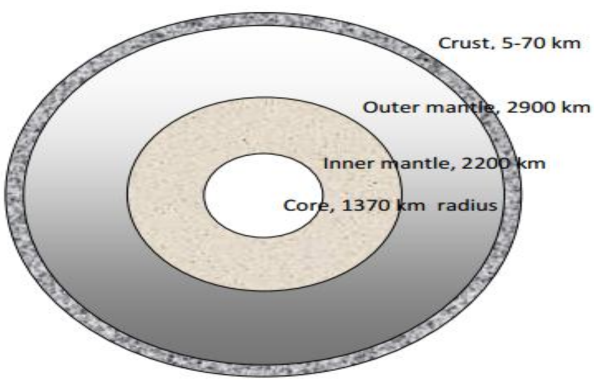


Figure 1.4 Cross-section of interior of earth.

Crust: or the lithosphere, is the outer part of the earth is where the life exist. The average thickness of crust beneath continents is about 40km where as it decreases to as much as 5km beneath oceans. The oceanic crust is constituted by basaltic rocks and continental part by granitic rocks overlying the basaltic rocks. Compared to the layers below, this layer has high rigidity and anisotropy. Mantle: is a 2900 km thick layer. The mantle consists of 1) Upper Mantle reaching a depth of about 400 km made of olivine and pyroxene and 2) Lower Mantle made of more homogeneous mass of magnesium and iron oxide and quartz. No earthquakes are recorded in the lower mantle. The specific gravity of mantle is about 5. The mantle has an average temperature of about 2200degree Celsius and the material is in a viscous semi molten state. The mantle act like fluid in response to slowly acting stresses and creeps under slow loads. But it behaves like as solid in presence of rapidly acting stresses, e.g. that caused by earthquake waves.

Core: has a radius of 3470 km and consists of an inner core of radius 1370 km and an outer core (1370 km $< R <$ 3470 km). The core is composed of molten iron, probably mixed with small quantities of other elements such as nickel and sulphur or silicon. The inner solid core is very dense nickel-iron material and is subjected to very high pressures. The maximum temperature in the core is estimated to be about 3000 degree Celsius

Plate tectonics

The theory of plate tectonics, presented in early 1960s, explains that the lithosphere is broken into seven large (and several smaller) segments called plates.These plates move independently relative to one another, with a restricted independence from the 7 large plates, however. The relative, horizontal movements are ideally described as rigid body motions that produce space and friction problems at the contacts between adjacent plates. Plate boundaries are not fixed; they also move and change shape. The global mosaic of plates periodically reorganizes itself and new plate boundaries form while others close up. Plate tectonics, the study of such relative motions and their consequences, allows relating surface, geological and geophysical structures with quantified movements attributed to deep processes of the Earth's heat engine.

This theory requires a source that can generate tremendous force is acting on the plates. The widely accepted explanation is based on the force offered by convection currents created by thermo-mechanical behavior of the earth's subsurface. The variation of mantle density with temperature produces an unstable equilibrium. The colder and denser upper layer sinks under the action of gravity to the warmer bottom layer which is less dense. The lesser dense material rises upwards and the colder material as it sinks gets heated up and becomes less dense. These convection currents create shear stresses at the bottom of the plates which drags them along the surface of earth.The earthquake that occurs at a plate boundary is known as inter-plate earthquake. Not all earthquakes occur at plate boundaries. Though, interior portion of a plate is usually tectonically quiet, earthquakes also occur far from plate boundaries. These earthquakes are known as intra-plate earthquakes. The recurrence time for an intraplate earthquake is much longer than that of inter-plate earthquakes

Movement of Plate Boundaries

Spreading ridges :Spreading ridges or divergent boundaries are areas along the edges of plates move apart from each other. This is the location where the less dense molten rock from the mantle rises upwards and becomes part of crust after cooling.

Convergent boundaries: The convergent boundaries are formed where the two plates move toward each other. In this process, one plate could slip below the other one or both could collide with each other.

Subduction boundaries: These boundaries are created when either oceanic lithosphere subducts beneath oceanic lithosphere (ocean-ocean convergence), or when oceanic lithosphere subducts beneath continental lithosphere (ocean-continent convergence). The junction where the two plates meet, a trench known as oceanic trench is formed.

Transform boundaries: Transform boundaries occur along the plate margins where two plate moves past each other without destroying or creating new crust.

Elastic Rebound theory

As the plate try to move relative to each other, strain energy gets built up along the boundaries. When the stress buildup reaches the ultimate strength of rock, rock fractures and releases the accumulated strain energy. The nature of failure dictates the effect of the fracture. If the material is very ductile and weak, hardly any strain energy could be stored in the plates due to their movement. But if the material is strong and brittle, the stress built up and subsequent sudden rupture releases the energy stored in the form of stress waves and heat. The propagation of these elastic stress waves causes the vibratory motion associated with earthquakes.

The region on the fault, where rupture initiates is known as the focus or hypocenter of an earthquake. Epicenter is the location on the earth surface vertically above the focus. Distance from epicenter to any place of interest is called the epicentral distance. The depth of the focus from the epicenter is the focal depth. Earthquakes are sometime classified into shallow focus, intermediate focus and deep focus earthquakes based on its focal depth. Most of the damaging earthquakes are shallow focus earthquakes.

Earthquake Terminology

Generally, the rupture causing earthquakes initiates from a point, termed as hypocenter or focus, which subsequently spreads over to a large area. Depending on the characteristics of strata where rupture occurs, the shape of the ruptured area could be highly irregular and the amount of interface slip along the ruptured surface could also vary.

Several terms associated with earthquake rupture/propagation are discussed given below:

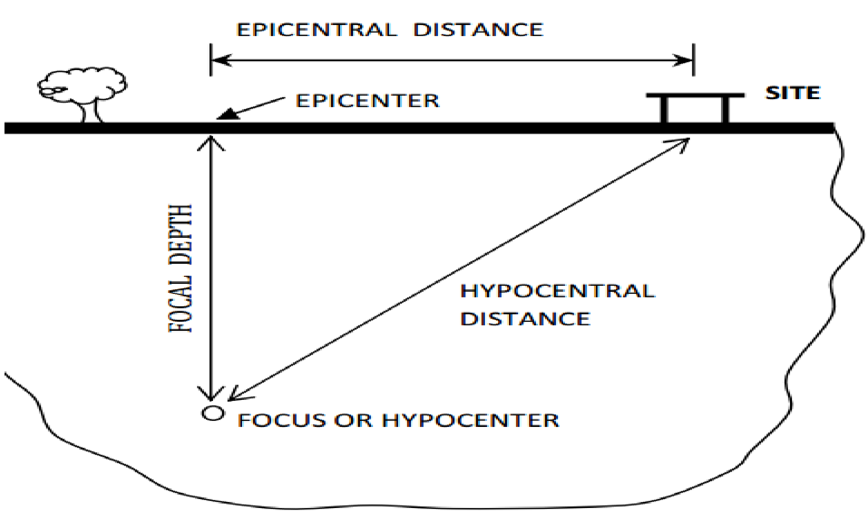


Figure 1.5 Various distance measurements associated with earthquake.

Focus: The place of origin of the earthquake in the interior of the earth is known as focus or origin or centre or hypocenter

Epicenter: The place on the earth's surface, which lies exactly above the centre of the earthquake, is known as **the 'epicenter**

Intensity of Earthquakes

The intensity of an earthquake refers to the degree of destruction caused by it. In other words, intensity of an earthquake is a measure of severity of the shaking of ground and its attendant damage.

The extent of destruction or damage that takes place to a construction at a given place depends on many factors. Some of these factors are: (i) distance from the epicenter, (ii) compactness of the underlying ground, (iii) type of construction (iv) magnitude of the earthquake (v) duration of the earthquake and (vi) depth of the focus. Intensity is the oldest measure of earthquake.

Numerous intensity scales have been developed over the last several hundred years to evaluate the effects of earthquakes, the most popular is the Modified Mercalli Intensity (MMI) Scale. This scale, composed of 12 increasing levels of intensity that range from imperceptible shaking to catastrophic destruction, is designated by Roman numerals. It does not have a mathematical basis; instead it is an arbitrary ranking based on observed effects. The lower numbers of the intensity scale generally deal with the manner in which the earthquake is felt by people. The higher numbers of the scale are based on observed structural damage.

Magnitude of Earthquake

The magnitude of an earthquake is related to the amount of energy released by thegeological rupture causing it, and is therefore a measure of the absolute size of theearthquake, without reference to distance from the epicenter.

Richter Magnitude

A workable definition of magnitude was first proposed by C.F. Richter. Based on the data from Californian earthquakes, defined the earthquake magnitude as thelogarithm to the base 10 of the largest displacement of a standard seismograph situated 100 km from the focus.

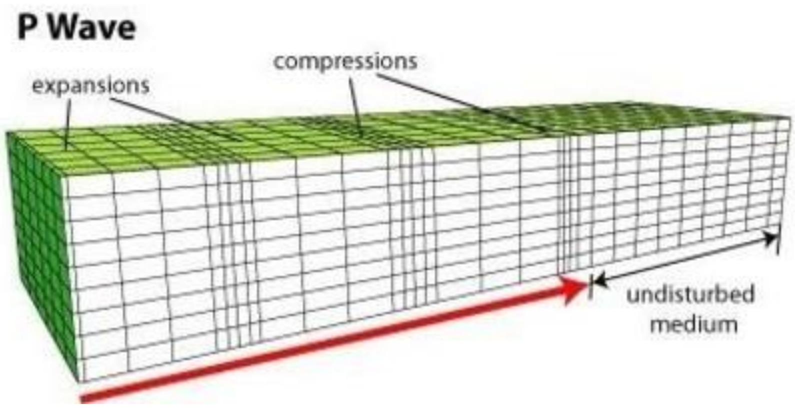
$$M = log_{10}A$$

Where A denotes the amplitude in micron (10^{-6}m) recorded by the instrument located at an epicentral distance of 100 km; and M is the magnitude of the earthquake

Earthquake Waves

Earthquake vibrations originate from the point of initiation of rupture and propagatesin all directions. These vibrations travel through the rocks in the form of elastic waves. Mainly there are three types of waves associated with propagation of an elastic stress wave generated by an earthquake. These are primary (P) waves, secondary (S) waves and surface waves

Primary (P) Waves

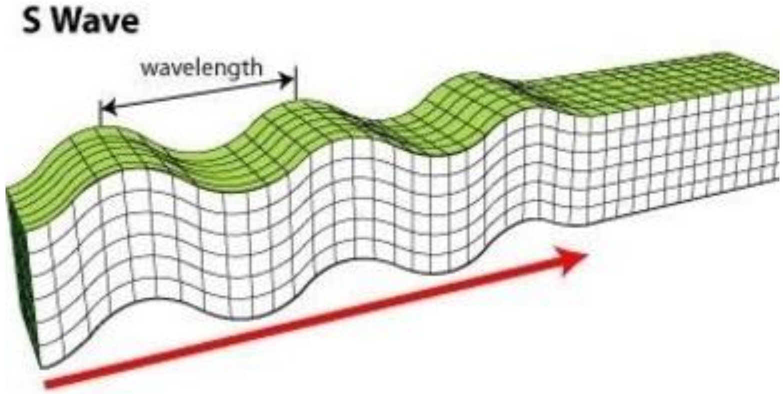


Primary (P) Waves

These are known as primary waves, push-pull waves, longitudinal wave compressional waves, etc. These waves propagate by longitudinal or compressiveaction, which mean that the ground is alternately compressed and dilated in thedirection of propagation, P waves are the fastest among the seismicwaves and travel as fast as 8 to 13 km per second. These waves are capable of traveling through solids, liquids and gases

Secondary (S) Waves

These are also called shear waves, secondary waves, transverse waves, etc.Compared to P waves, these are relatively slow. These are transverse or shear waves, which mean that the ground is displaced perpendicularly to the direction of propagation, Figure. In nature, these are like light waves, i.e., the waves move perpendicular to the direction of propagation.These waves are capable of traveling only through solids.They travel at the rate of 5 to 7 km per second. For this reason these waves are always recorded after P waves in a seismic station



Secondary (S) Waves

Surface Waves

When the vibratory wave energy is propagating near the surface of the earth rather than deep in the interior, two other types of waves known a Rayleigh and Love waves can be identified. These are called surface waves because their journey is confined to the surface layers of the earth only. Surface waves travel through the earth crust and does not propagate into the interior of earth unlike P or S waves.Surface waves are the slowest among the seismic waves.They travel at the rate of 4 to 5 km per second.

These waves are capable of travelling through solids and liquids. They are complex in nature and are said to be of two kinds, namely, Raleigh waves and Love waves.

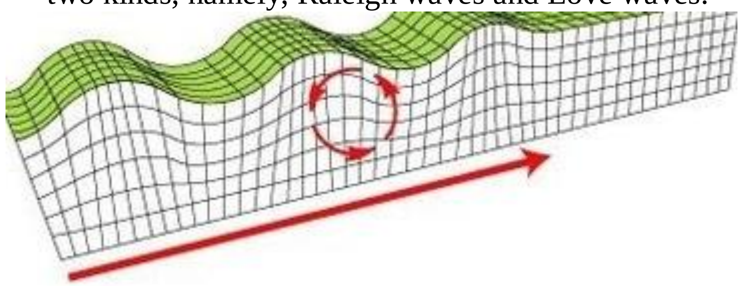


Figure Raleigh wave

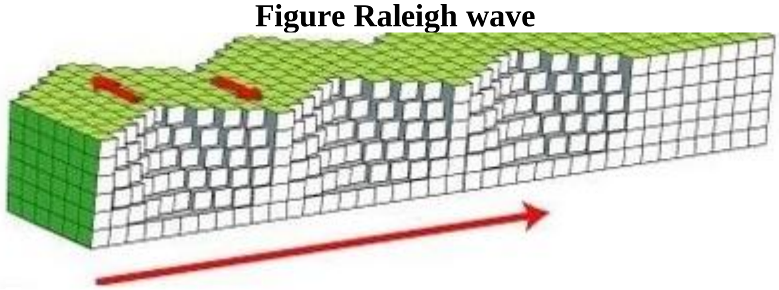


Figure Love wave

The Rayleigh surface waves are tension-compression waves similar to the P-waves expect that their amplitude diminishes with distance below the surface of the ground. Similarly, the Love waves are the counterpart of the "S" body waves; they are shear waves that diminishes rapidly with distance below surface.

