INTRODUCTION OF EARTHQUAKE

Introduction:

An earthquake is a sudden shaking movement of the surface of the earth. It is known as a quake, tremblor or tremor. Earthquakes can range in size from those that are so weak that they cannot be felt to those violent enough to toss people around and destroy whole cities.

An earthquake is measured on Richter's scale. A seismometer detects the vibrations caused by an earthquake. It plots these vibrations on a seismograph. The strength, or magnitude, of an earthquake, is measured using the Richter scale. Quakes measuring around 7 or 8 on the Richter scale can be devastating.

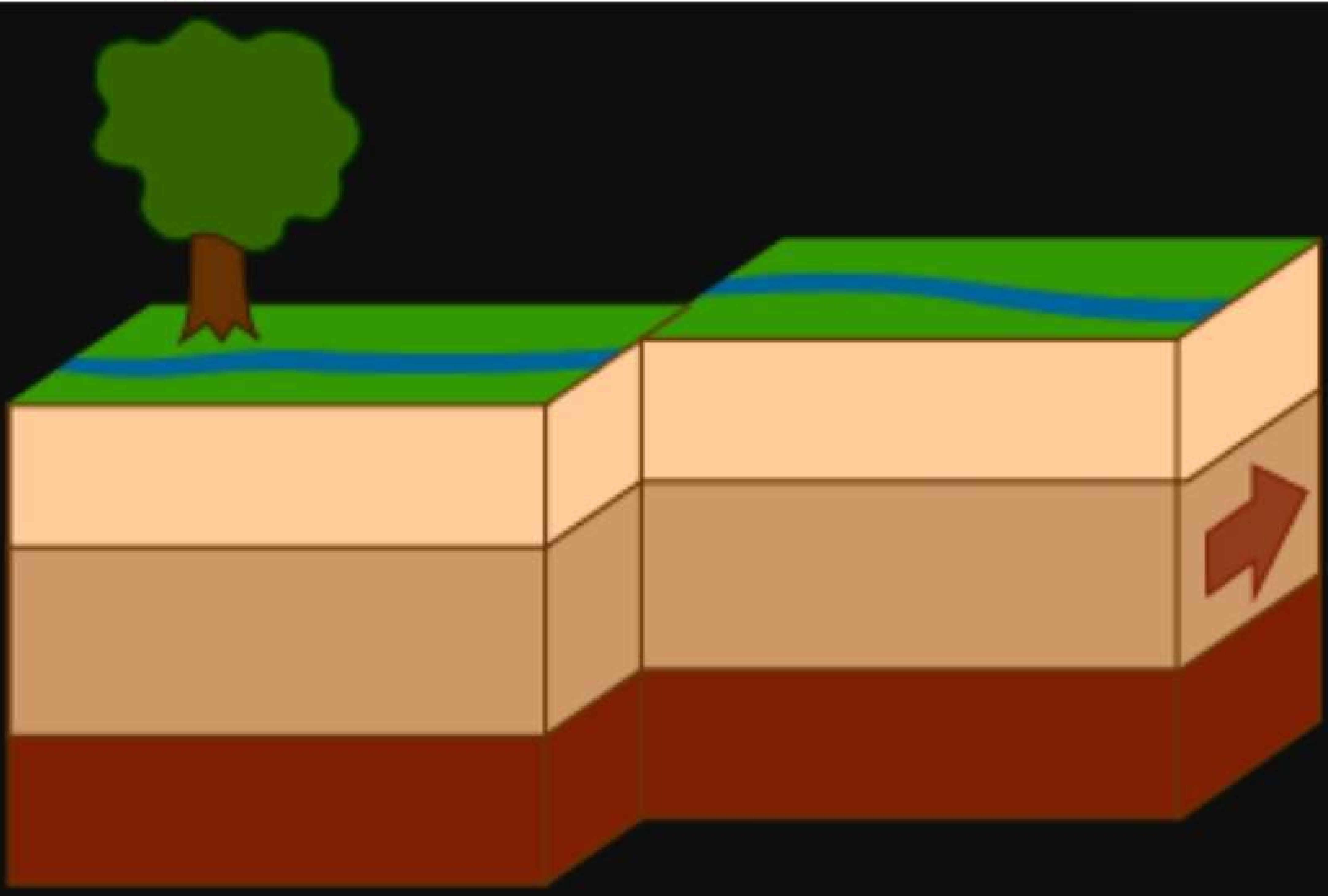
Three types of faults:

A. Strike-slip

B. Normal

C. Reverse

A



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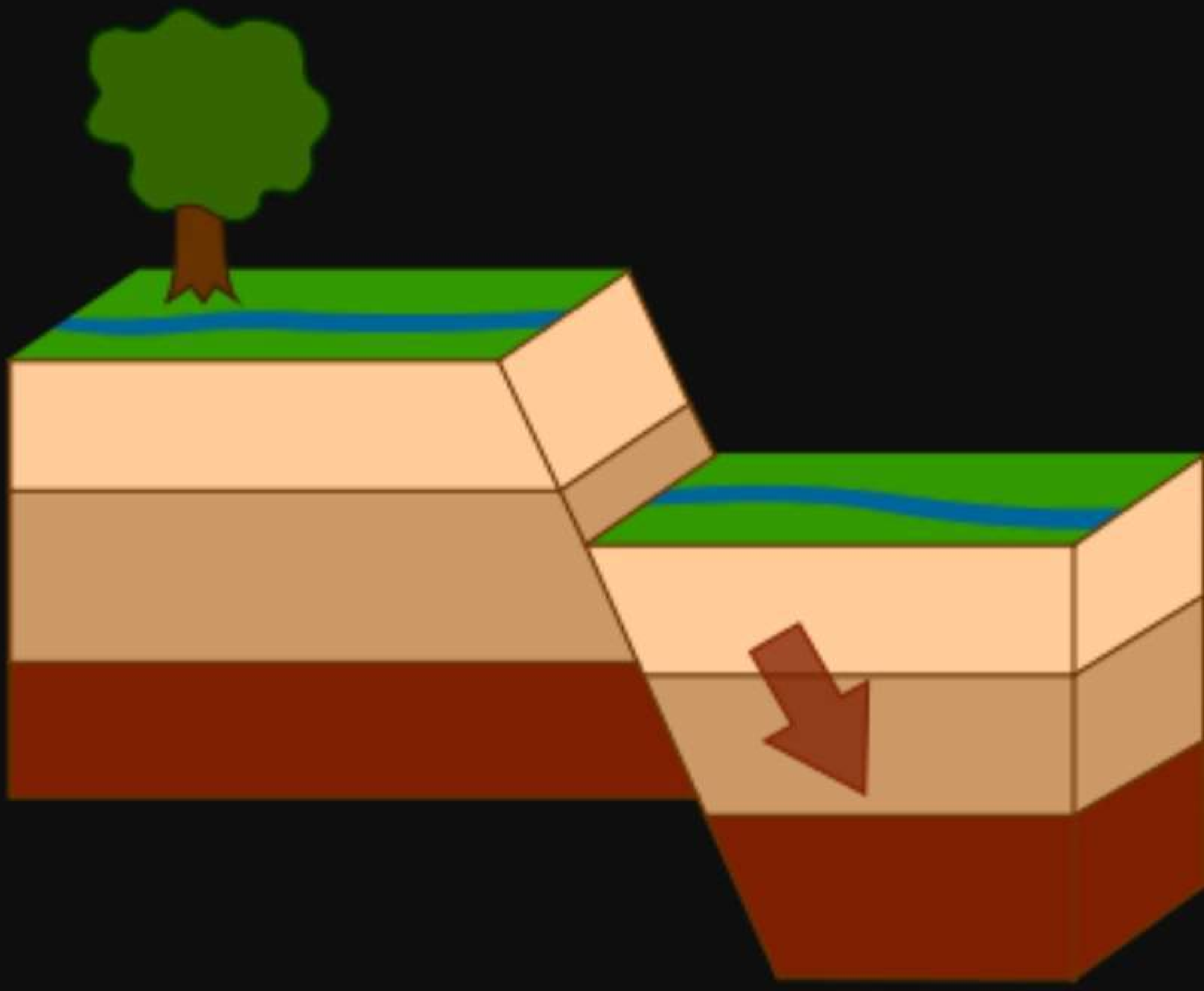

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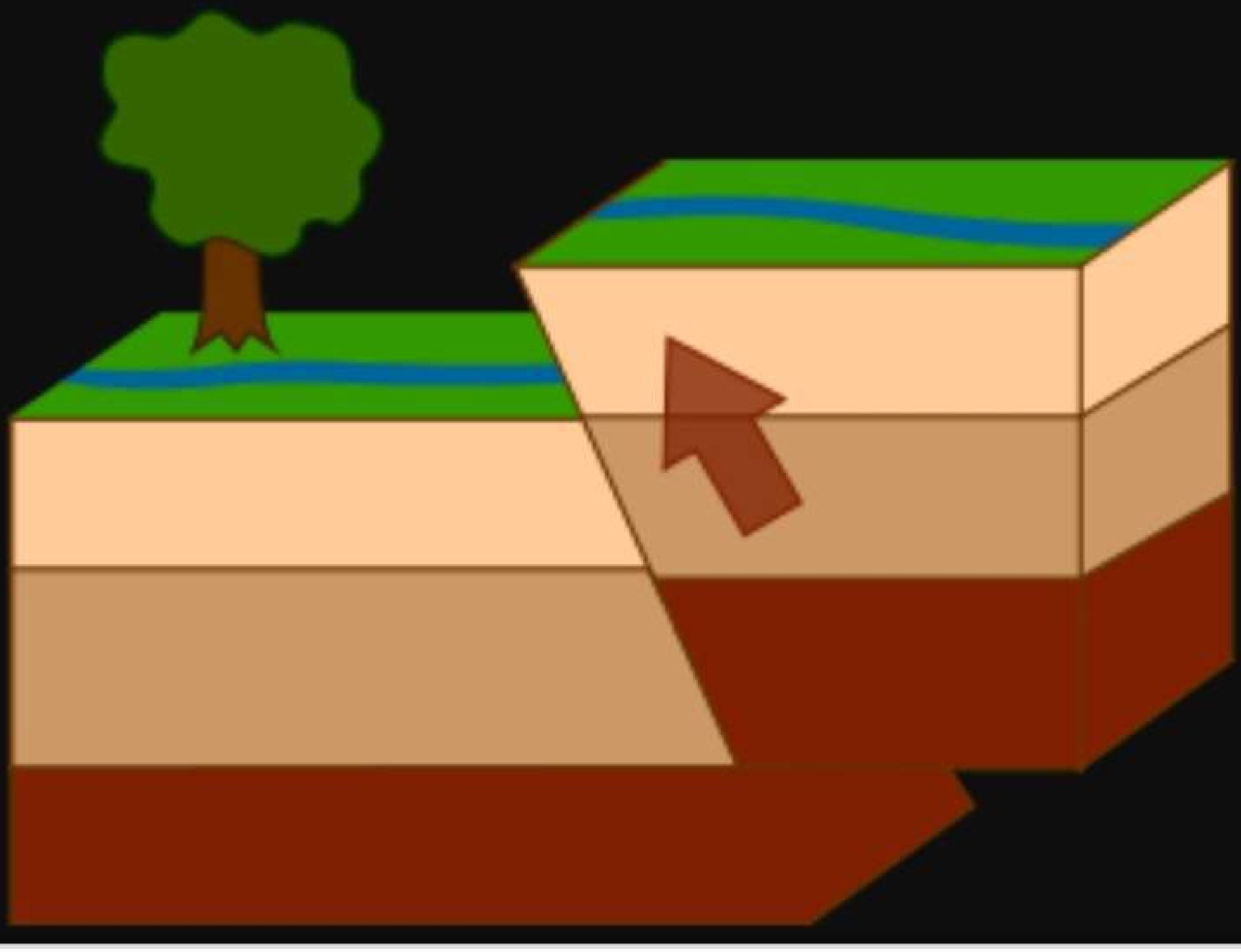