Seismic Zones of India

The varying geology at different locations in the country implies that the likelihood of damaging earthquakes taking place at different locations is different. Thus, a seismic zone map is required to identify these regions. Based on the levels of intensities sustained during damaging past earthquakes

The seismic zone maps are revised from time to time as more understanding is gained on the geology, the seismotectonics and the seismic activity in the country. The Indian Standards provided the first seismic zone map in 1962, which was later revised in 1967 and again in 1970. The map has been revised again in 2002 and it now has only four seismic zones – II, III, IV and V. The areas falling in seismic zone I in the 1970 version of the map are merged with those of seismic zone II. Also, the seismic zone map in the peninsular region has been modified.

Madras now comes in seismic zone III as against in zone II in the 1970 version of the map. This 2002 seismic zone map is not the final word on the seismic hazard of the country, and hence there can be no sense of complacency in this regard.

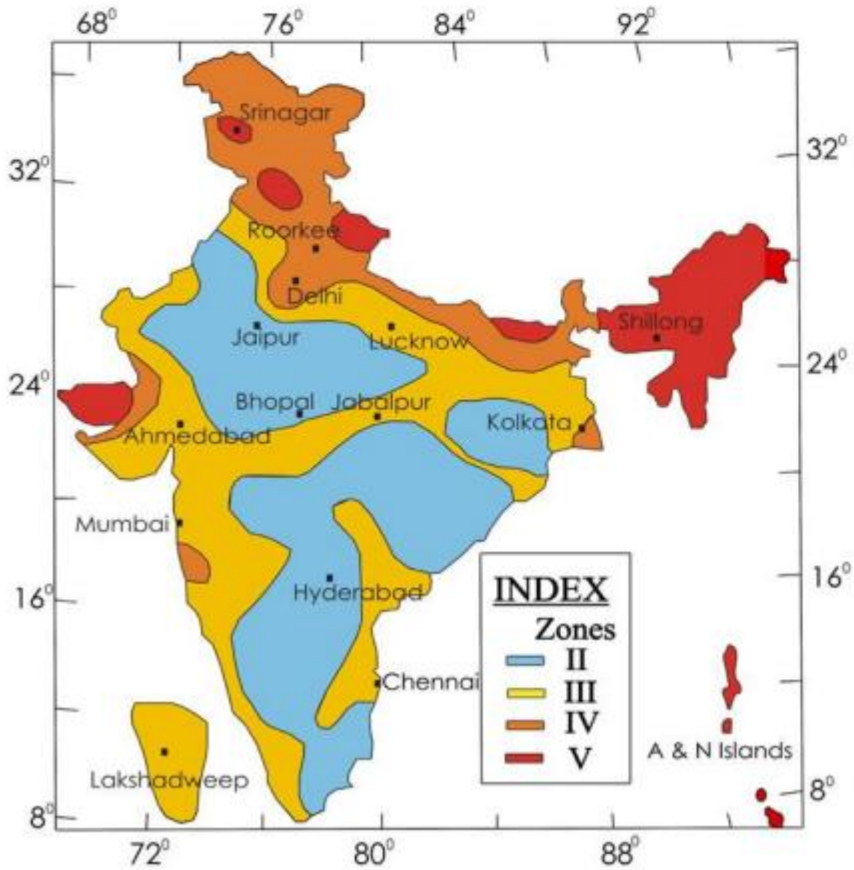
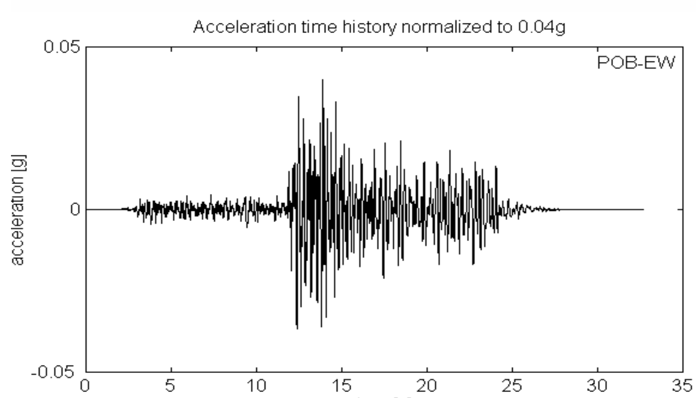


Figure Upgraded seismic zoning map of India (BIS, 2004)

Accelerogram

During ground shaking, one can measure ground acceleration versus time (accelerogram) using an accelerograph.Accelerograph is the instrument. Accelerogram is the record obtained from it. Accelerogram is the variation of ground acceleration with time.



Accelerogram

Seismogram

Seismogram is visual record of arrival time and magnitude of shaking associated with seismic wave, generated by a seismograph.A seismograph used to describe a recording device that detects ground motion due to earthquake.Device used to measure an earthquake is calledseismograph.

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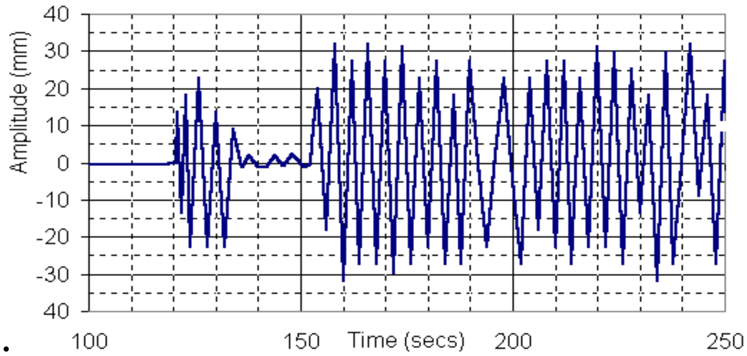


Figure seismogram

UNIT - II

INTRODUCTION TO STRUCTURAL DYNAMICS

Objective: The objective of this experiment is to introduce you to principles in structural dynamics through the use of an instructional shake table. Natural frequencies, mode shapes and damping ratios for a scaled structure will be obtained experimentally.

1.0 Introduction

The dynamic behavior of structures is an important topic in many fields. Aerospace engineers must understand dynamics to simulate space vehicles and airplanes, while mechanical engineers must understand dynamics to isolate or control the vibration of machinery. In civil engineering, an understanding of structural dynamics is important in the design and retrofit of structures to withstand severe dynamic loading from earthquakes, hurricanes, and strong winds, or to identify the occurrence and location of damage within an existing structure.

In this experiment, you will test a small test building of two floors to observe typical dynamic behavior and obtain its dynamic properties. To perform the experiment you will use a bench-scale shake table to reproduce a random excitation similar to that of an earthquake. Time records of the measured absolute acceleration responses of the building will be acquired.

2.0 Theory: Dynamics of Structures

To understand the experiment it is necessary to understand concepts in dynamics of structures. This section will provide these concepts, including the development of the differential equation of motion and its solution for the damped and undamped case. First, the behavior of a single degree of freedom (SDOF) structure will be discussed, and then this will be extended to a multi degree of freedom (MDOF) structure.

The number of degrees of freedom is defined as the minimum number of variables that are required for a full description of the movement of a structure. For example, for the single story building shown in figure 1 we assume the floor is rigid compared to the two columns. Thus, the displacement of the structure is going to be completely described by the displacement, x, of the floor. Similarly, the building shown in figure 2 has two degrees of freedom because we need to describe the movement of each floor separately in order to describe the movement of the whole structure.

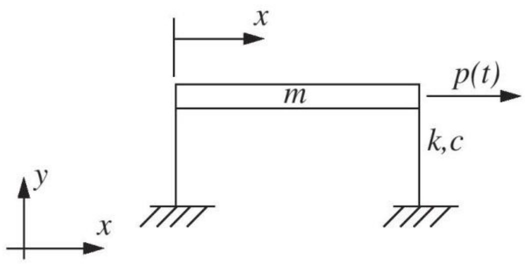


Figure 1. One degree of freedom structure.

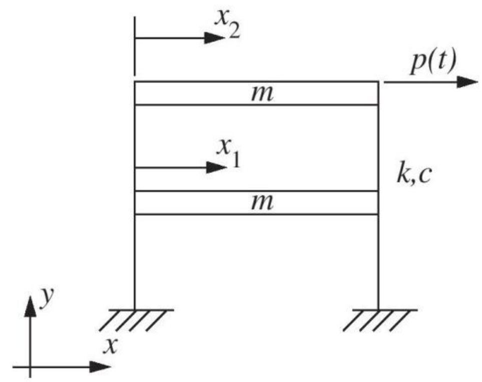


Figure 2. Two degree of freedom structure.

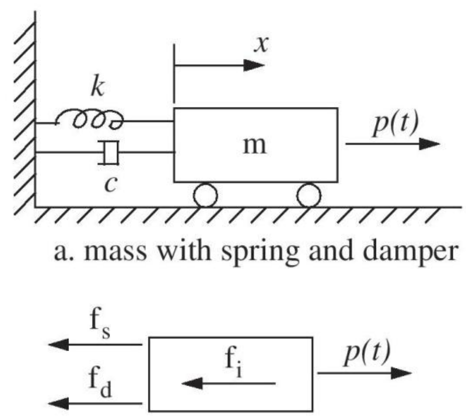
2.1 One degree of freedom

We can model the building shown in figure 1 as the simple dynamically equivalent model shown in figure 3a. In this model, the lateral stiffness of the columns is modeled by the spring (k), the damping is modeled by the shock absorber (c) and the mass of the floor is modeled by the mass (m). Figure 3b shows the free body diagram of the structure. The forces include the spring force $f_s(t)$, the damping force $f_d(t)$, the external dynamic load on the structure, $p(t)$, and the inertial force $f_i(t)$. These forces are defined as:

$$f_s = k \cdot x \tag{1}$$

$$f_d = c \cdot \dot{x} \tag{2}$$

$$f_i = m \cdot \ddot{x} \tag{3}$$



a. mass with spring and damper

b. free body diagram

Figure 3. Dynamically equivalent model for a one floor building.

where the $\dot{x}$ is the first derivative of the displacement with respect to time (velocity) and $\ddot{x}$ is the second derivative of the displacement with respect to time (acceleration).

Summing the forces shown in figure 3b we obtain

$$\Sigma F = m \cdot \ddot{x} = p(t) - c\dot{x} - kx \tag{4}$$

$$m\ddot{x} + c\dot{x} + kx = p(t) \tag{5}$$

where the mass m and the stiffness k are greater than zero for a physical system.

2.1.1 Undamped system

Consider the behavior of the undamped system (c=0). From differential equations we know that the solution of a constant coefficient ordinary differential equation is of the form

$$x(t) = e^{\alpha t} \tag{6}$$

and the acceleration is given by

$$\ddot{x}(t) = \alpha^2 e^{\alpha t}. \tag{7}$$

Using equations (6) and (7) in equation (5) and making $p(t)$ equal to zero we obtain

$$m\alpha^2 e^{\alpha t} + k e^{\alpha t} = 0 \tag{8}$$

$$e^{\alpha t}[m\alpha^2 + k] = 0. \tag{9}$$

Equation (9) is satisfied when

$$\alpha^2 = \frac{-k}{m} \tag{10}$$

$$\alpha = \pm i\sqrt{\frac{k}{m}}. \tag{11}$$

The solution of equation (5) for the undamped case is

$$x(t) = Ae^{\omega_n i t} + Be^{-\omega_n i t} \tag{12}$$

where A and B are constants based on the initial conditions, and the natural frequency ω_n is defined as

$$\omega_n = \sqrt{\frac{k}{m}}. \tag{13}$$

Using Euler's formula and rewriting equation (12) yields

$$e^{i\alpha t} = \cos\alpha t + i\sin\alpha t \tag{14}$$

$$x(t) = A(\cos(\omega_n t) + i\sin(\omega_n t)) + B(\cos(-\omega_n t) + i\sin(-\omega_n t)) \tag{15}$$

$$x(t) = A\cos(\omega_n t) + Ai\sin(\omega_n t) + B\cos(-\omega_n t) + Bi\sin(-\omega_n t). \tag{16}$$

Using $\cos(-\alpha) = \cos(\alpha)$ and $\sin(-\alpha) = -\sin(\alpha)$ we have

$$x(t) = A\cos(\omega_n t) + Ai\sin(\omega_n t) + B\cos(\omega_n t) - Bi\sin(\omega_n t) \tag{17}$$

$$x(t) = (A + B)\cos(\omega_n t) + (A - B)i\sin(\omega_n t). \tag{18}$$

Letting $A + B = C$ and $A - B = D$ we obtain

$$x(t) = C(\cos\omega_n t) + Di(\sin\omega_n t) \tag{19}$$

where C and D are constants that are dependent on the initial conditions of *x(t)*.

From equation (19) it is clear that the response of the system is harmonic. This solution is called the *free vibration response* because it is obtained by setting the forcing function, *p(t)*, to zero. The value of ω_n describes the frequency at which the structure vibrates and is called the *natural frequency*. Its units are *radians/sec*. From equation (13) the natural frequency, ω_n, is determined by the stiffness and mass of the structure.