CAUSES

Causes of Earthquake:-

An earthquake occurs because of the movements of tectonic plates beneath the surface of the earth. These movements create waves that propagate through the earth. These waves are known as seismic waves. It causes mild to heavy shaking and vibrations. The intensity of these vibrations can vary, but sometimes they can bring about great destruction. Earthquake in oceans is known as Tsunami, which is equally devastating. The main causes of earthquakes fall into five categories:

1. Volcanic Eruptions

The main cause of the earthquake is volcanic eruptions. Such type of earthquakes occurs in areas, with frequent volcanic activities. When boiling lava tries to break through the surface of the earth, with the increased pressure of gases, certain movements caused in the earth's crust.

2. Tectonic Movements

The surface of the earth consists of some plates, comprising of the upper mantle. These plates are always moving, thus affecting the earth's crust. These movements categorized into three types: constructive, destructive, and conservative. Constructive is when two plates move away from each other, they correspond to mild earthquakes.

3. Geological Faults

A geological fault is known as the displacement of plates of their original plane. The plane can be horizontal or vertical. These planes are not formed suddenly but slowly develop over a long period. The movement of rocks along these planes brings about tectonic earthquakes. These faults occur due to the impact of geological forces.

4. Man-Made

The interference of man with nature can also become a cause of the earthquake. The disturbance of crustal balance due to heavy clubbing of water in dams can cause earthquakes. Nuclear bombing can send specific types of shockwaves throughout the surface of the earth, which can disturb the natural alignment of tectonic plates.

5. Minor Causes

Some minor causes such as landslides, avalanches, the collapse of heavy rocks, etc. can also cause minor shockwaves. The gases beneath the surface of earth contract and expand, giving rise to movements in plates beneath the crust.

Basic Terminology:
Epicentre:
It is the point on the (free) surface of the earth vertically above the place of origin (hypocentre) of an eartquake. This point is expressed by its geographical latitude and longitude.
Hypocentre or Focus:
It is the point within the earth from where seismic waves orignate. Focal depth is the vertical distance between the hypocentre and epicentre.
Magnitude:
It is the quantity to measure the size of an earhquake in terms of its energy and is independent of the place of the observation.
Richter Scale:
Magnitude is measured on the basis of ground motion recorded by an instrument and applying standard correction for the epicentral distance from recording station. It is linearly related to the logarithm of amount of energy released by an earthquake and expressed in Richter Scale.
Intensity:
It is the rating of the effects of an earthquake at a particular place based on the observations of the affected areas, using a descriptive scale like Modified Mercalli Scale.

Classification of earthquakes

1)SlightMagnitude upto 4.9 on the Richter Scale.

2)Moderate Magnitude 5.0 to 6.9

3)Great Magnitude 7.0 to 7.9

4)Very Great Magnitude 8.0 and more

2.6.1. Peak Ground Acceleration

The earthquake time history contains several engineering characteristics of ground motion and maximum amplitude of motion is one of the important parameter among them. The PGA is a measure of maximum amplitude of motion and is defined as the largest absolute value of acceleration time history.

The response of very stiff structures (i.e., with high frequency) is related to PGA. Though PGA is not a very good measure of damage potential of ground motion; due to its close relation with response spectrum and usability in scaling of response spectrum, PGA is extensively used in engineering applications.

Generally, at distances several source dimensions away, vertical PGAs are found to be less than horizontal PGA though at near source distances it could be equal to higher than the corresponding horizontal PGA. For engineering purposes, vertical PGA is assumed to be two thirds of the horizontal PGA.

2.6.2. Peak Velocity

Peak velocity is the largest absolute value of velocity time history. It is more sensitive to the intermediate frequency components of motion and characterizes the response to structures that are sensitive to intermediate range of ground motions, e.g. tall buildings, bridges, etc.

2.6.3. Peak Displacement

Peak displacements reflect the amplitude of lower frequency components in ground motion. Accurate estimation of these parameters is difficult as the errors in signal processing and numerical integration greatly affect the estimation of amplitude of displacement time history.

2.7. Frequency Content of Motion

Earthquake ground motion is an amalgamation of harmonic motion with a range of frequency components and amplitudes. Several approaches have been proposed in the literature to quantitatively estimate these characteristics. Some of these are discussed below

2.7.1. Response Spectra

A plot showing the maximum response induced by ground motion in single degree of freedom oscillators of different fundamental time periods having same damping is known as response spectrum. The maximum response could be spectral acceleration, spectral velocity or spectral displacement.

The spectral velocity and spectral acceleration are related by:

$$SA = \omega_0 SV \tag{2.5}$$

where *SA* is the spectral acceleration, *SV* spectral velocity and ω_0 the natural circular frequency. Similarly, it can be shown that

$$SV = \omega_0 SD \tag{2.6}$$

where *SD* is the spectral displacement.

As response spectra represents the frequency content of the motion after propagation through earth's crust, large amount of variability is expected across the response spectra. Figure 2.3 depicts the time histories (all normalized to 1g PGA) and corresponding response spectra, bringing out the variations observed in the response spectra with respect to time histories.

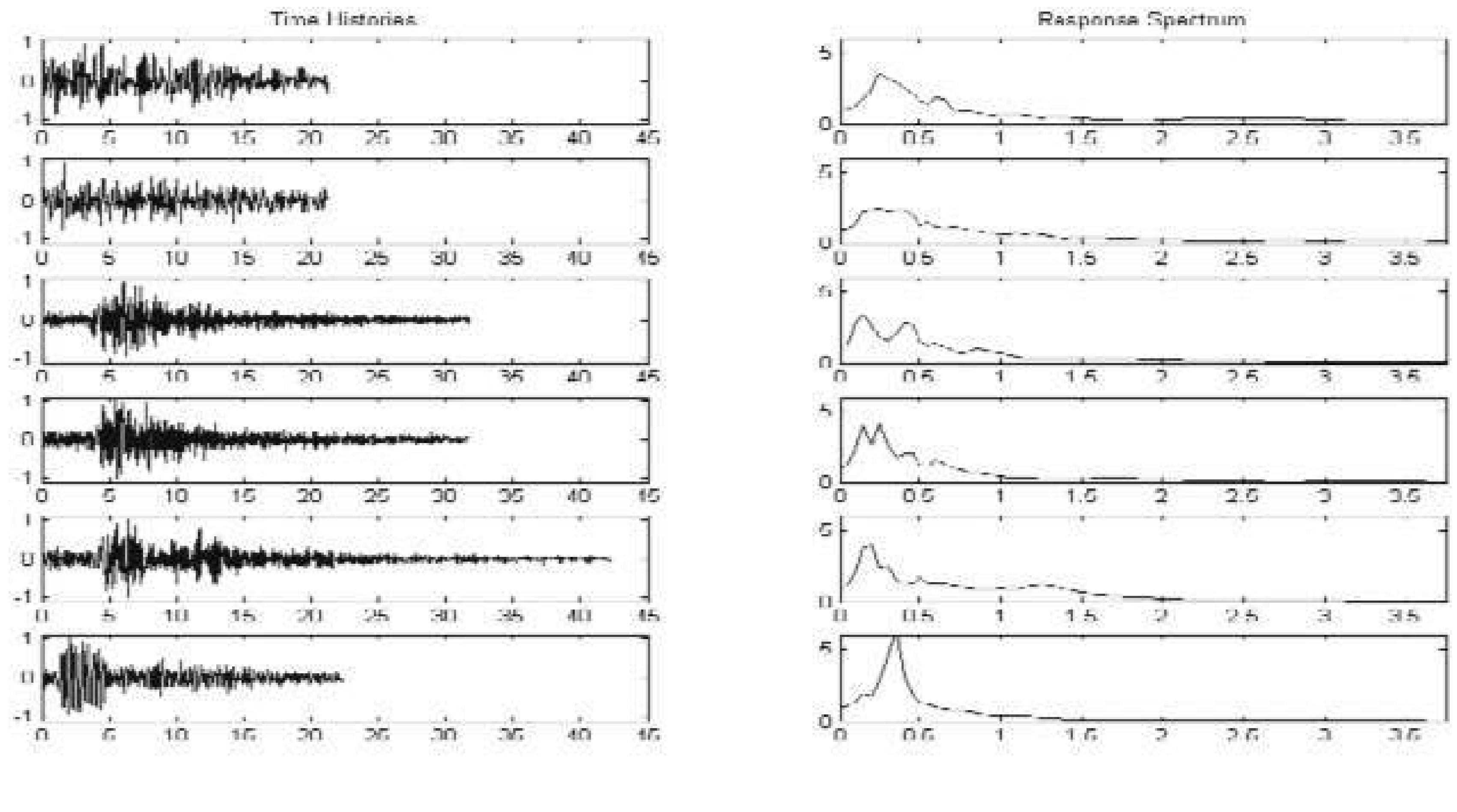

Figure 2.3 Plot of time histories and corresponding response spectra (Abscissa is in seconds and ordinate is in 'g').

The shape of response spectrum and location of peaks are controlled by characteristics of site condition (soil/rock) at the location of measurement, magnitude of earthquake, distance, etc. It is seen that in general, the motion on rock contains more shore period content of motion compared to that in soil. Similarly, the response spectra from a low magnitude event recorded at closer distances will be rich in small period or high frequency components compared to a large magnitude event recorded at farther distances. The design response spectral shape, as suggested by Indian standard, BIS 1893, for three types of site conditions are given in Figure 2.4.

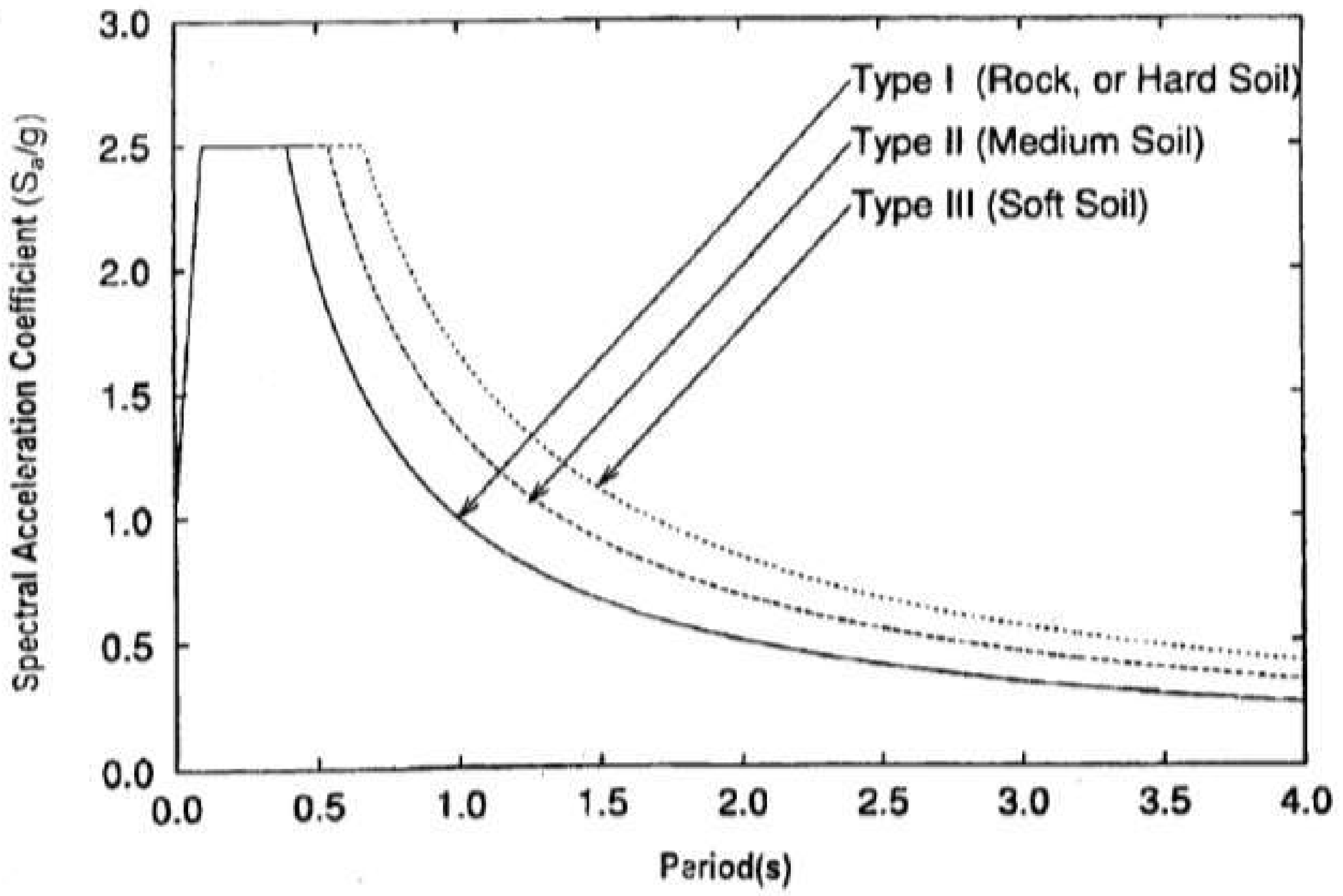

Figure 2.4 Design response spectral shape suggested by BIS (IS 1893-2002).

The response spectra can be plotted with any of the three parameters (acceleration, velocity and displacement) as mentioned above as ordinate and period/frequency as abscissa. Since these parameters are interconnected through the expressions (2.5 and 2.6) as given, all three parameters can also be represented in a single graph known as tripartite plot or Displacement velocity acceleration (DVA) spectrum. Figure 2.5 depicts a typical tripartite plot of a response spectrum for two levels of damping. It can be also noted from the figure that the effect of the damping on the response spectrum is greatest in the velocity sensitive region, and is least in the acceleration sensitive and displacement sensitive regions. This is possible due to the fact that in logarithmic domain, these relationships boils down to a group of straight lines, which are at 45 degree angles with the line corresponding to the spectral velocity.

It can be seen that there are three main regions in a DVA spectrum. The portion of the response spectrum to the left of point period = 0.1sec is almost constant and is most directly related to the maximum ground acceleration. Similarly, the portion of response spectrum to the right of 10 sec period is most directly related to the maximum ground displacement, which is also constant in that region. The intermediate portion is related to the maximum velocity of the ground motion.

1:21

Date	Location	Mag.	I	Deaths	Injuries	Total damage / notes	
2017-01-03	India, Bangladesh	5.7 M_w	V	3	8		
2016-01-04	India, Myanmar, Bangladesh	6.7 M_w	VII	11	200		
2015-10-26	Afghanistan, India, Pakistan	7.7 M_w	VII	399	2,536		
2015-05-12	Nepal, India	7.3 M_w	VIII	218	3,500+		
2015							

2015-04-25	Nepal, India	7.8 M_w	IX	8,964	21,952	$10 billion	
2013-05-01	Kashmir	5.7 M_w		3	90	$19.5 million	NGDC
2011-09-18	Gangtok, Sikkim	6.9 M_w	VII	>111			
2009-08-10	Andaman Islands	7.5 M_w	VIII			Tsunami warning issued	
2008-02-06	West Bengal	4.3 M_b		1	50	Buildings damaged	NGDC
2007-						Buildings	[2]

2006-11-29	Alwar district, Rajasthan	4.0 M_w		1	2	Minor damage to property	[3]
2006-03-07	Gujarat	5.5 M_w	VI		7	Buildings damaged	[4]
2006-02-14	Sikkim	5.3 M_w	V	2	2	Landslide	[5]
2005-12-14	Uttarakhand	5.1 M_w	VI	1	3	Building destroyed	[6]
2005-10-08	Kashmir	7.6 M_w	VIII	86,000–87,351	69,000–75,266	2.8 million displaced	
2002-	Andaman Islands	6.5 M		2		Destructive	NGDC

2002-09-13	Andaman Islands	6.5 M_w		2		Destructive tsunami	NGDC
2001-01-26	Gujarat	7.7 M_w	X	13,805–20,023	~166,800		
1999-03-29	Chamoli district-Uttarakhand	6.8 M_w	VIII	~103			
1997-11-21	Bangladesh, India	6.1 M_w		23	200		
1997-05-22	Jabalpur, Madhya Pradesh	5.8 M_w	VIII	38–56	1,000–1,500	$37–143 million	
1993-	Latur,						

2002-09-13	Andaman Islands	6.5 M_w		2		Destructive tsunami	NGDC
2001-01-26	Gujarat	7.7 M_w	X	13,805–20,023	~166,800		
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1997-05-22	Jabalpur, Madhya Pradesh	5.8 M_w	VIII	38–56	1,000–1,500	$37–143 million	
1993-	Latur,	[illegible]	[illegible]	[illegible]	[illegible]		

1993-09-30	Latur, Maharashtra	6.2 M_w	VIII	9,748	30,000		
1991-10-20	Uttarkashi, Uttarakhand	6.8 M_w	IX	768–2,000	1,383–1,800		
1988-08-21	Udayapur, Nepal	6.9 M_w	VIII	709–1,450			
1988-08-06	Myannmar, India	7.3 M_w	VII	3	12		[7]
1988-02-06	Bangladesh, India	5.9 M_w		2	100		[8]
1986-	India, Pakistan	5.3 M_s		6	30	Severe damage	NGDC

1986-04-26	India, Pakistan	5.3 M_s		6	30	Severe damage	NGDC
1984-12-30	Cachar district	5.6 M_b		20	100	Severe damage	NGDC
1982-01-20	Little Nicobar	6.3 M_s			Some	Moderate damage	NGDC
1980-08-23	Kashmir	4.8 M_s		Few		Limited damage / doublet	NGDC
1980-08-23	Kashmir	4.9 M_s		15	40	Moderate damage / doublet	NGDC

1980-07-29	Nepal, Pithoragarh district	6.5 M_s		200	Many	$245 million	NGDC
1975-01-19	Himachal Pradesh	6.8 M_s	IX	47			
1970-03-23	Bharuch district	5.4 M_b		26	200	Moderate damage	NGDC
1967-12-11	Maharashtra	6.6 M_w	VIII	177–180	2,272	$400,000	
1966-08-15	North India	5.6		15		Limited damage	NGDC
1966-	Nepal, India	5.3 M	VIII	80	100	$1 million	NGDC

1966-06-27	Nepal, India	5.3 M_s	VIII	80	100	$1 million	NGDC
1963-09-02	Kashmir	5.3		80		Moderate damage	NGDC
1960-08-27	North India					Moderate damage	NGDC
1956-07-21	Gujarat	6.1 M_s	IX	115	254		
1954-03-21	India, Myannmar	7.4 M_s				Moderate damage	NGDC
1950-	Assam, Tibet	8.6 M	XI	1,500–			

1950-08-15	Assam, Tibet	8.6 M_w	XI	1,500–3,300				
1947-07-29	India, China	7.3 M_w						
1941-06-26	Andaman Islands	7.7–8.1 M_w		8,000			Destructive tsunami	
1935-05-31	Quetta, Baluchistan	7.7 M_w	X	30,000–60,000				
1934-01-15	Nepal	8.0 M_w	XI	6,000–10,700				
1932-	Assam, Myanmar	7.0 M_w					Moderate	NGDC

1932-08-14	Assam, Myannmar	7.0 M_s				Moderate damage	NGDC
1905-04-04	Kangra	7.8 M_s	IX	>20,000			
1897-06-12	Shillong, India	8.0 M_w	X	1,542			
1885-06-06	Kashmir					Severe damage	NGDC
1885-05-30	Srinagar			3,000		Extreme damage	NGDC
1881-	Andaman Islands	7.9 M	VII			Significant in	

1881-12-31	Andaman Islands	7.9 M_w	VII			Significant in seismology	
1869-01-10	Assam, Cachar	7.4 M_w	VII	2		Severe damage	
1845-06-19	Rann of Kutch	6.3 M_s	VIII	Few		Limited damage / tsunami	NGDC
1843-04-01	Deccan Plateau					Moderate damage	NGDC
1833-08-26	Bihar, Kathmandu	8.0 M_s				Severe damage	NGDC
1828-	Kashmir			1,000		Severe damage	NGDC

04-01						damage	
1833-08-26	Bihar, Kathmandu	8.0 M_s				Severe damage	NGDC
1828-06-06	Kashmir			1,000		Severe damage	NGDC
1819-06-16	Gujarat	7.7–8.2 M_w	XI	>1,543		Formed the Allah Bund	
1618-05-26	Bombay		IX	2,000		Severe damage	NGDC
1505-06-06	Saldang, Karnali zone	8.2–8.8		6,000			

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Points to remember

- Local climate, materials available for construction and traditions of the locality influence the construction of buildings

- The box action helps in limiting the deformations imposed on masonry during an earthquake and, hence, prevents extensive damages and collapse

- Random rubble (RR) masonry consists of rough cut or natural stones set in mud mortar

- Typical failures in RR masonry buildings are: Overturning of walls due to out-of-plane inertia forces, separation of the two leaves of stone walls, and collapse of roof due to very heavy self weight

- X-cracks are common in URM buildings
- During an earthquake lateral inertial forces are induced due to the shaking of structures
- If the inertial forces are in the plane of the wall a masonry wall can also undergo in-plane shear stresses
- 'Naliyawali Deewal' was made out of country tiles with thin sticks on either face of the wall, held together by coir ropes
- A two-storey house in concrete block masonry with lintel band in Bheematala, UP survived in spite of the quality of construction being poor
- The same building constructed using two different materials at different times could survive the earthquake without any damage because of RC lintel band
- The Chinese seismic code recommends provision of ties in the construction of masonry to resist the tension induced by earthquake
- Reinforced Concrete (RC) buildings, particularly mid-rise, multi-family condominium structures, experienced significant damage in the 2009 earthquake that struck L'Aquila, Italy
- Common failure features observed in Taiwan earthquake were that columns were provided with stirrups with large spacing, splices were provided without sufficient development lengths, stirrups were having only 90° bend, etc.