The vibration of the structure can also be described by the natural period, T_n. The period of the structure is the time that is required to complete one cycle given by

$$T_n = \frac{2\pi}{\omega_n}. \tag{20}$$

2.1.2 Damped system

Consider the response with a nonzero damping coefficient $c \neq 0$. The homogenous solution of the differential equation is of the form

$$x(t) = e^{\alpha t} \tag{21}$$

and

$$\dot{x}(t) = \alpha e^{\alpha t} \tag{22}$$

$$\ddot{x}(t) = \alpha^2 e^{\alpha t}. \tag{23}$$

Using equations (21), (22) and (23) in equation (5) and making the forcing function $p(t)$ equal to zero we have

$$e^{\alpha t}[m\alpha^2 + c\alpha + k] = 0. \tag{24}$$

Solving for α we have

$$\alpha_{1,2} = \frac{-c \pm \sqrt{c^2 - 4km}}{2m} \tag{25}$$

Defining the critical damping coefficient as

$$c_{cr} = \sqrt{4km} \tag{26}$$

and the damping ratio as

$$\zeta = \frac{c}{c_{cr}} \tag{27}$$

we can rewrite equation (25)

$$\alpha_{1,2} = -\zeta\omega_n \pm i\omega_n\sqrt{1-\zeta^2}. \tag{28}$$

Defining the damped natural frequency as

$$\omega_d = \omega_n\sqrt{1-\zeta^2} \tag{29}$$

equation (28) can be rewritten as

$$\alpha_{1,2} = -\zeta\omega_n \pm i\omega_d. \tag{30}$$

Thus, the solution for the differential equation of motion for a damped unforced system is

$$x(t) = Ae^{-\zeta\omega_n t}e^{-i\omega_d t} + Be^{-\zeta\omega_n t}e^{i\omega_d t}\text{, or} \tag{31}$$

$$x(t) = e^{-\zeta\omega_n t}(Ae^{-i\omega_d t} + Be^{i\omega_d t}) \tag{32}$$

Using equation (14) (Euler's formula)

$$x(t) = e^{-\zeta\omega_n t}(C\cos(\omega_d)t + Di\sin(\omega_d)t) \tag{33}$$

where *C* and *D* are constants to be determined by the initial conditions.

Civil structures typically have low damping ratios of less than 0.05 (5%). Thus, the damped natural frequency, ω_d, is typically close to the natural frequency, ω_n.

Comparing the solutions of the damped structure in equation (19) and the undamped structure in equation (33), we notice that the difference is in the presence of the term $e^{-\zeta\omega_n t}$. This term forces the response to be shaped with an exponential envelope as shown in figure 4.

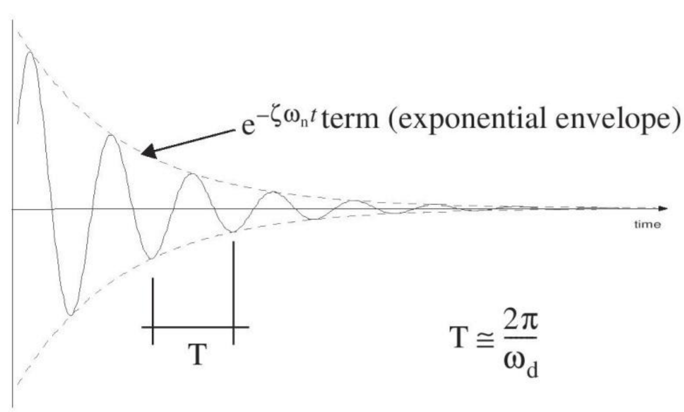


Figure 4. Response of damped structures.

Summary: In this section you learned basic concepts for describing a single degree of freedom system (SDOF). In the followings section you will extend these concepts to the case of multiple degree of freedom systems.

2.2 Multiple degree of freedom systems

A multiple degree of freedom structure and its equivalent dynamic model are shown in figure 5. The differential equations of motion of a multiple degree of freedom system is

$$\mathbf{M}\ddot{\mathbf{x}} + \mathbf{C}\dot{\mathbf{x}} + \mathbf{K}\mathbf{x} = \mathbf{p}(t) \tag{34}$$

where $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ are matrices that describe the mass, damping and stiffness of the structure, $\mathbf{p}(t)$ is a vector of external forces, and $\mathbf{x}$ is a vector of displacements. A system with n degrees of freedom has mass, damping, and stiffness matrices of size $n \times n$, and n natural frequencies. The solution to this differential equation has $2n$ terms.

The structure described by Eq. (34) will have n natural frequencies. Each natural frequency, ω_n, has an associated mode shape vector, ϕ_n, which describes the deformation of the structure when the system is vibrating at each associated natural frequency. For example, the mode shapes for the four degree of freedom structure in figure 5 are shown in figure 6. A node is a point that remains still when the structure is vibrating at a natural frequency. The number of nodes is related with the natural frequency number by

$$\#\text{nodes} = n - 1 \tag{35}$$

where n is the frequency number associated with the mode shape.

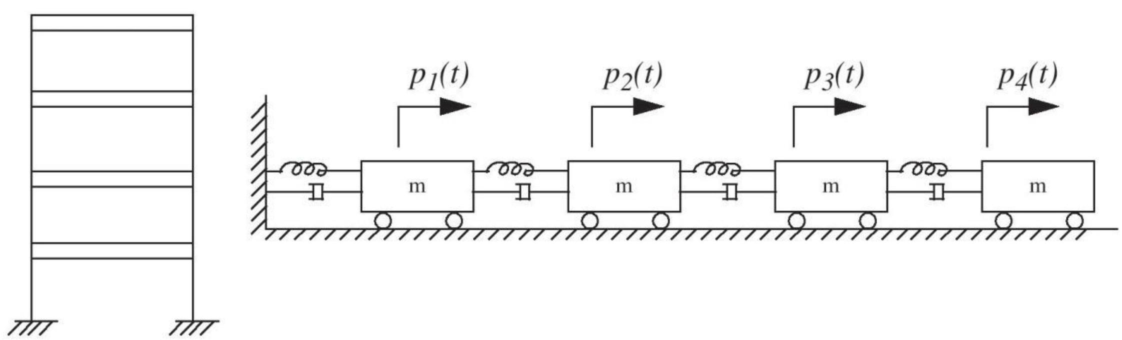


Figure 5. Multiple degree of freedom system and dynamic model.

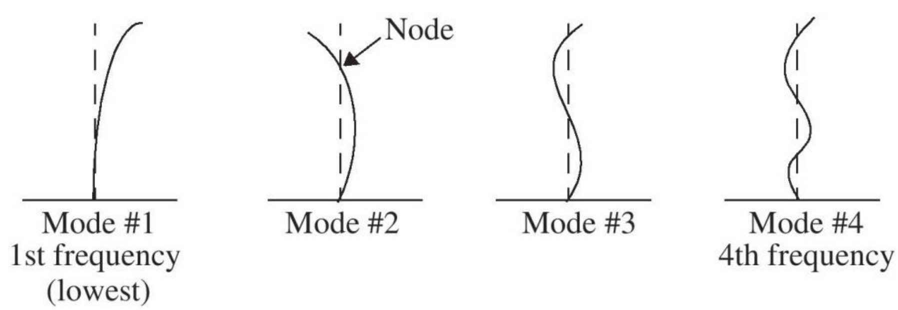


Figure 6. Diagram of mode shapes for a four degree of freedom structure.

2.3 Frequency Domain Analysis

The characteristics of the structural system can also be described in the frequency domain. The Fourier transform of a signal *x(t)* is defined by

$$X(f) = \int_{-\infty}^{\infty} x(t)e^{-i2\pi ft}dt \tag{36}$$

and is related to the Fourier transform of the derivatives of this function by

$$[\dot{x}(t)] = i2\pi f X(f) \tag{37}$$

$$[\ddot{x}(t)] = -(2\pi f)^2 X(f) \tag{38}$$

Plugging this into the equation of motion (equation (5)) for the SDOF system, we obtain

$$[-(2\pi f)^2 m + i2\pi fc + k]X(f) = P(f) \tag{39}$$

and the ratio of the frequency domain representation of the output to the frequency domain representation of the input is determined

$$H(f) = \frac{X(f)}{P(f)} = [k - (2\pi f)^2 m + i2\pi fc]^{-1}. \tag{40}$$

which is called the complex frequency response function, or transfer function. Note that this is a function of the frequency, *f*, and provides the ratio of the structural response to the input loading at each frequency.

Figure 7 shows an example of a transfer function for a two degree of freedom structure. Here the magnitude of the complex function in Eq. (40) is graphed. The *X* axis represents frequency (in either radians per second or Hz) and the *Y* axis is provided in decibels. One decibel is defined as

$$dB = 20\log(\text{Amplitude}) \tag{41}$$

Peak(s) in the transfer function correspond to the natural frequencies of the structure, as shown in Figure 7.

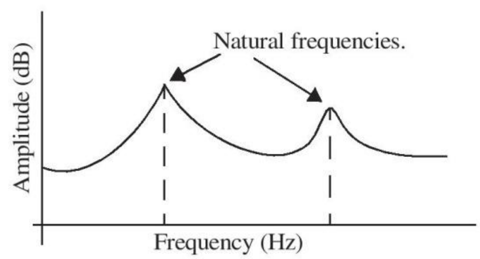


Figure 7. Transfer function.

2.4 Experimental determination of the damping in a structure

A structure is characterized by its mass, stiffness and damping. The first two may be obtained from the geometry and material properties of the structure. However, damping should be determined through experiments. For purposes of this experiment you will assume that the only damping present in the structure is due to viscous damping. Two commonly used methods to determine the damping in structures are the logarithmic decrement method and the half power bandwidth method. The logarithmic decrement technique obtains the damping properties of a structure from a free vibration test using time domain data. The half power bandwidth method uses the transfer function of the structure to determine the amount of damping for each mode.

2.4.1 Exponential decay

Using free vibration data of the acceleration of the structure one may obtain the damping ratio. Figure 8 shows a free vibration record of a structure. The logarithmic decrement, δ, between two peaks is defined as

$$\delta = \ln\frac{y_1}{y_2} \tag{42}$$

where y_1 and y_2 are the amplitudes of the peaks.

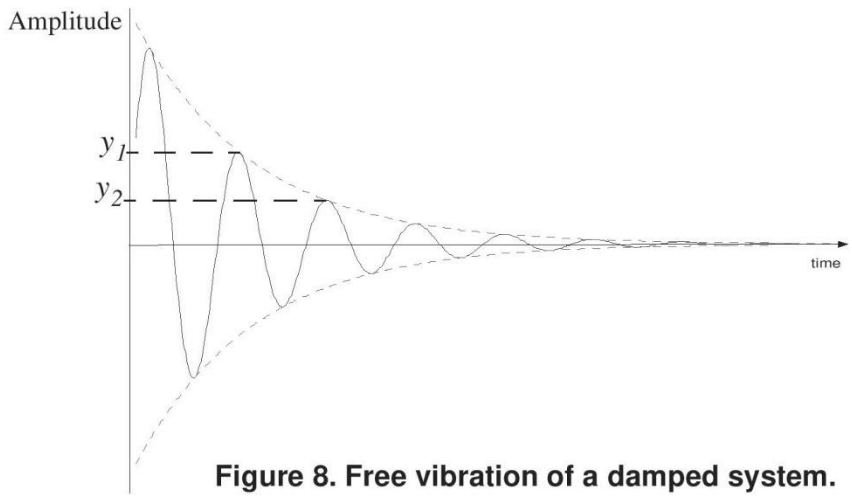


Figure 8. Free vibration of a damped system.

From the solution of the damped system (equation (33)) we can say that y_1 and y_2 can be written as

$$y_1 = \mathrm{C}e^{-\zeta\omega_n t} \tag{43}$$

$$y_2 = \mathrm{C}e^{-\zeta\omega_n (t + \mathrm{T})} \tag{44}$$

where the constant C includes the terms of the sine and cosines in equation (33), and T is the period of the system. Using equations (43) and (44) in equation (42)

$$\delta = \ln\frac{y_1}{y_2} = \ln\frac{\mathrm{C}e^{-\zeta\omega_n t}}{\mathrm{C}e^{-\zeta\omega_n (t + \mathrm{T})}} = \zeta\omega_n \mathrm{T} \tag{45}$$

and when the damping ratio is small, can be approximated as

$$\delta \cong 2\pi\zeta . \tag{46}$$

Solving for ζ

$$\zeta = \frac{\delta}{2\pi} = \frac{\ln\frac{y_1}{y_2}}{2\pi} \tag{47}$$

Using equation (47) we can obtain the damping ratio ζ of the structure using the amplitude of the signal at two consecutive peaks in a free vibration record of displacement or acceleration.

2.4.2 Half power bandwidth method

The second method to obtain an estimation of the damping of a structure is the half power bandwidth method. In contrast to the previous method, the half power bandwidth method uses the transfer function plot to obtain the damping. The method consists of determining the frequencies at which the amplitude of the transfer function is A_2 where

$$A_2 = \frac{A_1}{\sqrt{2}} \tag{48}$$

and A_1 is the amplitude at the peak. The frequencies f_a and f_b associated with the half power points on either side of the peak are obtained, as shown in figure 9. Then the damping ratio ζ is obtained using the formula

$$\zeta = \frac{f_b - f_a}{f_b + f_a} \tag{49}$$

The damping ratio associated with each natural frequency can be obtained using the half power bandwidth method.

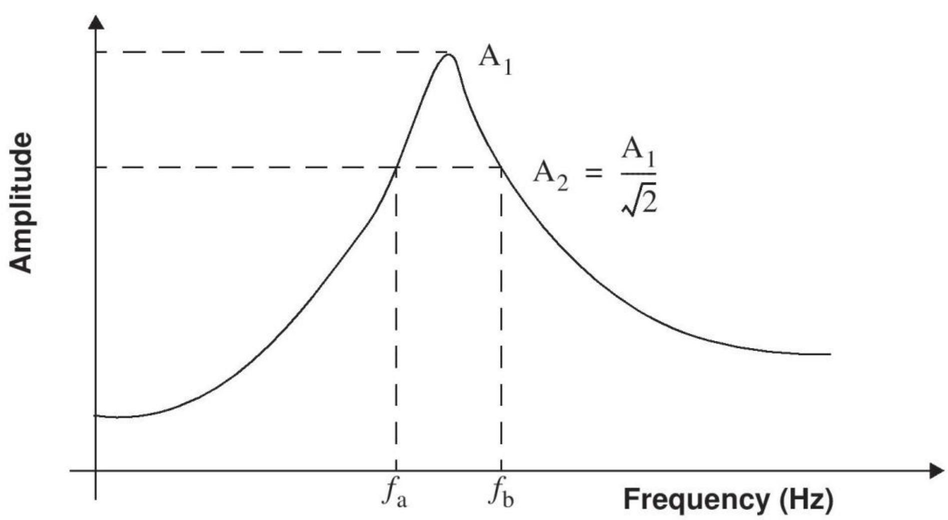


Figure 9. Half Power Bandwidth method.