1. THROUGH DIAGONAL CRACKS OR THROUGH X-SHAPE CRACKS ON THE WALL

- Through diagonal cracks or through X-shape cracks are the common earthquake induced damages on the walls of the masonry buildings, as shown in Fig.1.
- This kind of earthquake damage belongs to shear failure, which is caused by the principal tensile stress exceeding shear strength of masonry.
- The through X-shape cracks are very popular on the longitudinal walls, especially between the door or window openings of nearly every floor.
- The diagonal cracks usually appear mostly on the bearing transverse walls. These cracks may lead to the obvious decrease of structural capacity and even collapse o the buildings.

Fig. 1. Through diagonal or X-shape cracks on the wall

2. HORIZONTAL CRACK ON THE WALL

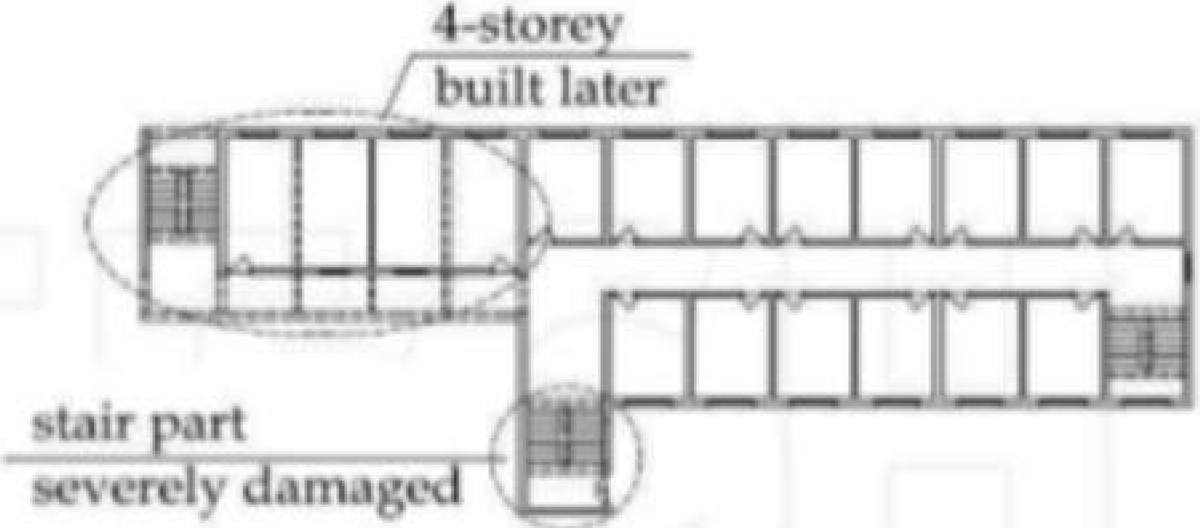
- Another main seismic damage is the horizontal crack on the wall.
- Horizontal cracks usually appear at the wall near the elevation of floor or roof, which enlarges the damage and results in collapse of pre-cast hollow slab.
- Meanwhile, horizontal cracks also appear on the end of some bearing brick columns, which lead to the decrease and even loss of the structural capacity.
- This kind of cracks means horizontal shear failure of walls.
- It is deduced that the large vertical ground motion may lead to this kind of earthquake damage.
- Typical phenomenon of horizontal cracks on the wall is shown in Fig. 2.

Fig. 2. Horizontal cracks on the wall or the bearing brick column

3. DAMAGE OF THE STAIR PART

- Comparing with the other parts, the damage of stair part is relative severe because of the relative large stiffness of the slope structural members.
- Fig. 3 shows the severe damage of the stair part in the building with irregular plan.
- From the layout of this building, it is seen that the stair part is the convex part of the T-shaped plan.

Fig. 3. Partial collapse of stair part



Layout of the building

4. DAMAGES OF NON-STRUCTURAL COMPONENTS

- Severe damages on non-structural components, such as horizontal crack, diagonal crack, even partial collapse can be easily found due to no reliable connections with the main structures.
- Fig.4 shows the partial collapse of the parapet, which even leads to the damage of the roof slab.

- Fig. 5 shows the falling of the corridor fence, severe damage of the protruding member in the roof.
- These are typical phenomena of the seismic induced damage of the non-structural components.

Fig. 4. Failure of the parapet wall

5. DAMAGE CAUSED OR AGGRAVATED BY STRUCTURAL IRREGULARITY

- Two examples are given here to show the harmful influence of structural irregularity on the building damage.
- Fig. 6 shows the severe damage of the L-shaped building with unequal height.
- Fig.7 shows the severe damage in the part of staggered elevation.

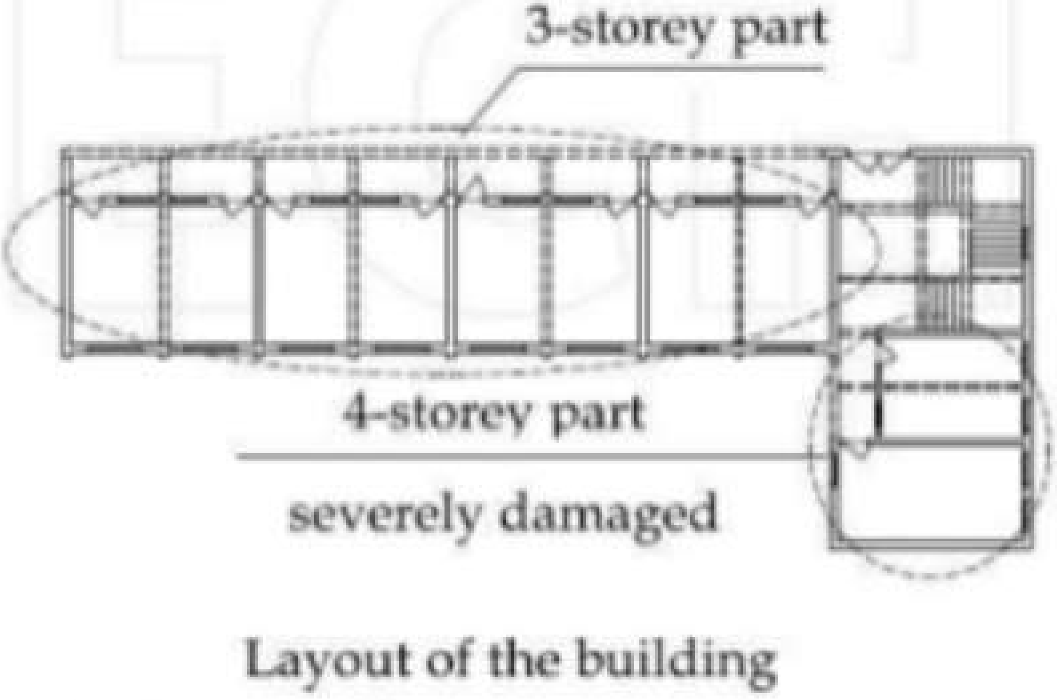
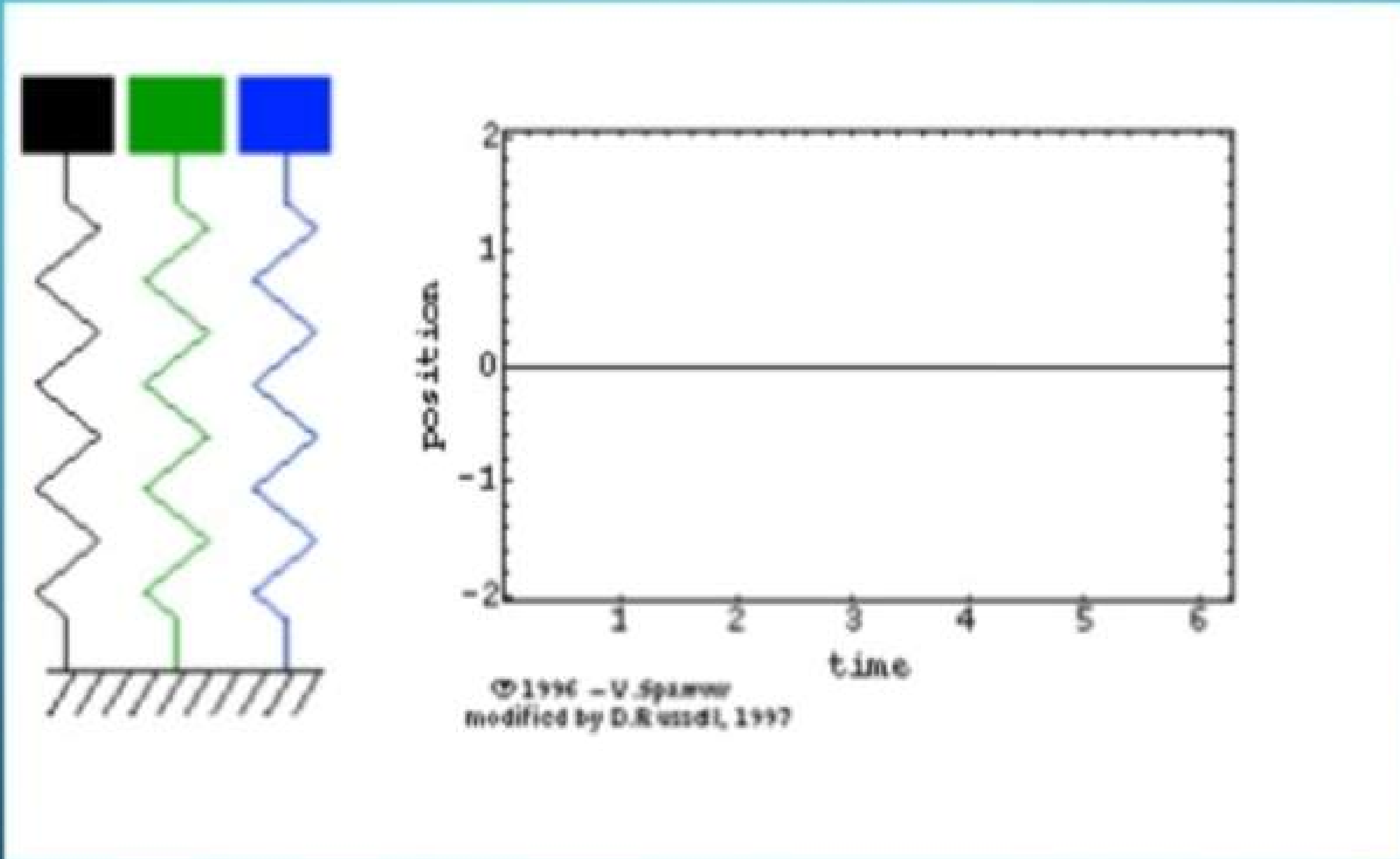


Fig. 6. Severe damage of masonry buildings with plan irregularity

WHAT İS VİBRATİON ?