3.0 Experimental Setup: Equipment

3.1 Required Equipment

- Data acquisition system (MultiQ board and computer)
- Instructional shake table
- Standard test structure
- Three accelerometers (one on the shake table)
- Power unit for sensors and shake table
- Relevant cables
- Software: Wincon and Matlab (signal processing toolbox required)
- Passive control device (optional, instructions provided)

3.2 Data Acquisition System

A data acquisition system is used to obtain measurements of physical quantities using sensors. These measurements may be temperature, pressure, wind, distance, acceleration, etc. In civil engineering applications the most common types of sensors measure displacement, acceleration, force and strain. In this experiment, we are going to use acceleration sensors to obtain records of acceleration over time for a simple model of a building.

Photos of the experimental components are shown in figure 10. The data acquisition system consists of a computer and a MultiQ board. Accelerometers are attached to each floor of the test structure to measure accelerations. A power supply is used to provide current to the accelerometers and to the shake table. The accelerometers are connected to the power supply. The ground accelerometer should be connected at “S1”, the first floor accelerometer should be connected at “S2”, and the second floor accelerometer should be connected at “S3”. The MultiQ board should be connected to the power supply as well. To make this connection use the appropriate cable to connect the “From MultiQ” on the power supply to the “Shaker X” on the MultiQ board. (Hint: Also, be sure that all the accelerometers are facing in the same direction as the ground accelerometer).

3.3 Shake Table

Figure 11 shows the bench-scale shake table used to excite the structure. This small shake table is a uniaxial shake table with a design capacity of 25 pounds. It is controlled by a computer which has the capability to excite the building with different types of signals including sine wave, random or step signals. With this instrument, it is also possible to reproduce an earthquake and study the characteristics of structures under specific earthquakes. A safety circuit is provided which stops the shake table in case it travels beyond the range of operation. To enable the shake table, one must depress the deadman button. The shake table stops when the deadman button is released. For a more detailed guide on how to operate the shake table see the “Bench-Top Shake Table User’s Guide” available in the University Consortium of Instructional Shake Tables web page (*http://ucist.cive.wustl.edu/*)

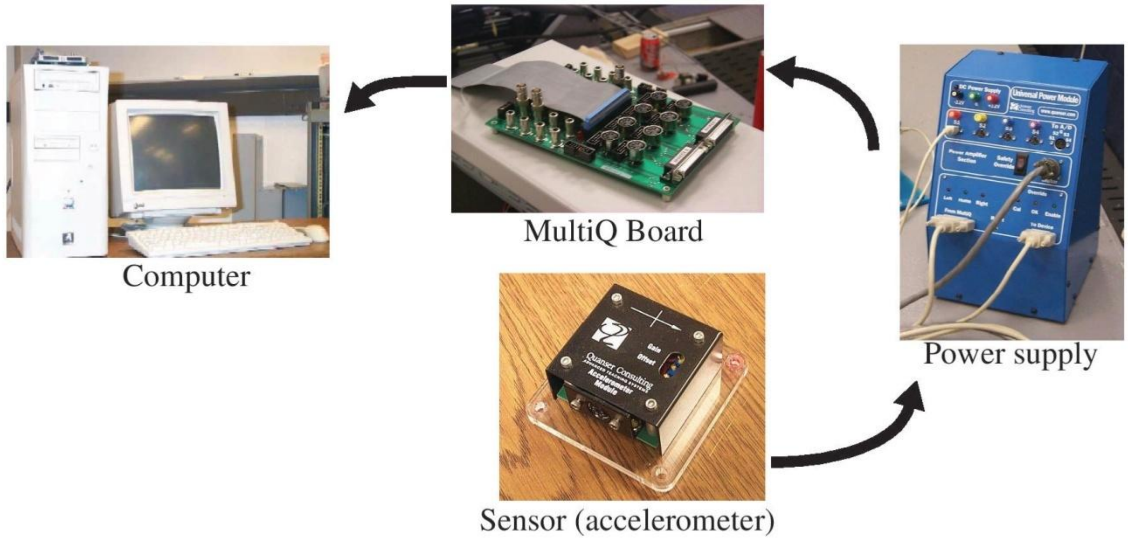


Figure 10. Data acquisition system.

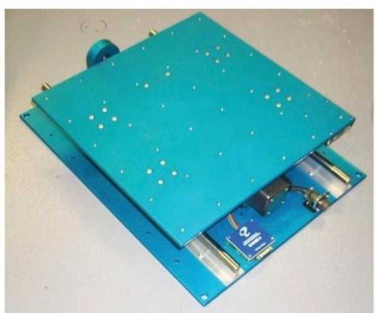


Figure 11. Bench scale shake table.

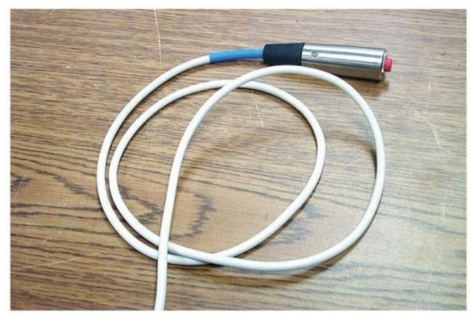


Figure 12. Deadman button.

3.4 Test Structure

The test structure is a simple model of a two story building. The building's height is 50 centimeters (19.68 inches) per floor and has a weight of 2 kilograms per floor. A small shock absorber can be attached to the structure as a passive control device and an active mass driver can be adapted to the structure as an active control device.

Figure 13. Test structure.

4.0 Experimental Procedure

<u>Important Notes: Safe Operation of the Shake Table</u>

- The "safety override" button on the power supply should *ALWAYS* remain in the off position.
- Turn the power supply *off* if you turn off or reboot the computer.
- The deadman switch must be depressed to excite the shake table. Press this button and hold it before you begin each segment of the experiment (before you hit the "Start" button on the Wincon server).
- It may be necessary to reboot the computer if it locks up during the test.

4.1 Transfer function calculation

1. Check that all the connections are correct (accelerometers, shake table, MultiQ board, *etc.*).
2. Turn on the power supply and wait to see that the right and left indicator lights blink.
3. On the computer, use Windows Explorer to open UCIST directory: C:/UCIST/boot
4. Double click on the file <boot.exe>. The left and right indicator lights on the power supply should stop blinking.
5. Double click on the shortcut <sample>. This program will start matlab and a figure window with a menu will appear (see figure 14). Now you are ready to begin the experiment.

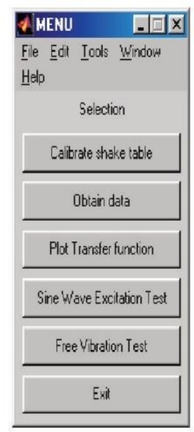


Figure 14. Menu.

6. Calibrate and center the shake table by clicking the first button of the menu "Calibrate shake table" (see figure 14). When the Wincon server starts hit the "Start" button to perform the calibration. The program will ask you if you can download the program and you should select "yes".
7. Click on the second button, "Obtain data", to run a sine sweep excitation. Hit the "Start" button on the Wincon server to generate the excitation. Graphs of the acceleration responses obtained with the data acquisition system will appear. Three plots should appear, one for the acceleration on the ground (shake table) and one for each floor of the building.
8. Click the third button "Plot transfer functions", to calculate the transfer functions. This will take a few seconds. Two transfer functions will be computed, including
   - Ground excitation to first floor
   - Ground excitation to second floor

When the plots are displayed another menu will appear. This tool is provided so that you can graphically determine the frequencies of the structure. You should identify the natural frequencies of the structure using this tool. Be sure to hit "quit" when you have identified all of the frequencies, or you will lock up the computer.

You will use the transfer function data to obtain the damping ratio using the half-power bandwidth method. Zoom in on the peaks in the transfer functions and make a printout of these plots so that you can obtain the peaks and the half-power points from the graphs. Remember that the data in the plot is in decibels.

> Please answer the following questions.
>
> - How many natural frequencies does the structure have?
> - What are the values of the natural frequencies?
> - Are these values the same in the two transfer functions? Why or why not?

4.2 Determination of Mode Shapes

In Section 4.1 you found the natural frequencies of the test structure. Now you are going to identify the mode shapes of the structure using a sine wave excitation. Use the next button of the menu, "Sine Wave Excitation Test" (see figure 14). A new window will appear with two dials. One dial allows you to change the frequency and the second allows you to change the amplitude. Make sure that the structure is at rest before starting the excitation. Please keep the amplitude low as you are exciting the structure at resonance. Also, change the frequency of the excitation slowly.

Excite the building with each of the natural frequencies obtained in Section 4.1. Turn the frequency indicator that appears on the screen until you reach the value you are looking for. The value of the frequency chosen appears above the control dial.

> Please do the following.
>
> - Sketch each of the mode shapes of the structure.
> - Obtain the number of nodes in each mode shape.
> - Does this result satisfy equation (35)? Explain.

4.3 Damping estimation

4.3.1 Exponential decay

In this test you will excite the structure with a finite duration sinusoidal excitation to examine the free response in each mode of the structure. The sinusoidal excitation lasts for 30 seconds, and then the structure is in free vibration. Then a record of the acceleration of the two floors of the structure will appear.

The next button on the menu, “Free Vibration Test” (See figure 14), will perform this test. When you hit this button a control panel will appear and you must insert the frequency into the blue box for each test. You must then hit the “Start” button on the Wincon server to begin the excitation. Be sure that the structure is at rest before performing this test. When the test is over, a plot will appear for the free vibration portion of the response. Do this test for each mode of the structure. Using these records obtain the damping ratio using the exponential decay method described in 2.4.1.

Please do the following.

- What is the damping ratio obtained using this method?
- Compare this damping ratio with that obtained in 4.3.2.

4.3.2 Half Power Bandwidth method

Use the half power bandwidth method (section 2.4.2) and transfer function obtained in section 4.1 to determine the damping in the structure.

Please do the following.

- From the transfer functions obtained in 4.1 estimate the damping using the half power bandwidth method described in 2.4.2. What is the damping ratio associated with each natural frequency?
- Compare the damping values for each of the two modes.
- Discuss the advantages and disadvantages of these two methods?

Formulation of equation of motion

the nonlinear governing equation of motions of multi-degree of freedom nonlinear systems will be derived by using Newton's 2^{nd} Law or d'Alembert's principle. The approach is similar to that of the single degree of freedom system. One can derive the equation of motion by drawing the free body diagrams and then writing the force or moment equilibrium equations by including the inertia force. Let us consider following simple examples to derive the equation of motions.

Example : Derive the equation motion of system shown in Fig.3.1 Consider the last spring to be nonlinear where the spring force is given by . Consider other spring and damper behaviour to be linear.

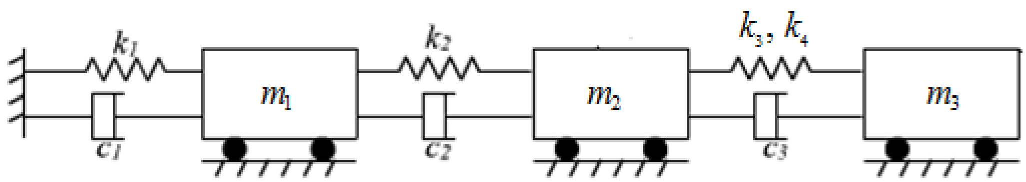


Figure 3.1. A multi degree of freedom system

Solution:

Considering the equilibrium of the mass m_1,

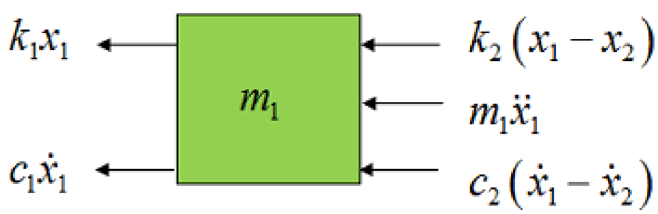


Figure 3.1.1: Free body diagram of part with mass m_1

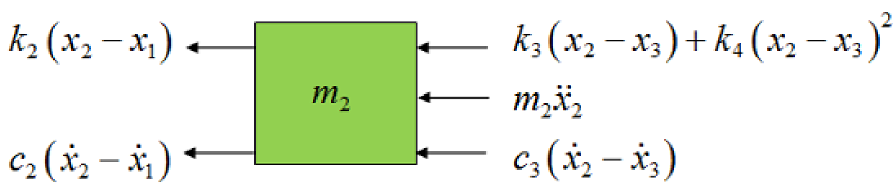


Figure 3.1.2: Free body diagram of part with mass m_2

Equating the forces acting on mass m_1 as shown in Fig. 3.1.1 one obtains

$$m_1\ddot{x}_1+k_1x_1+c_1\dot{x}_1+k_2(x_1-x_2)+c_2(\dot{x}_1-\dot{x}_2) = 0$$

Similarly considering the free body figure for 2^{nd} mass the equation of motion can be written as

$$m_2\ddot{x}_2+k_2 (x_2-x_1)+c_2(\dot{x}_2- \dot{x}_1)+k_4(x_2-x_3)+k_3(x_2-x_3)^2 +(\dot{x}_2-\dot{x}_3)$$

Free Vibration

In case of free vibration, the structure is not subjected to any dynamic excitation or external forces or support motion. The motion of a structure is influenced only by the initial conditions. For a vibrating system of ‘n’ degrees of freedom, the motion is represented by ‘n’ differential equation of motion. The methods such as Newton’s second law, Lagrange’s equation and influence coefficients method are used to form the equations of motion.

The solution of these equations may be obtained by matrix method, Stodola method and Holzer method. These are called approximate methods of determining natural frequencies.

Determination of natural frequencies of vibration and mode shapes

The equation of motion of an MDOF system subjected to free vibration is given as

$$[m][\ddot{x}]+[k][x] = \{0\} \quad \text{--------} \; 3.1$$

For an undamped free vibration system, the solution of equation of motion are in the form of $\{x\} = \{X \sin(\omega_n t+\emptyset)\}$ ------3.2a

And $\{\ddot{x}\}= -\omega_n^2\{ X \sin(\omega_n t+\emptyset)\}$ ------3.2b

where X is the amplitude of motion. After substituting of Eq.(8.2) into Eq. (8.1), we get $-\omega_n^2[M]\{X \sin(\omega_n t+\emptyset)\}+ [k]\{X \sin(\omega_n t+\emptyset)\}=\{0\}$

rearranging,

$$[[k]- \omega_n^2[M]]\{X\} = 0 \text{------} 3.3$$

The problem of determining the constant (ω_n^2) is an important mathematical problem known as characteristic value or eigen value problem. Its non-trivial solution is possible only when the determinant of the coefficient matrix vanishes, that is

$$|[k]- \omega_n^2[M]|=0 \text{-----} 3.4$$

The expansion of the determinant in eq.3.4 results in a algebraic equation of nth order in ω_n^2 which should be satisfied for n values ω_n^2. This equation known as characteristic equation of the system. The roots of the characteristic equation are called the **eigen** values which are nothing but natural frequencies ω_i of the MDOF system. The resulting simple harmonic motions can takes place in such a way that all the masses move in phase at the same frequency. For each natural frequency, the resulting deflected shape is known as normal mode shape or eign vector. The mode corresponding to the lowest frequency is called first mode or fundamental mode. The other modes are called higher modes. The normal modes depends on mass and stiffness properties of the system. The normal modes are completely determined by assuming unit values for the amplitude of motion at the first degree of freedom. So that the normal modes of the remaining coordinate are computed relatively. This process of normalizing each mode is called normalisation.

Orthogonal properties of normal modes

The mode shapes re eigen vectors are mutually orthogonal with respect to the mass and stiffness matrices. Orthogonality is the important property of the normal modes or eigen vectors and it is used to uncouple the modal mass and stiffness matrices.

As we know that, $\omega^2[M]\{Ø\} = [k]\{Ø\}$

For i^{th} mode (or) eigen value,

$$\omega_i^2[M]\{Ø\}_i = [k]\{Ø\}_i \quad \text{--------3.5}$$

For j^{th} mode,

$$\omega_j^2[M]\{Ø\}_j = [k]\{Ø\}_j \quad \text{--------3.6}$$

Multiply Eq.(3.5) by $\{Ø\}_j^T$ and Eq. (3.6) by $\{Ø\}_i^T$ to get

$$\omega_i^2\{Ø\}_j^T [M]\{Ø\}_i = \{Ø\}_j^T [k]\{Ø\}_i \quad \text{------3.7}$$

and $$\omega_j^2\{Ø\}_i^T [M]\{Ø\}_j = \{Ø\}_i^T [k]\{Ø\}_j \quad \text{------3.8}$$

By transposing LHS and RHS of Eq.(3.8), we get

$$\omega_j^2\{Ø\}_j^T [M]\{Ø\}_i = \{Ø\}_j^T [k]\{Ø\}_i \quad \text{--------3.9}$$

In which $[M]^T = [M]$

And $[k]^T = [k]$

Subtract Eq.(3.7) from Eq. (3.9), we get

$$(\omega_i^2 - \omega_j^2) \{Ø\}_j^T [M]\{Ø\}_i = 0$$

For $\omega_i \neq \omega_j$, we get

$$\{Ø\}_i^T [M]\{Ø\}_i = 0 \quad i \neq j \quad \text{--------- 3.10}$$

Similarly, $\{Ø\}_i^T [M]\{Ø\}_j = 0$

This condition is called the **orthogonality principle.**

Eigen vectors are unique; a scalar multiplier of an eigen vector is also an eigen vector.

For i^{th} eigen value,

$$\{Ø\}_i^T [M]\{Ø\}_i = \text{scalar} \quad \text{---------3.11}$$

If the scalar is equal to unity, the resulting eigen vectors are called normal modes, that is $\{Ø\}_i^T [M]\{Ø\}_i = I$ --------- 3.12

Where I is the identity matrix which is a diagonal matrix with unit values along the main diagonal. From Eq.(3.12), the natural modes are not only orthogonal but are normalized with respect to mass matrix. These two are called mass orthonormal set.

For i^{th} mode $\omega_i^2[M]\{Ø\}_i = [k]\{Ø\}_i$

Multiply by $\{Ø\}_i^T$,

$$\omega_i^2\{Ø\}_i^T [M]\{Ø\}_i = \{Ø\}_i^T [k]\{Ø\}_i$$

since, $\{Ø\}_i^T [M]\{Ø\}_I = 1$, $\omega_i^2 (1) = \{Ø\}_i^T[k]\{Ø\}_i$

$$\{Ø\}_i^T[k]\{Ø\}_I = \omega_i^2 \quad \text{--------(3.13)}$$

Equations (3.12) and (3.13) are called **normality principle**

Mode Superposition Method,

A set of coupled equations are transformed into a set of uncoupled equations through use of the orthogonal vectors of the system.

$[M]\{u\&\&\} + [K]\{u\} = \{ p(t)\}$ Equilibrium Equation

Figure 3 shows two masses with m1 and m2 three springs having spring stiffness k1, k2, k3 free to move on the horizontal surface. Let x_1 and x_2 be the displacement of mass m_1 and m_2 respectively

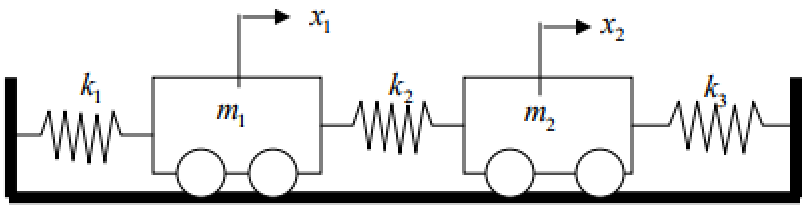
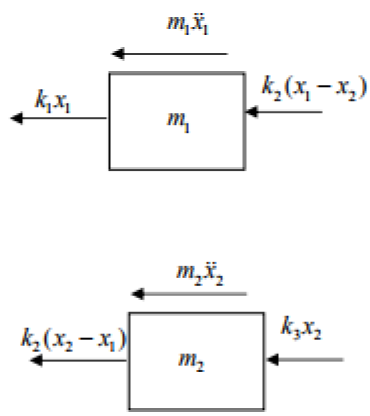


Figure 3

Free body diagram



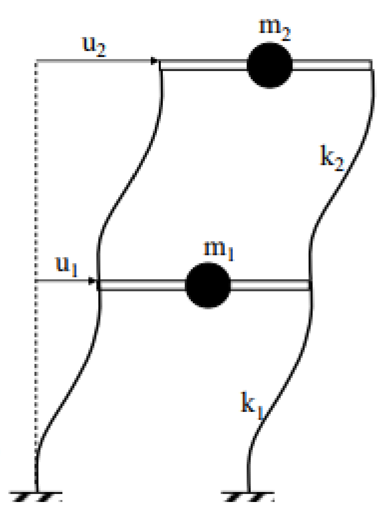
$m_1\ddot{x}_1+(k_1+k_2)x_1-k_2x_2=0$

$m_2\ddot{x}_2+(k_2+k_3)x_2-k_1x_1=0$

Now writing the equation of motion in matrix form

$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}\begin{pmatrix} \ddot{x}_1 \\ \ddot{x}_2 \end{pmatrix}+\begin{bmatrix} k_1+k_2 & -k_2 \\ -k_2 & k_2+k_3 \end{bmatrix}\begin{pmatrix} x_1 \\ x_2 \end{pmatrix}=\begin{pmatrix} 0 \\ 0 \end{pmatrix}.$$

2-Story Shear Building (2-DOF system)



$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}\begin{bmatrix} \ddot{u}_1 \\ \ddot{u}_2 \end{bmatrix}+\begin{bmatrix} (k_1+k_2) & -k_2 \\ -k_2 & k_2 \end{bmatrix}\begin{bmatrix} u_1 \\ u_2 \end{bmatrix}=-\begin{bmatrix} m_1 \\ m_2 \end{bmatrix}\ddot{u}_g$$

$$(\mathbf{k} - \lambda\mathbf{m})\mathbf{u} = \mathbf{0}$$

$$\begin{bmatrix} (k_1+k_2)-\lambda m_1 & -k_2 \\ -k_2 & k_2-\lambda m_2 \end{bmatrix}\begin{bmatrix} u_1 \\ u_2 \end{bmatrix}=\begin{bmatrix} 0 \\ 0 \end{bmatrix}$$

Set Determinant = 0:

$$\begin{vmatrix} (k_1+k_2)-\lambda m_1 & -k_2 \\ -k_2 & k_2-\lambda m_2 \end{vmatrix}=0$$

$$((k_1+k_2)-\lambda m_1)(k_2-\lambda m_2)-(-k_2)(-k_2)=0$$

$$(m_1m_2)\lambda^2-((m_1k_2+m_2(k_1+k_2))\lambda+(k_1k_2)=0$$

Solve for the λ_1 and λ_2 using the standard approach

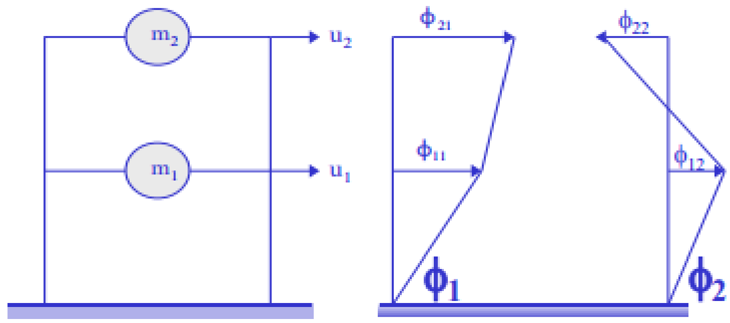
$$a\lambda^2+b\lambda+c=0 \qquad \lambda_{\frac{1}{2}}=\frac{-b\pm\sqrt{b^2-4ac}}{2a}$$

Note: For $\begin{bmatrix} W & X \\ Y & Z \end{bmatrix}$

Determinant = WZ-XY

For a non-trivial solution (which will allow for computing the natural frequencies during free vibration), the determinant of $|k-\omega^2 m| = 0$ where $\omega=\lambda$ corresponding mode shape can be obtained from $(k-\omega^2 m)\,\acute{\O}_n = 0$

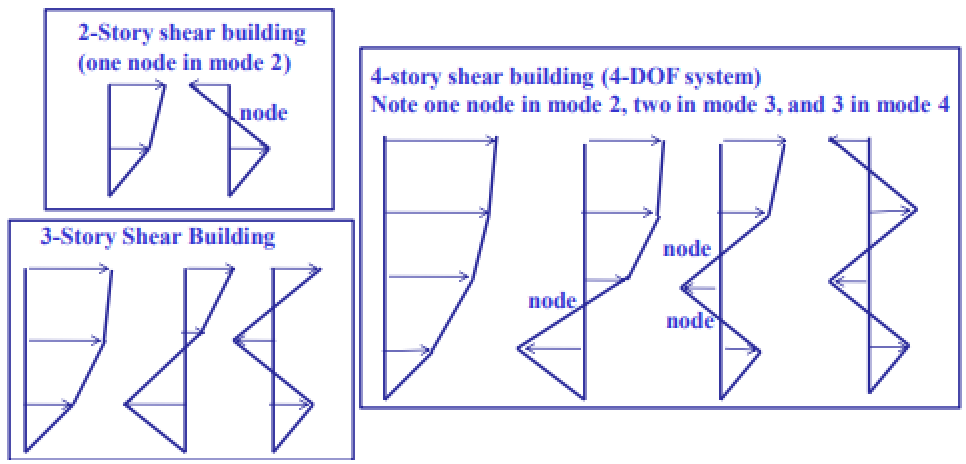
2-DOF system (2 mode shapes ϕ_1 and ϕ_2)



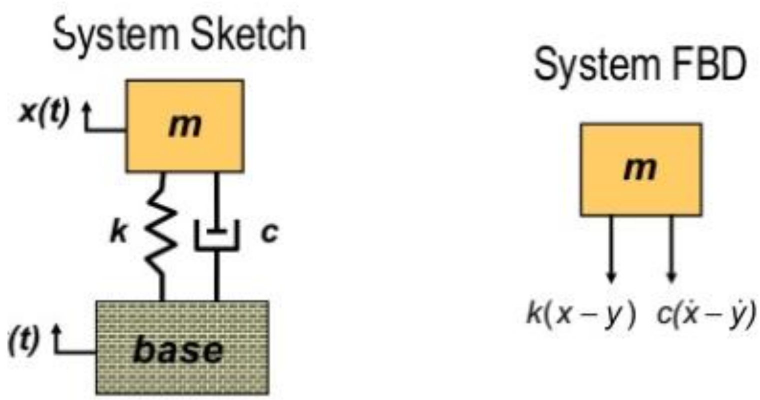
$$\phi_1 = \begin{bmatrix} \phi_{11} \\ \phi_{21} \end{bmatrix}$$

$$\phi_2 = \begin{bmatrix} \phi_{12} \\ \phi_{22} \end{bmatrix}$$

Sample Mode shape Configurations



Rigid base excitation



$$\sum F = -k(x-y) - c(\dot{x}-\dot{y}) = m\ddot{x}$$

$$m\ddot{x} + c\dot{x} + kx = c\dot{y} + ky$$

UNIT-III
EARTHQUAKE ANALYSIS

GENERAL PRINCIPLES

Clause 6.1 of IS 1893 (part 1): 2002 provides the following seismic design principles:

I. The random click click ground motions which cars the structure to vibrate can be resolved in any three mutually perpendicular directions. The predominate direction of ground motion is usually horizontal.