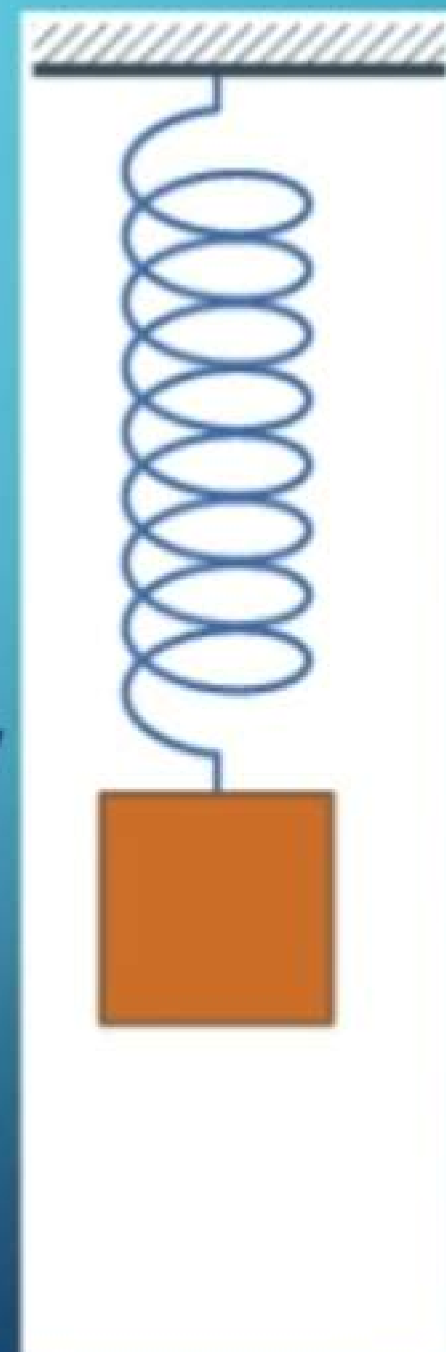
- Vibration is defined as mechanical oscillations of a system about an equilbrium position.



Vibratory System consists of:

1) spring or elasticity
2) mass or inertia
3) damper

- Involves transfer of potential energy kinetic energy and vice versa.



TYPES OF VIBRATIONS

LONGITUDINAL VIBRATIONS

A continuing periodic change in th
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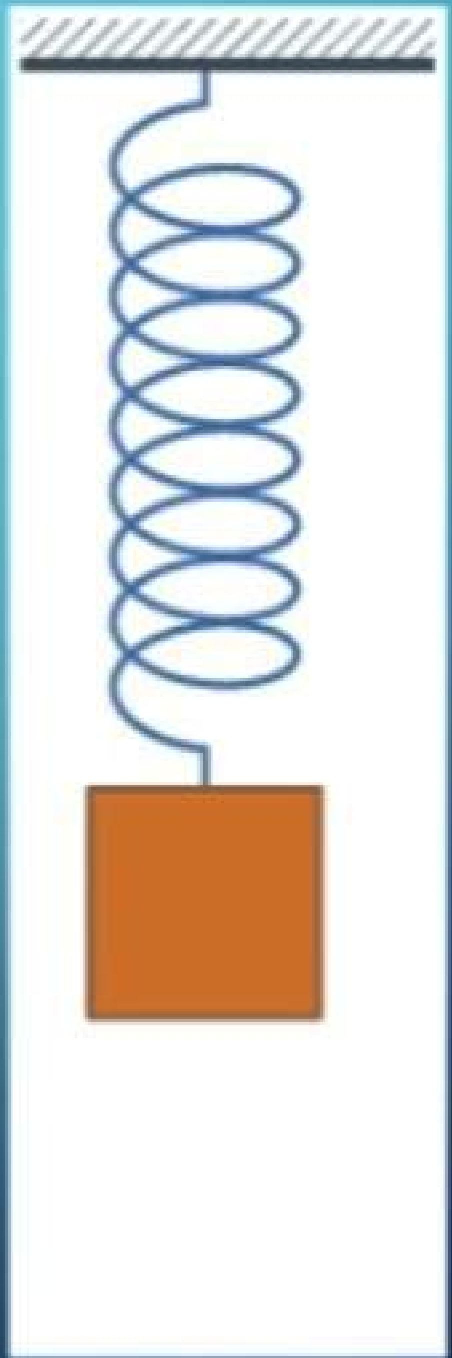
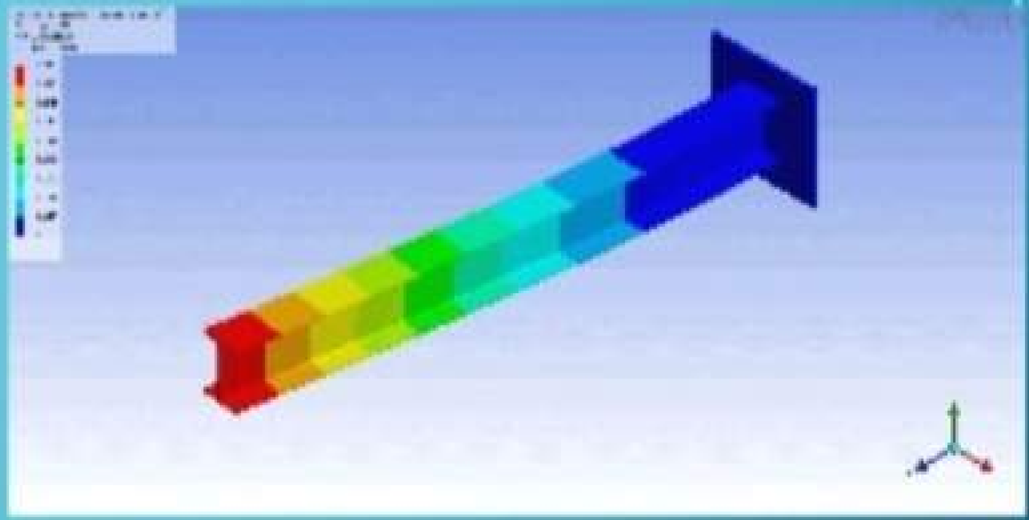
displacement of elements of an object in the direction of the long axis of the rod



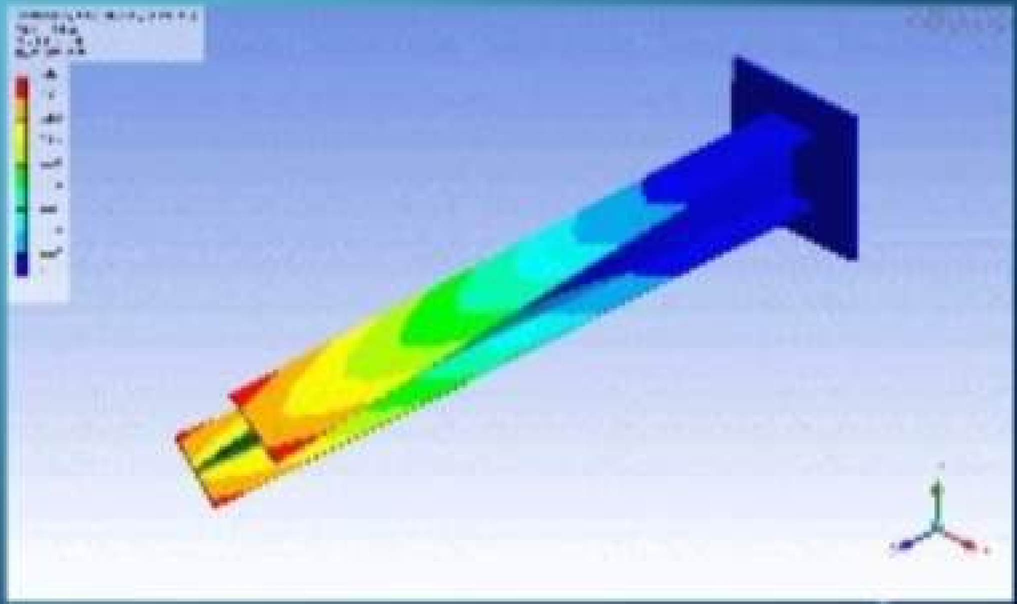
TRANSVERSE VIBRATION

a vibration in which the element moves to and fro in a direction perpendicular to the direction of the advance of the wave.