II. Earthquake generated vertical inertia forces are to be considered in design unless it is not sufficient. Vertical acceleration should be considered in structures with large spans, and criteria should contain design for overall stability of structures. Reduction in gravity force due to vertical component of ground motions can be particularly detrimental in case of precious precious horizontal members and of cantilevered members. Hence, special attention should be paid to the effect of vertical component of the ground motion on prestressed or cantilever beams girders and slabs.

III. The response of structure to ground vibrations is a function of the nature of foundation soil materials form size and mode of construction of structures and the duration and characteristics of ground motion this standard specifies designer forces for structures standing on rocks or soils which do not settle or liquify or side due to loss of stunt during ground motions.

IV. the design approach adopted in this code is to ensure that structures process at least a minimum strength to

1. Withstand minor earthquake (design based earthquake) which may occur frequently without damage.

2. Resist moderate earthquakes (design based earthquake) without significant structural damage through some non structural damage may occur;

3. With stand a major earthquake (maximum considered earthquake) without collapse.

V. Actual forces that appear on success during earthquakes are much greater than the design forces specified in this code. However, ductility arising from the inelastic material behavior, and detailing and overstrength arising from the additional reserve strength in structures over and above the design strength, are relied upon to account for this difference in actual and design loads.

VI. The design lateral forces specified in this standard shall be considered in each of the two orthogonal horizontal directions of the structure for structures which have lateral force resisting element in the two orthogonal directions only, The design lateral force shall be considered along one direction at a time and not in both directions simultaneously. Structures having lateral force resisting elements in directions other than the two orthogonal directions, shall be analysed considering the load combinations specified in section 14.3.2.

LOAD COMBINATION AND INCREASE IN PERMISSIBLE STRESSES

Load Combinations

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

1. 1.5(DL+IL)
2. 1.2(DL+IL+EL)
3. 1.5(DL+EL)
4. 0.9(DL+1.5EL)

Where DL, IL and EL denote dead, imposed and earthquake loads in the plastic design of steel structures the following load combinations shall be accounted for

1. 1.7(DL+IL)
2. 1.7(DL+EL)
3. 1.3(DL+IL+EL)

Design Horizontal Earthquake Load

There are two cases:

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

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

Design Vertical Earthquake Load

When effects due to vertical earthquake loads are to be considered, design vertical force shall be calculated with two thirds of the design horizontal acceleration spectrum specified.

Combination of Two or Three Component Motion

There are 3 cases:

I. When responses from the three separate components of to be considered, the responses due to each component may be combined using the assumption that when the maximum response from one component of course the responses from the other two components are 30% of their maximum. All possible combinations of the three components (EL_x,El_y and EL_z) including variations in sign (±) shall be considered. Thus, the response due to earthquake force(EL) is the maximum of the following three cases:

1. ±ELx±0.3ELy±0.3ELz
2. ±Ely±0.3ELx±0.ELz
3. ±ELz±0.3ELx±0.3Ely

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

II. The response you to dance combined effect of the three components can be obtained on the basis of **square root of the sum of the square** **(SRSS)** that is

$$EL = \sqrt{(EL_x)^2 + \left(EL_y\right)^2 + (EL_z)^2}$$

III. When two component motions (say one organ and one vertical or only two horizontal) are combined, the equations in Eqs. 14.1 And 14.2 should be modified by deleting the term representing the response due to the component of motion not being considered.

Increase in Permissible Stresses

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

DESIGN SPECTRUM

1. For the purpose of determining seismic forces the country is classified into 4 seismic zones as shown in figure

The provision suggests a common designer spectrum for both sesame Coefficient method and response spectrum method.

2. the design horizontal seismic Coefficient A_h for a structure shall be determined by the following expression:

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

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

Where

Z - Zone factor given in table 14.1 is for the maximum considered earthquake and earthquake and service life II of the structure in a zone. The factor 2 in the denominator of Z is used so as to reduce the maximum considered earthquake zone factor to the factor of Design Basis Earthquake.

I - Important factor depending upon the functional use of the structure characterized by hazards consequences of its failure postpaid functional needs historical value or economic importance

R - Response reduction factor depending on the persuade seismic damage performance of the structure characterized by ductile or brittle deformation however the ratio (I/R) shall not be greater than 1.0 tables 14.4 the values of R for buildings are given in table 14.4.

$\frac{S_a}{g}$ - Average acceleration Coefficient for rock or file sites has given by figure 14.2 and Table 14.2 are based on appropriate natural periods and damping of the structures. These curves represent free field ground motions.

Table 14.1 zone factor, Z

Seismic	**II**	**III**	**IV**	**V**
Seismic intensity	low	moderate	severe	very severe
Z	0.10	0.16	0.24	0.36

3. Where a number of nodes are to be considered for dynamic analysis the value of as defined in equation 14.3 for each node shall be determined using natural period of vibration of that mode.

4. For underground structures and foundations at the depth of it matters or below the horizontal spectrum value shall be taken as faster value obtained from equation 14.3 for structures and foundations placed between the ground level and 30 m depth the design horizontal acceleration Spectrum Mall you shall be linearly interpolated between A_h and $0.5A_h$ where is as specified in equation 14.3

5. The design of acceleration spectrum for vertical motions, when required, may be taken as two-thirds of the design horizontal acceleration spectrum specified in equation 14.3.

figure 14.2 shows the proposed 5% spectra for Rocky and soil sites and table 14.2 used by multiplying factor for obtaining spectral values for various other damping values.

For Rocky and hard soil sites,

$$\frac{S_a}{g} = \begin{cases} 0.00 \leq T \leq 0.10 \\ 0.10 \leq T \leq 0.40 \\ 0.40 \leq T \leq 4.00 \end{cases}$$

For medium soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 1.5T & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.55 \\ \dfrac{1.36}{T} & 0.55 \leq T \leq 4.00 \end{cases}$$

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 1.5T & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.40 \\ \dfrac{1.67}{T} & 0.67 \leq T \leq 4.00 \end{cases}$$

6. In case of design spectrum is specifically prepared for hey structure at a particular project site the same may be used for designer at the description of the project authorities.

Table 14.2 Multiplying factors for obtaining spectral values for various other damping values

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

Table 14.3 Importance factors, I

S.No.	Structure	Importance Factor
(i)	important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchanges; television stations; radio stations, railway stations, fire stations buildings; largest community halls like cinemas, assembly halls and Subway stations, power stations.	1.5
(ii)	All other buildings	1.0

Table 14.4 Response reduction factor, R for building systems

S.No	Lateral load resisting system	R
	Building Frame system	
i.	Ordinary RC Moment Resisting Frame (OMRF)[2]	3
ii.	Special RC Moment Resisting Frame (SMRF)[3]	5
iii.	Steel Frame with	
(a)	Concentric braces	4
(b)	Eccentric braces	5
iv.	Steel moment Resisting Frames designed as per SP:(6)	5
	Building with shear walls[4]	
	Load bearing Masonry Wall buildings[5]	
v.	(a) Unreinforced	1.5
	(b) Reinforced with horizontal RC bands	2.5
	(c) Reinforce with horizontal RC bands and vertical bars at corners of rooms	

	And jambs of openings	3
vi.	Ordinary reinforced concrete shear walls[6]	4
vii.	Ductile shear walls	4
	Buildings with Dual systems[8]	
viii.	Ordinary shear wall with OMRF	3
ix.	Ordinary shear wall with SMRF	4
x.	Ductile shear wall with OMRF	4.5
xi.	Ductile shear wall with OMRF	5

Note 1: The above values of response correction factors are to be used for buildings with lateral load resisting elements and not just for the lateral load resisting elements built in isolation.
Note 2: OMRF are those design and details as per IS 456 or IS 800 but not for IS 13920 or SP(6)
Note 3: SMS has been defined in Section 4. 1.5.2.
Note 4: Buildings with shear walls also include buildings falling share balls and frames but where
(a) Frames are not designed to carry lateral loads or
(b) Frames are designed to carry lateral loads but not fulfill the requirements of dual systems.
Note 5: reinforcement should be as per IS 4326.
Note 6: prohibited in Zones IV and V.
Note 7 ductile shear walls are those design and detailed as per IS 13920.
Note 8: Buildings with real dual systems consist of shear walls or braced frames Anna moment resisting friends such that

(a) The two systems are designed to resistor the total force in proportion today lateral stiffness stiffness considering the interaction of the dual systems at all floor levels; and

(b) The moment resisting frames are designed to independently register at least 25% of the designe seismic base shear.

DYNAMIC ANALYSIS

General

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

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

b) **Irregular buildings:** All framed buildings higher than 12 m in Zones IV and V and those greater than 40 m in height in Zone II and III.

c) The analytical model for dynamic analysis of buildings with unusual configuration should be such that it adequately models the types of irregularities present in the building configuration. Buildings with plan irregularities, as defined in table 14.4, cannot be modelled for dynamic analysis by the method given in clause 7.8.4.5.

Note: for irregular buildings, less than 40m in height in Zones II and III, dynamic analysis, even though not mandatory, is recommended.

Methods of Dynamic Analysis

Dynamic analysis performed either by the time history method or by the Response Spectrum Method. However, in either method, the design base shear (V_B) shall be compared with a base shear ($\bar{V}_B$) calculated using a fundamental period T_a, where T_a, is as per section 14.5.4. Where V_B is less than $\overline{V_B}$ all the response quantities (for example, member forces, displacements, storey forces, storey shears and base reaction) shall be multiplied by $\overline{V_B}$ / V_B.

Amount of Damping

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

Time History Method

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

Response Spectrum Method

Response Spectrum Method of analysis shall be performed using the design spectrum specified in, or by a site-specific design spectrum mentioned in section 14.4.6.

Free Vibration Analysis

Undamped free vibration analysis of the entire building shall be performed as the established methods of mechanics using the appropriate masses and elastic stiffness of the structural systems, to obtained natural periods (T) and mode shaped (φ) of those of its modes of vibration that need to be considered as per Section 14.6.4.2.

Modes to be considered

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

Analysis of the Building Subjected to Design Forces

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

Modal Combination

The peak response quantities (for example, member forces, displacement, storey forces, storey shears and base reactions) shall be combined as per complete quadratic combinations (CQC) method.

$$\lambda = \sqrt{\sum_{i=1}^{r}\sum_{j=1}^{r} \lambda i\ \rho ij\ \lambda j}$$

Where,

r = number of modes been considered

p_{ij} = Cross model coefficient

λ_i = response quantity in node i (including sign)

λ_j = response quantity in mode j(including sign)

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

ρ = model damping ratio (in fraction) as specified in section 14.6.2.1

β = frequency ratio = ω_j/ω_i

ω_i = Circular frequency in i^{th} mode, and

ω_j = Circular frequency in j^{th} mode.

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

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

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

Where

λ_k = Absolute value of quantity in mode k and r = Number of modes being considered.

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

$\lambda^* = \sum_c^r \lambda_c$

Where the summation is for the closely-spaced modes only. This peak response quantity due to the closely-spaced modes (λ*) is then combined with those of the well separated modes by the method described in section 14.6.4.4(a).

Calculation of design lateral forces due to all modes

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

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

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

Where

g = Acceleration due to gravity,

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

w_I = seismic weight of floor i.

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

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

(c) **Design lateral Force at Each Floor in each mode:** The peak lateral force (Q_{ik}) at floor I in mode K is given by:

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

Where,

A_k = design horizontal acceleration Spectrum value as per section 14.6.2 using the natural period of vibration (T_k) of mode k.

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

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

(e) **Storey Shear Forces due to all Modes Considered:** The peak storey shear forces (V_i) in storey I due to all modes considered is obtaining by combined those due to each mode in accordance with section 14.6.4.4.

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

$F_{roof} = V_{roof}$ and

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

TORSION

General

Provision shall be made in all buildings for increase in shear forces on the lateral foci resisting elements resulting from the horizontal torsional moment arising due to eccentricity between the centre of mass and centre of rigidity. The design forces calculated as in section 14.6.4.5 are to be applied at the centre of rigidity. The design forces calculated as in section 14.6.4.5 is to be applied at the centre of mass appropriately displaces so as to cause design eccentricity (section 14.7.2) between displaced centre of mass and centre of rigidity. However, negative torsional shear shall be neglected.

Design Eccentricity:

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

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

Whichever of those gives the more severe effect in the shear of any frame

Where,

e_{di}= static eccentricity e_{si} at floor I defined as the distance between centre of mass and centre of rigidity, and

b_i = floor plan dimension of floor I, perpendicular to the direction of force

Additive Shear

In case of highly irregular building analyse according to section 14.6.4.5, additive shears will be superimposed for a statically applied eccentricity of ± 0.05 b_i with respect to the centre of rigidity.

UNIT IV
CODAL PROVISIONS AND DUCTILE DETAILING

IMPACT OF DUCTILITY

The structural engineering must have to understand the impact of ductility on the building response when it is subjected to earthquake force. For example consider a single degree of freedom system consisting of a metal rod and weight, as shown in fig. As the ground moves or displace, the characteristics of the ground to weight connection will play a vital role. If this connection is very rigid, the weight will experience the same or larger forces as compared to ground force but if the connection is very flexible as is in case of a metal rod, it will bend or deform and the weight will subjected to lesser forces because some of the energy will be consumed to displace the system. Most of the building responses under earthquake are within these two extremes.