TORSIONAL VIBRATIONS

Angular vibration of an object (shaft) along its axis of rotation. Torsional vibration is often a concern in power transmission systems using rotating shafts or couplings where it can cause failures if not controlled



- **Undamped Vibration:**
  When *no* energy is lost or dissipated in friction or other resistance during oscillations
- **Damped Vibration:**
  When *any* energy is lost or dissipated in friction or other resistance during oscillations
- **Linear Vibration:**
  When *all* basic components of a vibratory system, i.e. the spring, the mass and the damper behave linearly
- **Nonlinear Vibration:**
  If *any* of the components behave nonlinearly

MAIN CAUSE OF VIBRATION

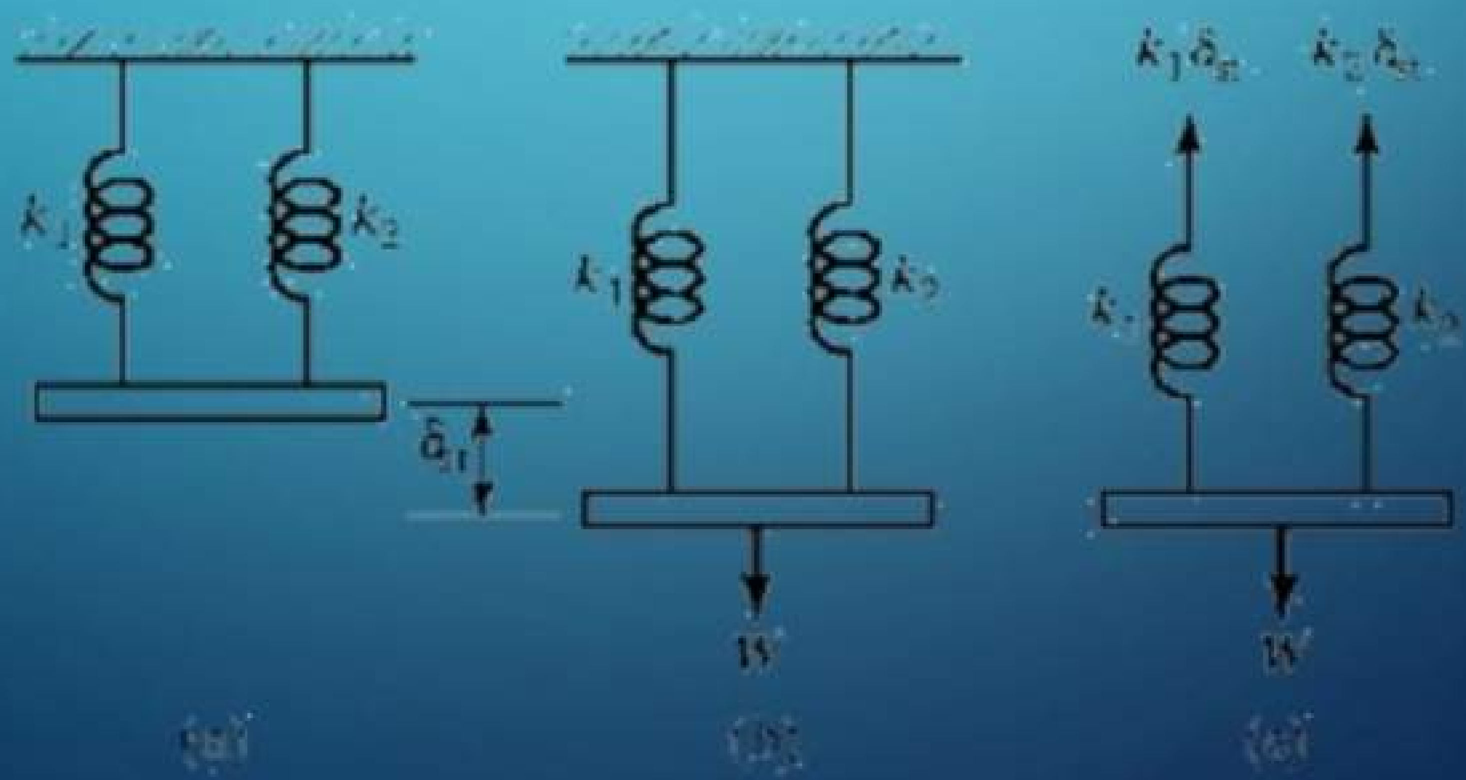
1. Unbalanced force in the different parts of the machine.
2. Lack of lubricants between two mating surface.
3. External load or force which make system vibrant.
4. Lack of balancing of force in machine part.
5. Earthquakes
6. Winds which may cause vibration in transmission and telephone lines.

SPRING ELEMENTS

- Combination of Springs:

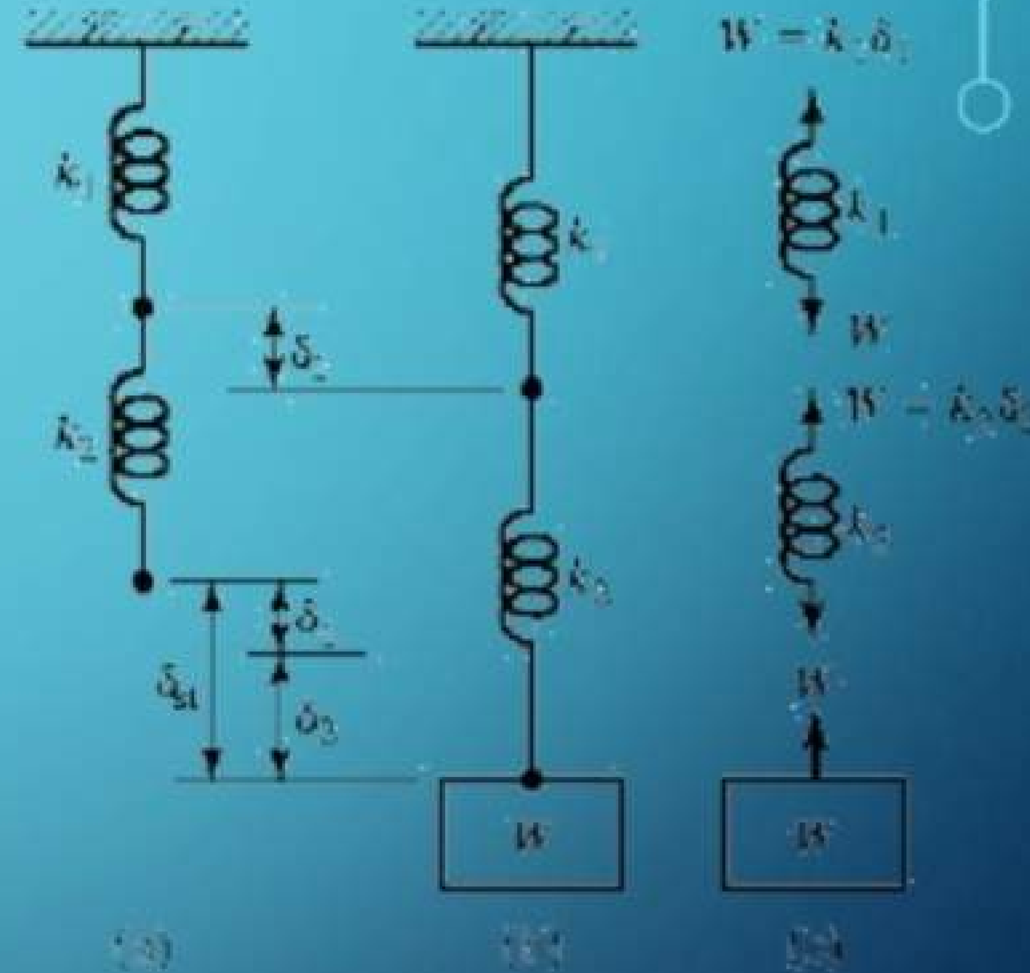
i) Springs in parallel – if we have n spring constants k_1, k_2, …, k_n in parallel, then the equivalent spring constant k_{eq} is:

$$k_{eq} = k_1 + k_2 + \ldots + k_n \qquad (1.11)$$



Combination of Springs:

a) Springs in series – if we have n spring constants $k_1, k_2, \ldots, k_n$ in series, then the equivalent spring constant k_{eq} is:



$$\frac{1}{k_{eq}} = \frac{1}{k_1} + \frac{1}{k_2} + \ldots + \frac{1}{k_n} \qquad (1.17)$$

- Damping is the conversion of mechanical energy of a structure into thermal energy.
- The amount of energy dissipated is a measure of the structure's damping level.
- Damping is very important with earthquakes since it dissipates the destructive energy of an earthquake which will help reduce the damage to the building.

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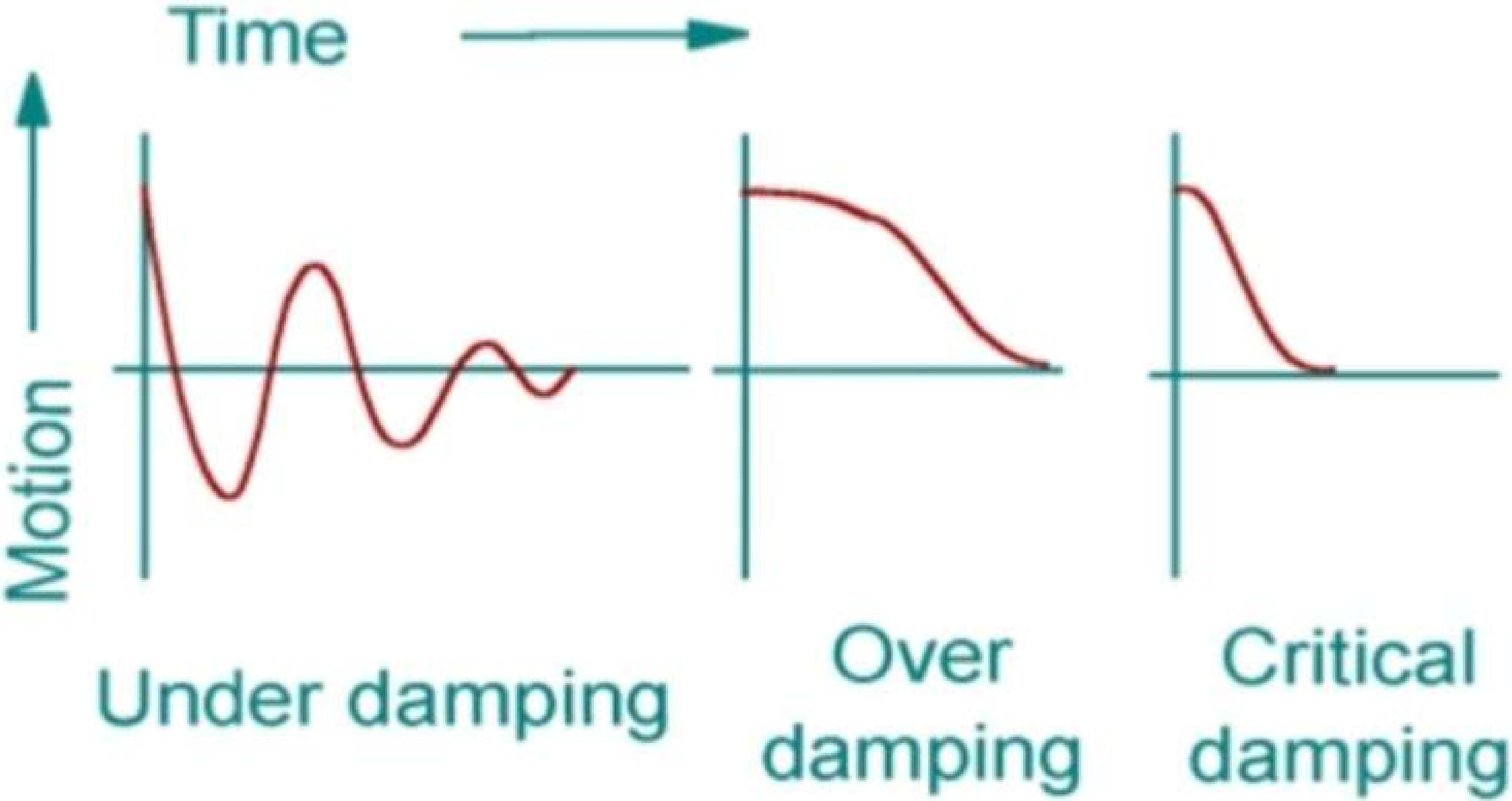
TYPES OF DAMPING

- **Viscous Damping:**
Damping force is proportional to the velocity of the vibrating body in a fluid medium such as air, water, gas, and oil.

- **Coulomb or Dry Friction Damping:**
Damping force is constant in magnitude but opposite in direction to that of the motion of the vibrating body between dry surfaces.

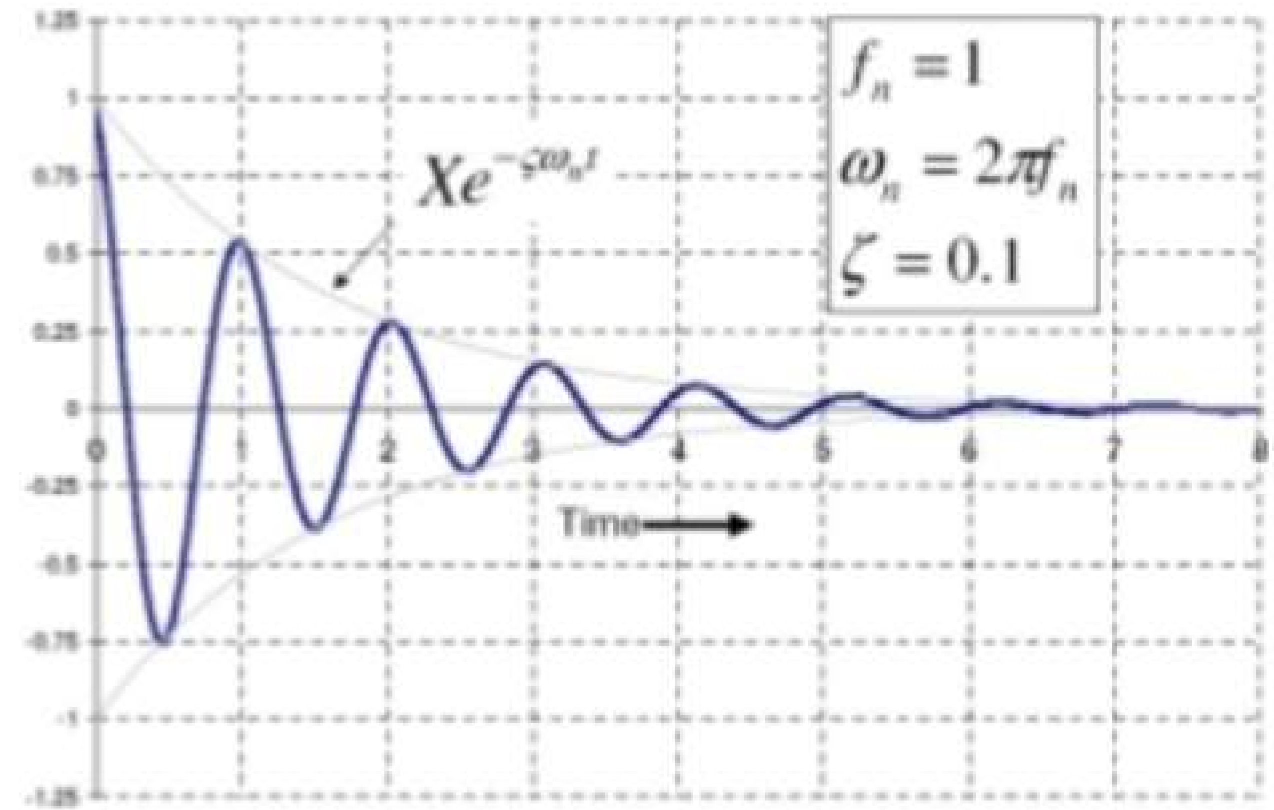
- **Material or Solid or Hysteretic Damping:**
Energy is absorbed or dissipated by material during deformation due to friction between internal planes.



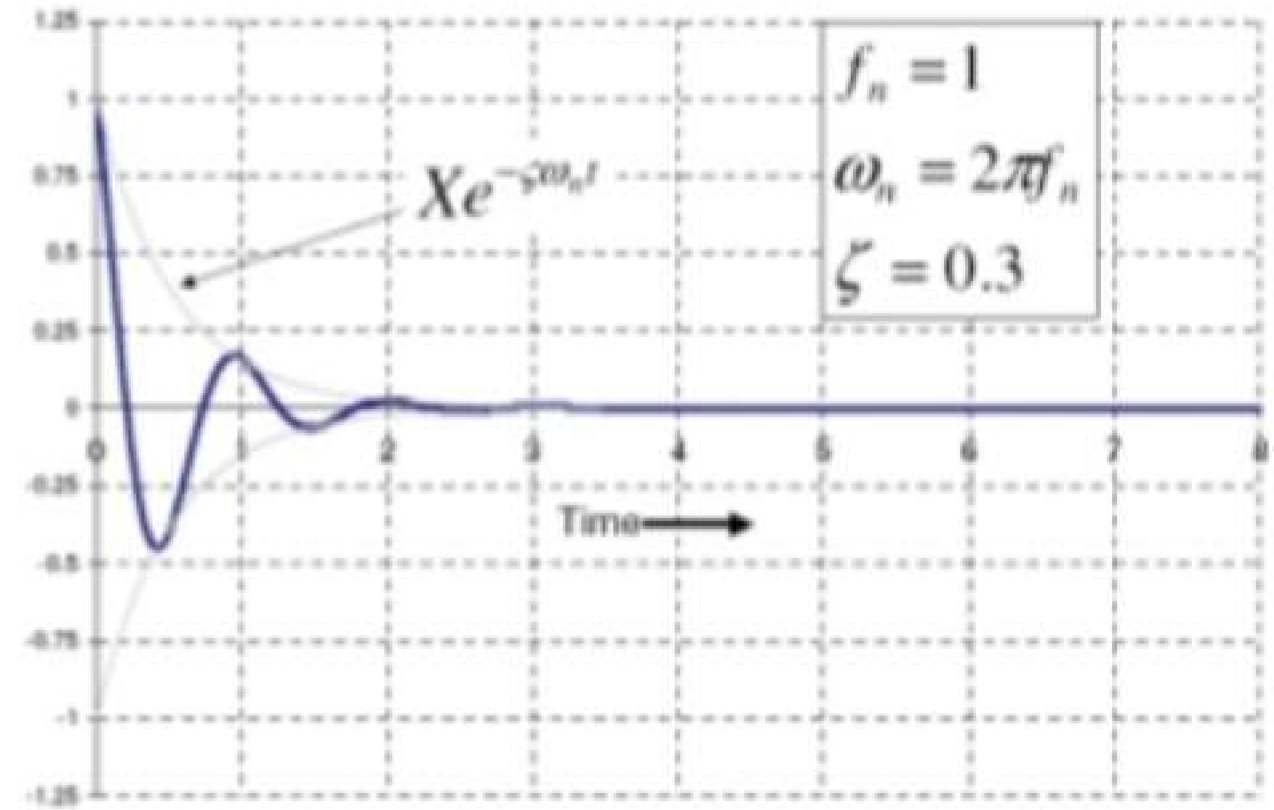
Time
Motion
Under damping
Over damping
Critical damping

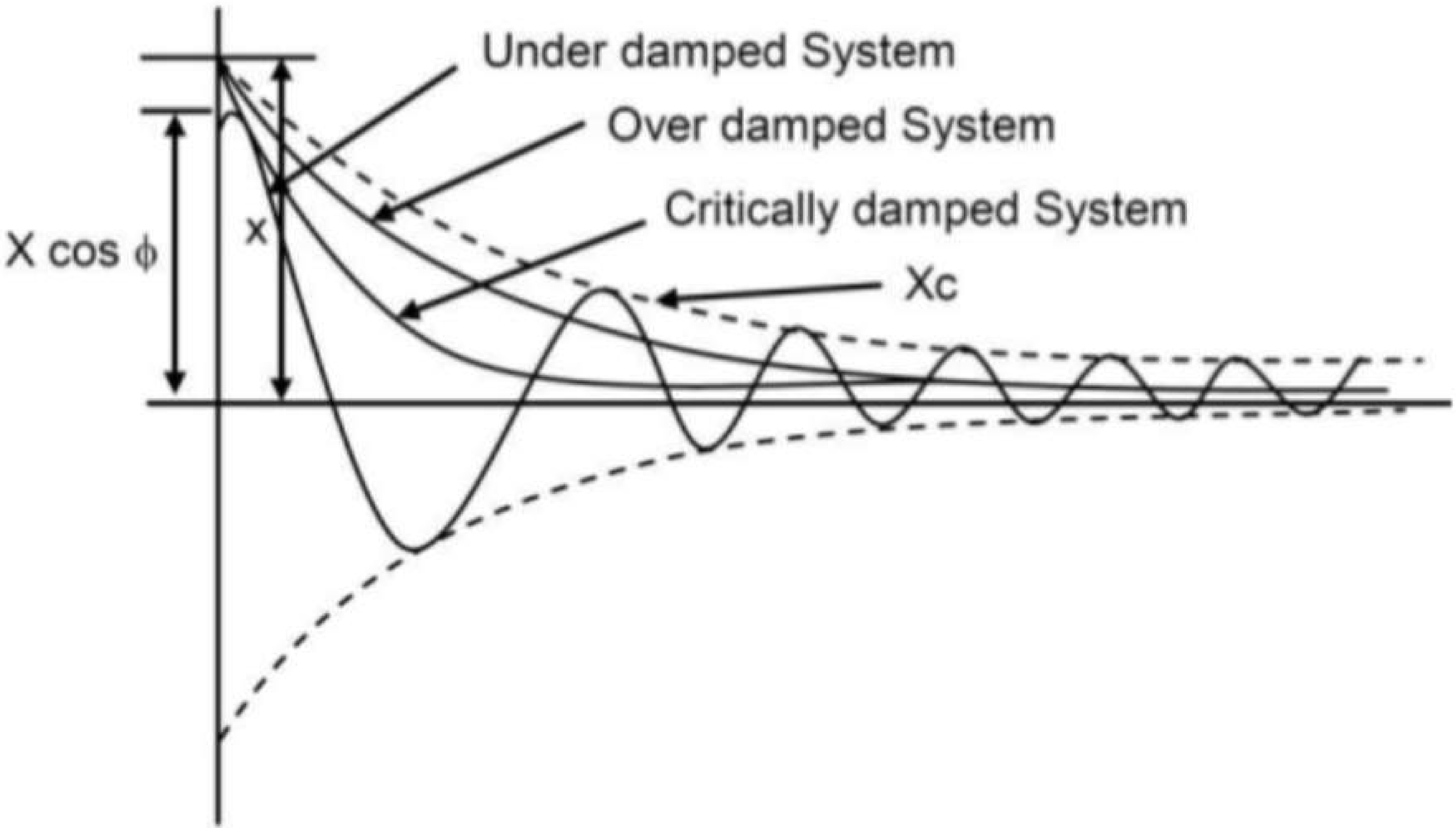
Damped and un-damped natural frequencies

- The exponential term defines how quickly the system "damps" down. The larger the damping ratio, the quicker it damps to zero.