From this simple example we can easily conclude that ductility, properly induced in the building system, will improve the behaviour of the building- primarily by refusing the forces in the structure. Therefore, ductility is an essential attribute of an earthquake resistant design of structure that serves as a shock absorber in the structure and reduces the transmitted force to one that is sustainable.

REQUIREMENTS OF DUCTILITY

In order to achieve a ductile structure we must give stress on three key areas during the design process. Firstly, the overall design concept for the building configuration must be sound. Secondly, individual members must be design for ductility, and finally connection and other structural details need careful attention. It is well recognized and accepted analysis of experimental results and analytical studies, that in earthquake resistant design of structures, all structural members and their connections and supports that is all critical region whose yielding strength may be reached and exceeded by a severe earthquake, should be designed (sized and detailed) with large ductility and stable hysteresis behaviour so that the entire structure will remain ductile displaying stable hysteresis behaviour. There are two main reasons for this ductility requirement: first, it allows the structure as a whole, to develop its maximum potential strength, through distribution of internal forces, which is given by the combination of maximum strengths of all components; and second, large structural ductility allows the structure to move as a mechanism under its maximum potential strength, resulting in the dissipation of large amount of energy (Bertero, 1991).

FACTORS AFFECTING DUCTILY

Some important factors on which the ductility will depend are:

i. Ductility increases linearly with an increase in the shear strength carried by concrete for small value of axial compressive stress ($0 \le \sigma_0 \le 1$Mpa)

ii. Ductility linearly reduces up to the point where axial compressive stress becomes equals to the axial compressive stress at balanced failure.

iii. With the increase as ultimate strain of concrete, the ductility factor increases. Thus confining of concrete the ductility appreciably.

iv. An increase in yield strength of steel with all other variables constantly decreases ductility. The ductility increases with increases the concrete strength.

v. The lateral reinforcement tends to improve ductility by preventing shear failures, restraining the compression steel against buckling. The lateral reinforcement in the form of closed stirrups is effective in binding the compression zone there by confining the concrete and increasing the ductility of the section.

vi. Shear failure occurs at a smaller deflection than the flexural failure and hence absorbs much less energy. Members should be designed and detailed by providing web reinforcement so that their strength in shear exceeds the strength in flexure. Therefore, ductility increases as the stirrups in the specimen increases.

vii. Bond failure and anchorage failures are sudden and brittle, special attention must be given in details to prevent them from occurring in structures, which must behave in a ductile manner.

DUCTILE DETAILING CONSIDERATIONS AS PER IS13920

General specifications

1. The design and construction of the reinforced concrete buildings shall be governed by the provisions of IS 456 : 1978 (now IS 456 : 2000), expect as the modified by the provisions of this code.
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 the 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.

3. Steel reinforcements of grade Fe 415 or less shall be used

Explanations:

- 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 in advantageous. Provided that the yield strength is confined to specified limits, design can determine section maximum flexural strengths in order to design other areas of the structure to prevent premature brittle shear failure (capacity design approach).
- 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 > 300 Mpa) is appropriate for other structural elements where flexural yielding can't occur under earthquake load.

FLEXURAL MEMBERS

General

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

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

Explanation:

- Generally, axial force in the flexural member is relatively very less but if factored axial compressive stress in the frame member exceeds to $0.1f_{ck}$, axial force will also be considered besides bending and member will be designed as per clause 7.0.

The member shall preferably have a width to depth ratio of more than 0.3.

Explanation:

- To provide more uniform design approach
- To minimize the risk of lateral instability
- Experience gain from past

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

Explanation:

- To decrease the sensitivity to geometric error
- Experience gained from practice with R.C. frame resisting earthquake induced forces

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

Explanation:

- To take into account the non linearity 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.

Longitudinal reinforcement

(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 reserved loading
- It is a construction requirement rather than behavioural requirements

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

Explanation:

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

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

Explanations:

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

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

Explanations:

- 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.

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

Explanations:

- This is to ensure some positive and negative moment capacity throughout the beam in ordered to allow unexpected deformations and moment distribution from the severe earthquake action
- To allow the possibility of the positive moment at the end of the beam due to earthquake induced lateral displacements exceeding the negative moments due to gravity loads

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

Explanations:

- 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 longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at spacing not exceeding 150mm. 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 earthquake forces. Not more than 50 percent of the bars shall be spliced at on section.

Explanations:

- Lap splices of reinforcement are prohibited at region 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 concrete cover

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

Explanations:

- Welded splice are one in which the bars are lap welded are but welded to develop the breaking strength of the bar
- A mechanical connection is a connection which release on mechanical inter lock with the bar deformations to develop the connection capacity
- In a structure undergoing inelastic deformation during an earthquake, tensile stress in reinforcement may approach the tensile strength of the reinforcement. The requirement of welded splice and mechanical connections is intended to avoid a splice failure where the reinforcement is subjected to expected stress levels in yield regions.
- The location of welded splice is restricted because tensile stress in reinforcement in yielding regions cannot exceed the strength requirement

Web reinforcement

Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrups having a 135^0 hook with a 10 diameter extension (but not < 75 mm) at each end that is emended in the confined core. In compelling circumstance, it may also be made up of two splices of reinforcement: AU stirrup with a 135^0 hook and a 10 diameter extension (but not < 75 mm) at each end, embedded in the confined core and cross tie. A cross tie is a bar having a 135^0 hook with a 10 diameter extension (but not <75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

Explanations:

•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 the strength degradation

•To provide shear strength for full flexural capacity of the member

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

Explanation:

This refers to construction and durability (corrosion of reinforcement) rather than behavioural requirements

The shear force to be resisted by the vertical hoops shall be the maximum of: (a) calculated factor shear force as per analysis, and (b) shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span.

Explanations:

•Actual force that appear on structures during earthquake are much higher than the design force specified in the code, it is assumed that frame member will dissipate energy in the nonlinear range response, unless a frame member posses a strength that is multiplied on the order of 3 or 4 of the design force. It is desirable that the beams should yield in flexural before failure in shear

•The design shear force should be a good approximation of the maximum shear that may develop in a member at any event. Therefore, required shear strength for frame member is related to flexural strength of the designed member rather than factored shear force indicated by lateral load analysis.

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

Explanation:

•Spalling of the concrete shall be anticipated during strong motion, especially at and near regions of flexural yielding, all web reinforcement should be provided in the form of closed hoops

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

Explanations:

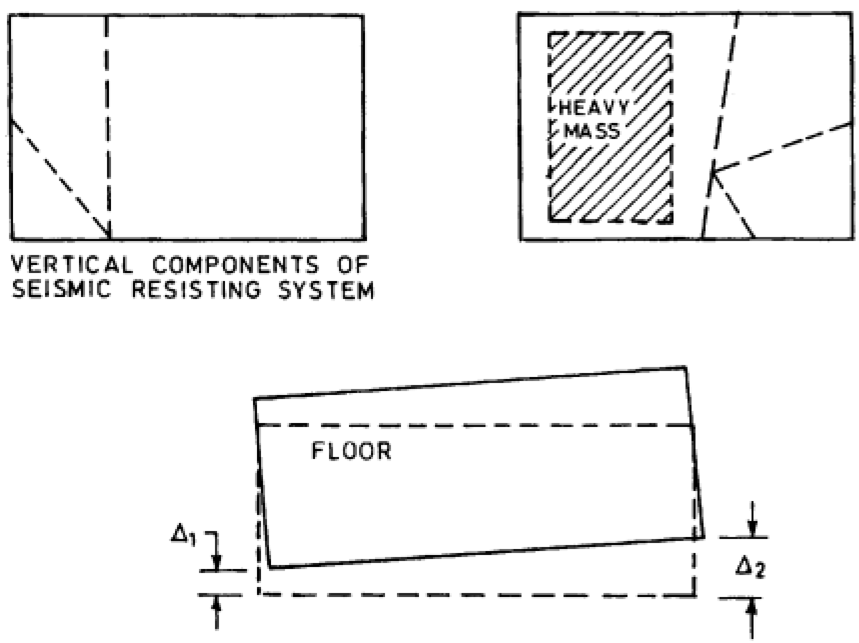
•Potential plastic hinge regions in beams require special detailing where plastic hinge develops. It severs three main purposes (I) prevents buckling of longitudinal bars in compression; (ii) provides some confinement of the concrete ; and (iii) acts as a shear reinforcement

•In the case of members with varying strength long the span or member for which the permanent load represents a large proportions of the total design load, concentration of inelastic rotation may occur within the span. If such a condition is anticipated, transverse reinforcement should also be provided in regions where yielding is expected

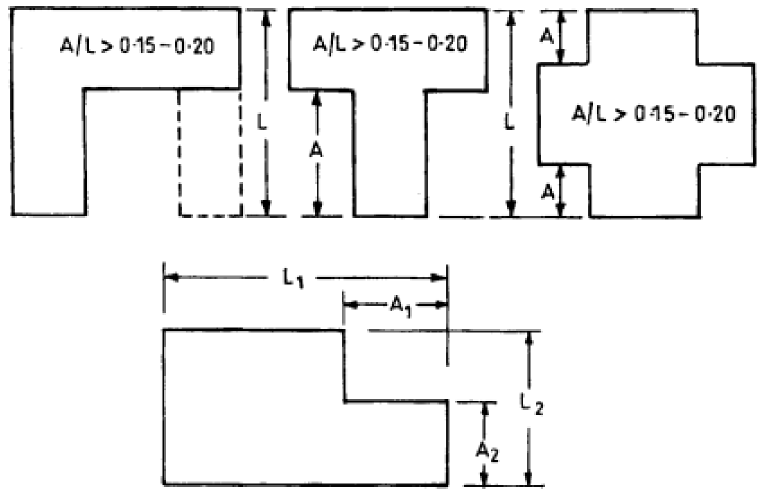
UNIT-V
SEISMIC PLANNING DESIGN OF SHEAR WALL

Definitions of Irregular Buildings — **Plan Irregularities**

- Irregularity Type and Description (2)
- **Torsion Irregularity** To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure



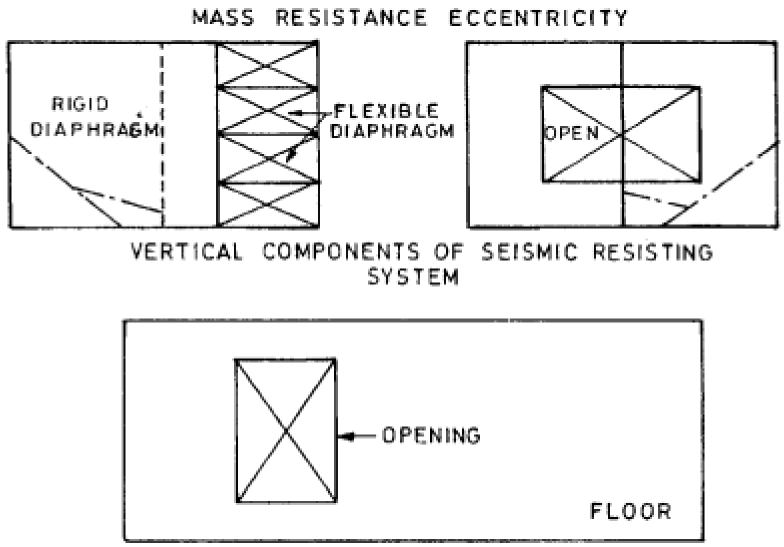
3 A Torsional Irregularity



3 B Re-entrant Corner

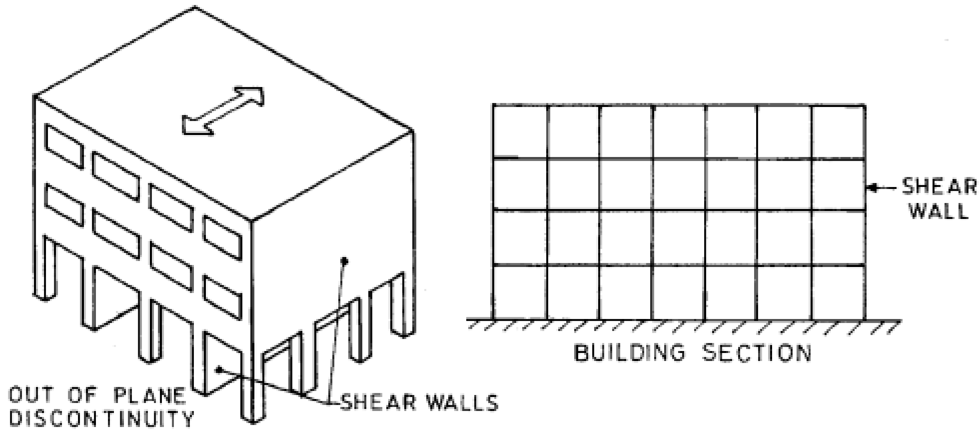
FIG. 3 PLAN IRREGULARITIES — *Continued*

- **Re-entrant Corners** Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction
- **Diaphragm Discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next

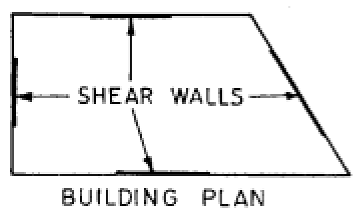


3 C Diaphragm Discontinuity

- **Out-of-Plane Offsets** Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements
- **Non-parallel Systems** The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements



3 D Out-of-Plane Offsets



3 E Non-Parallel System

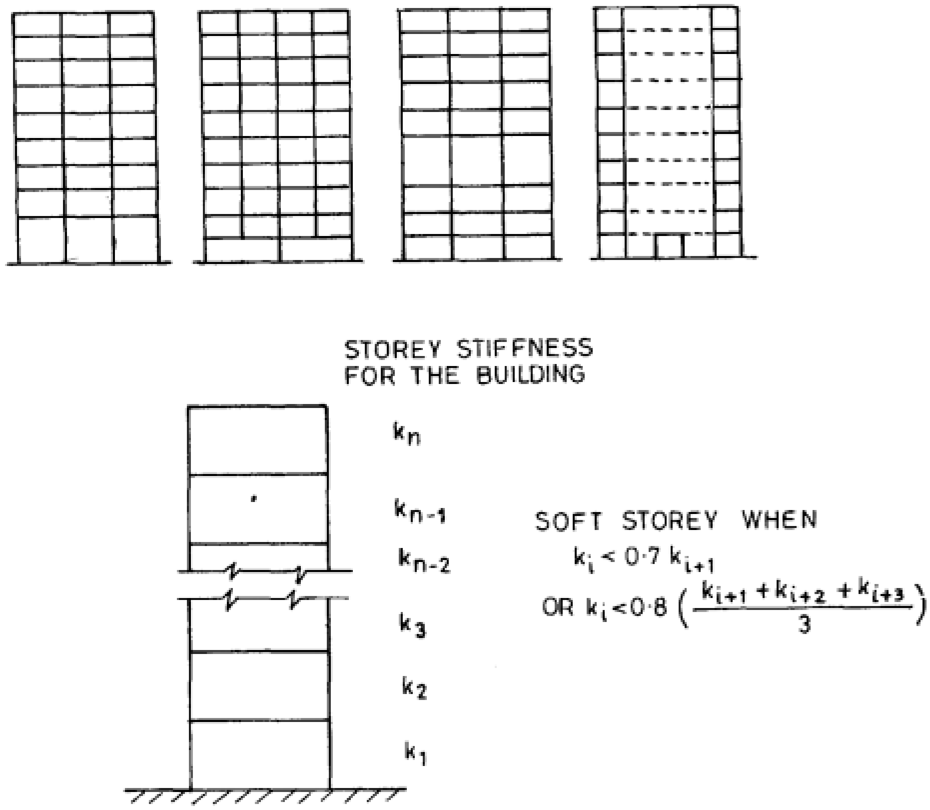
FIG. 3 PLAN IRREGULARITIES

Definition of Irregular Buildings —

Vertical Irregularities (Clause 7.1) S1 No. Irregularity Type and Description

a) Stiffness Irregularity — Soft Storey A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above

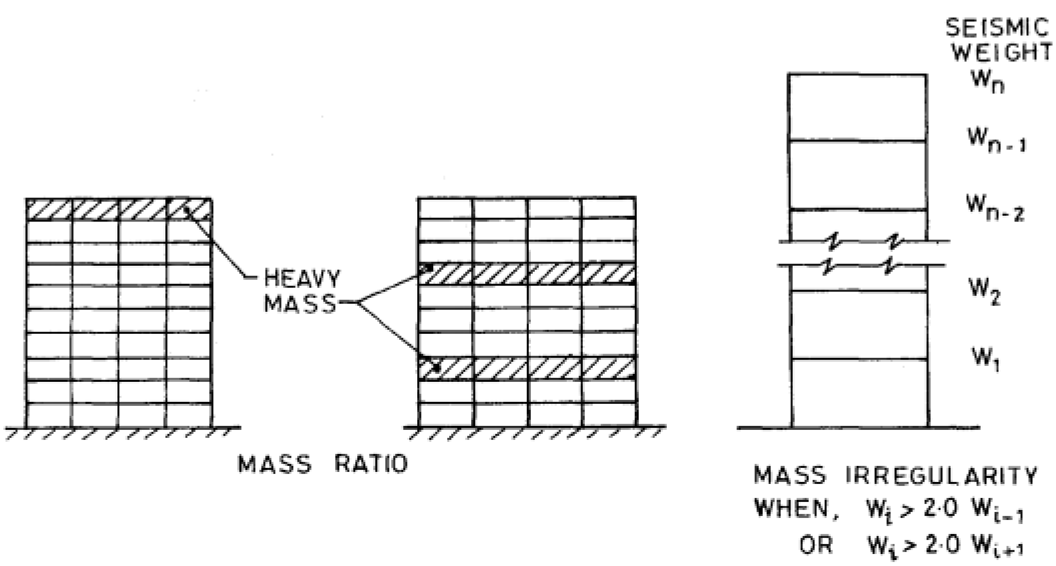
b) Stiffness Irregularity — Extreme Soft Storey A extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent ofthe average stiffness of the three storeys above. For example, buildings on STILTS will fall under this category



4 A *Stiffness Irregularity*

Irregularity Type and Description

Mass Irregularity: Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs Vertical Geometric Irregularity

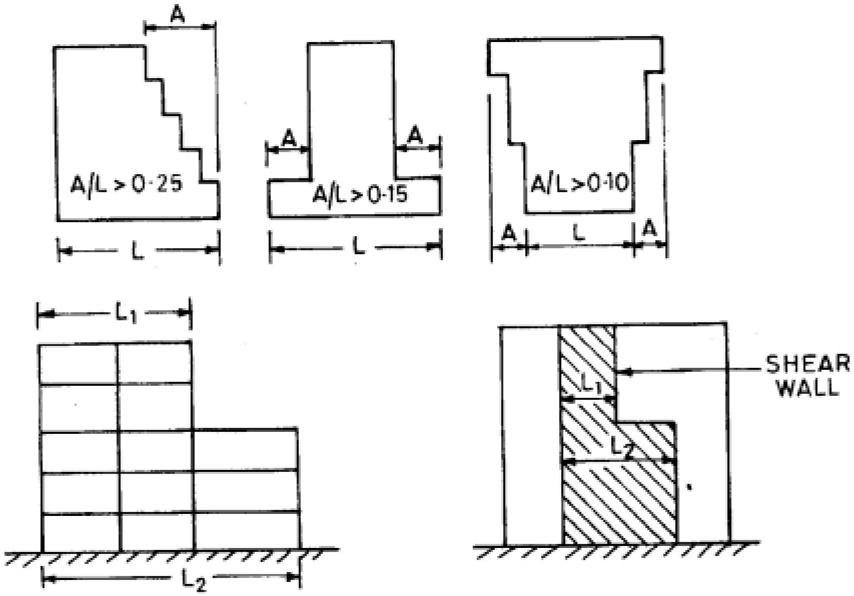


4 B Mass Irregularity

FIG. 4 VERTICAL IRREGULARITIES — *Continued*

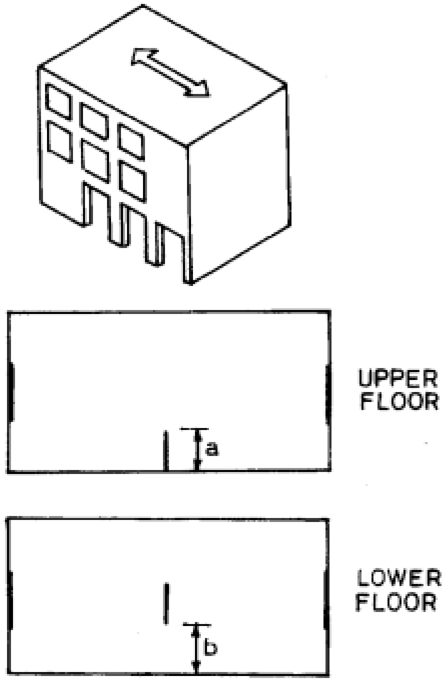
Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey

In-Plane Discontinuity in Vertical Elements Resisting Lateral Force .A in-plane offset of the lateral force resisting elements greater than the length of those elements

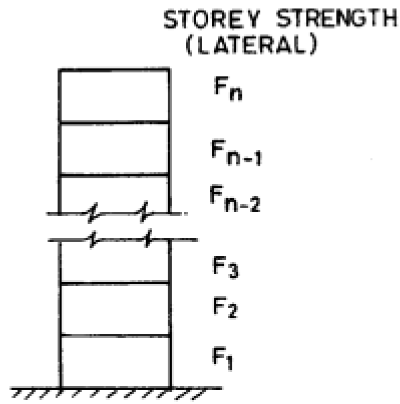


4 C Vertical Geometric Irregularity when $L_2 > 1.5\ L_1$

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



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



4 E Weak Storey when $F_i < 0.8\ F_i + 1$

FIG. 4 VERTICAL IRREGULARITIES

UNIT-VI
LATERAL LOAD ANALYSIS OF MASONRY BUILDINGS
INTRODUCTION :

Masonry buildings are widely used for housing construction not only in India but in many other countries of the world. There are innumerable advantages of masonry construction over both types of construction i.e. reinforced concrete and steel such as, thermal comfort, sound control, possibility of addition and alteration after construction, less formwork, easy and inexpensive repair, use of locally available materials, need of less skilled labour, less engineering intervention etc. However, there are some disadvantages as well, particularly, when it is built in seismic environment. The seismic resistance capacity of masonry construction is relatively low in comparision to engineered constructions. Therefore, many developed nations have imposed certain restrictions on the use of unreinforced masonry constructions. However, in developing nations unreinforced masonry construction is still being used frequently. In India, masonry constructions are generally made by using locally available materials like stone, brick, timber, adobe, mud etc. and are constructed in a traditional manner with or without the earthquake resistant features mentioned in IS : 4326 and 13927. Therefore, this type of construction is treated as non-engineered construction and most of the casualities are due to collapse of these constructions in earthquakes. Moreover the plight is that even after gaining knowledge of any earthquake engineering since the last three decades, neither a proper method has been developed for the seismic analysis and design of masonry buildings nor the topic is fairly covered in the current Indian curriculum inspite of the fact that about 90% population of India lives in masonry buildings. The present and subsequent chapters are a step towards with regard to develop a procedure seismic analysis and design of masonry buildings may also be designed as engineered constructions.

PROCEDURE FOR LATERAL LOAD ANALYSIS OF MASONRY BUILDINGS :

To understand the proper design procedure for low-rise masonry buildings, this procedure is divided into several distinctive steps. In actual practice, these various steps may not be so clearly delineated nor so distinctly separated, but at this stage, atleast, this step-by-step procedure is recommended in order to understand it properly. Figure shows masonry building subjected to a lateral load and its resisting mechanism. In load bearing masonry buildings, the walls, which carry gravity loads, also act as shear walls to resist lateral load. The structural wall parallel to lateral force and subjected to in-plane(shear) forces and bending are called shear walls. The walls perpendicular to seismic force/lateral force and subjected to out-of-plane bending are called flexural walls.

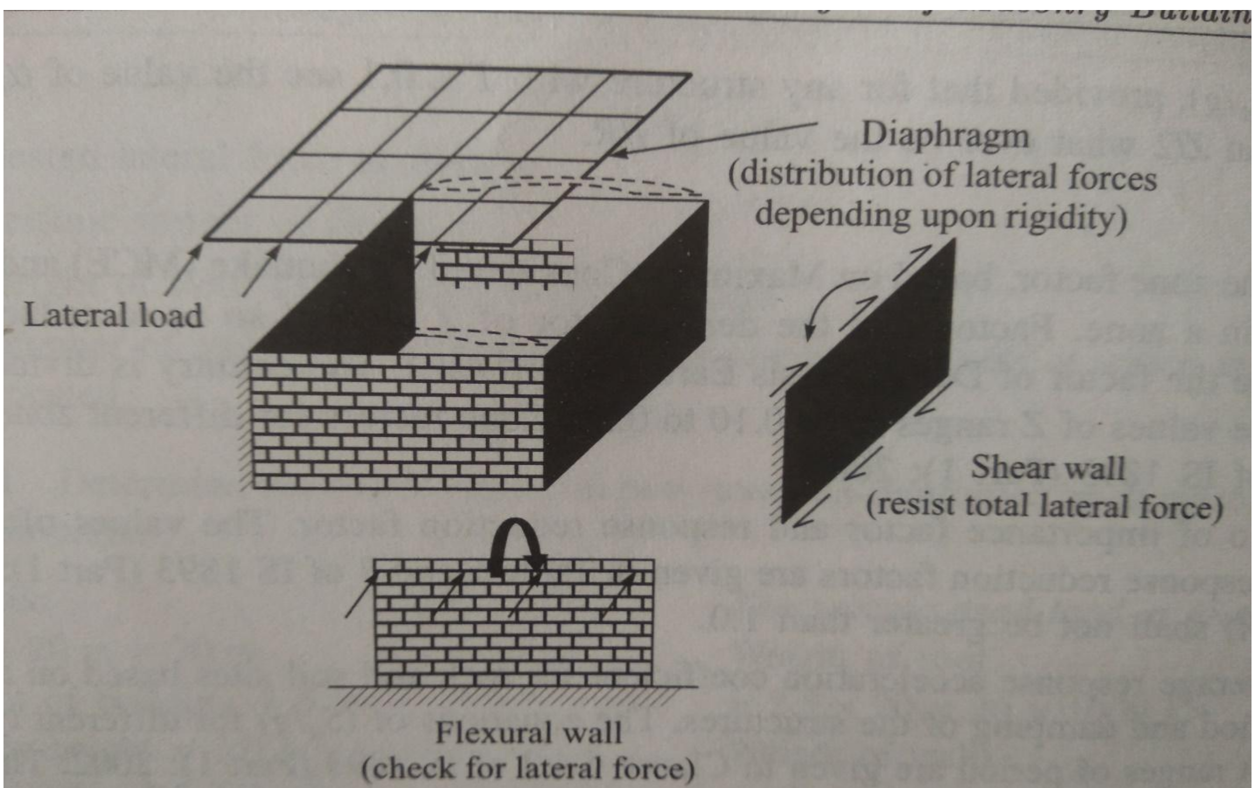


FIGURE 26.1 Force resisting mechanism in masonry buildings.

Following are the major steps for the lateral load analysis of masonry buildings:
Step 1: Determination of lateral load based on IS 1893 (Part 1):2002
Step 2: Distribution of lateral forces on the basis of flexibility of diaphragms
Step 3: Determination of rigidity of shear wall by considering the openings
Step 4: Determination of direct shear forces and torsional shear forces in shear walls
Step 5: Determination of increase in axial load in piers due to overturning
Step 6: check the stability of flexural wall for out-of-plane forces