- The cosine function is the oscillating portion of the solution, but the frequency of the oscillations is different from the un-damped case.




Under damped System
Over damped System
Critically damped System
Xc
X cos ϕ
x

DEGREE OF FREEDOM (DOF)

- Mathematical modeling of a physical system requires the selection of a set of variables that describes the behavior of the system.

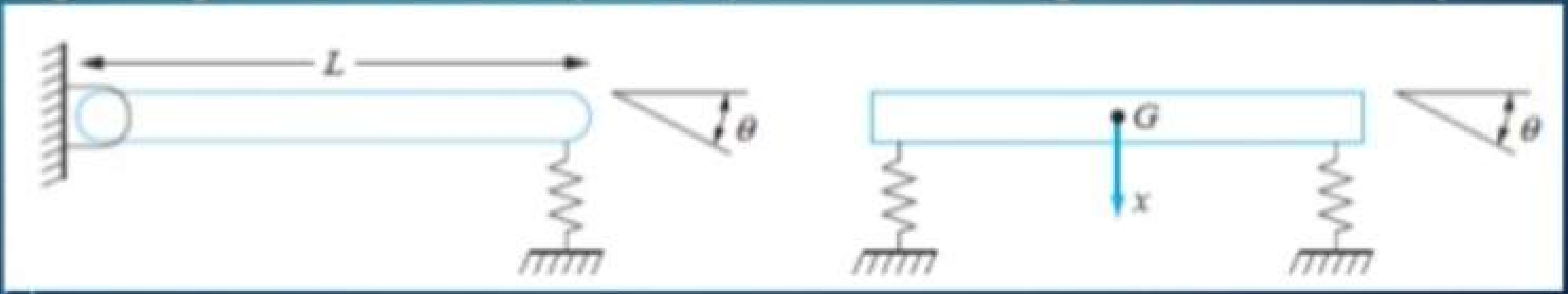
- The number of *degrees of freedom* for a system is the number of kinematically independent variables necessary to completely describe the motion of every particle in the system

DOF=1

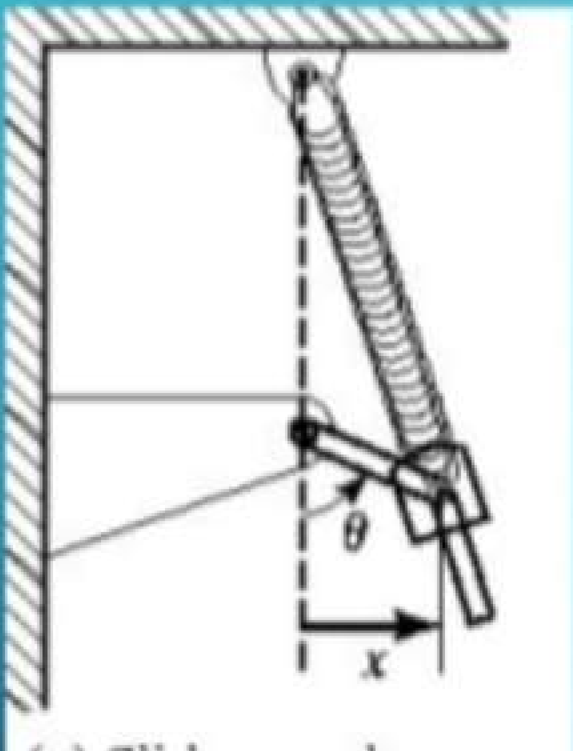
Single degree of freedom (SDOF)

DOF=2

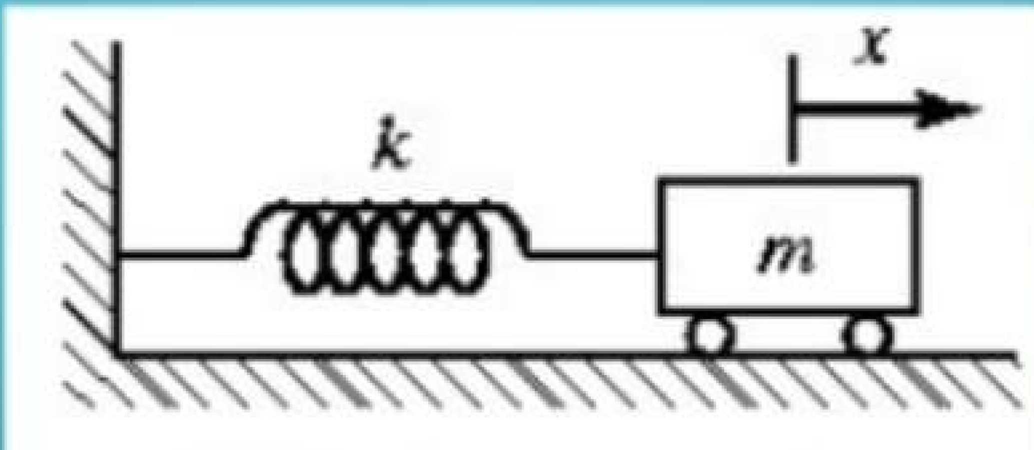
Multi degree of freedom (MDOF)



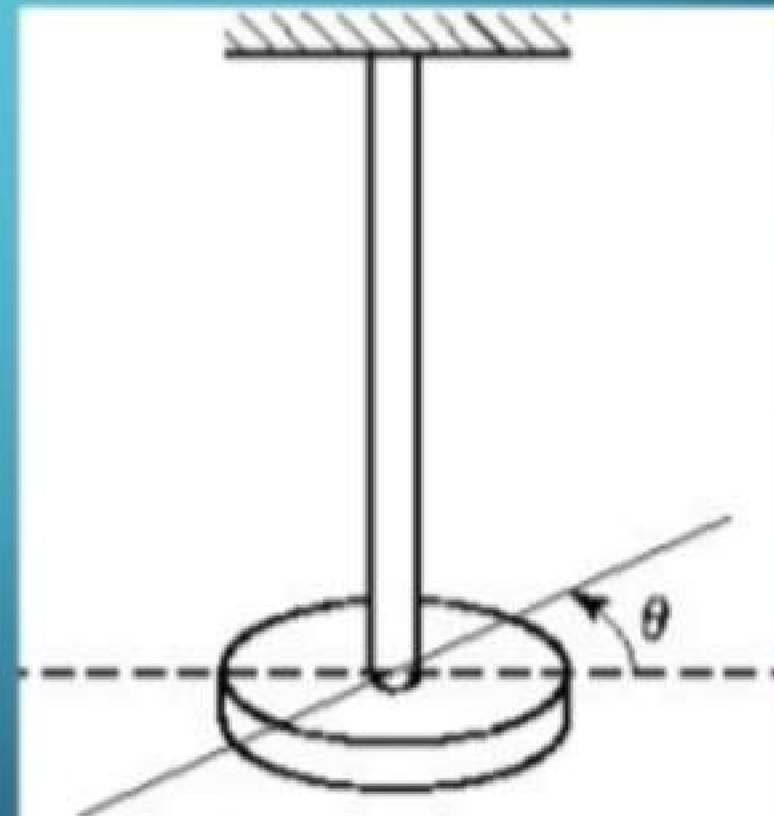
Examples of single degree-of-freedom systems:



(a) Slider-crank-spring mechanism

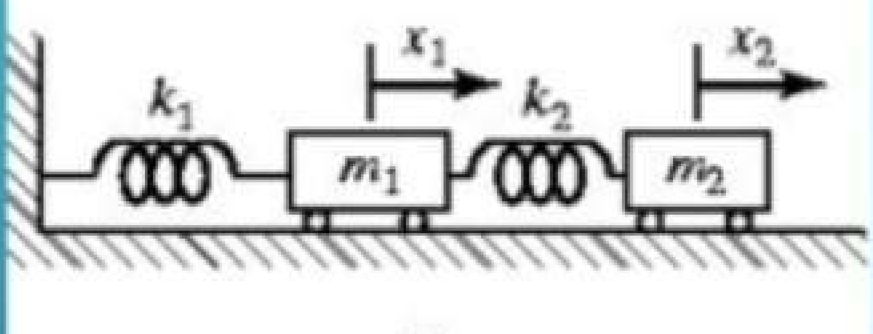


(b) Spring-mass system

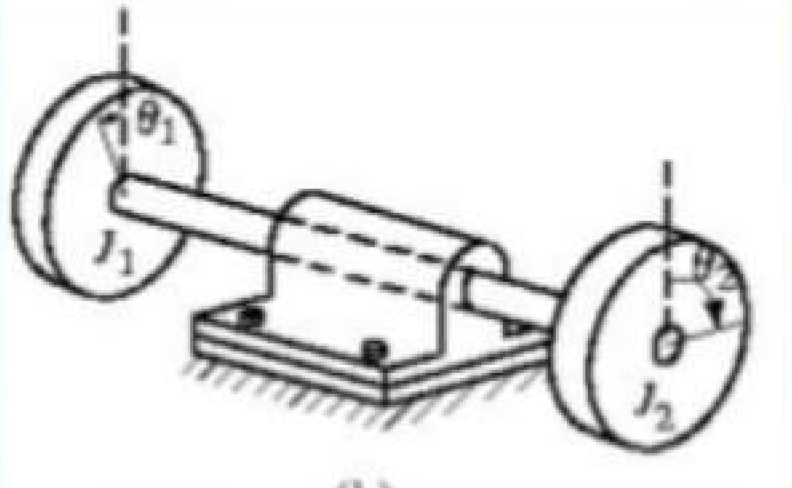


(c) Torsional system

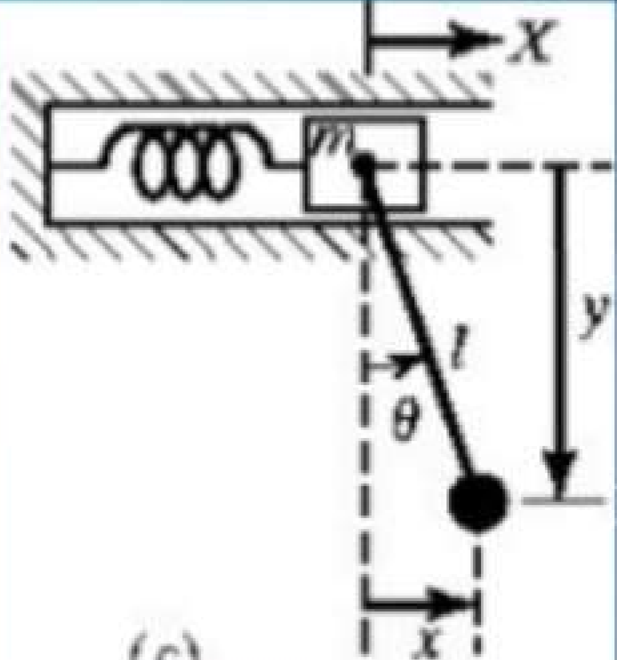
Examples of Two degree-of-freedom systems:



(a)

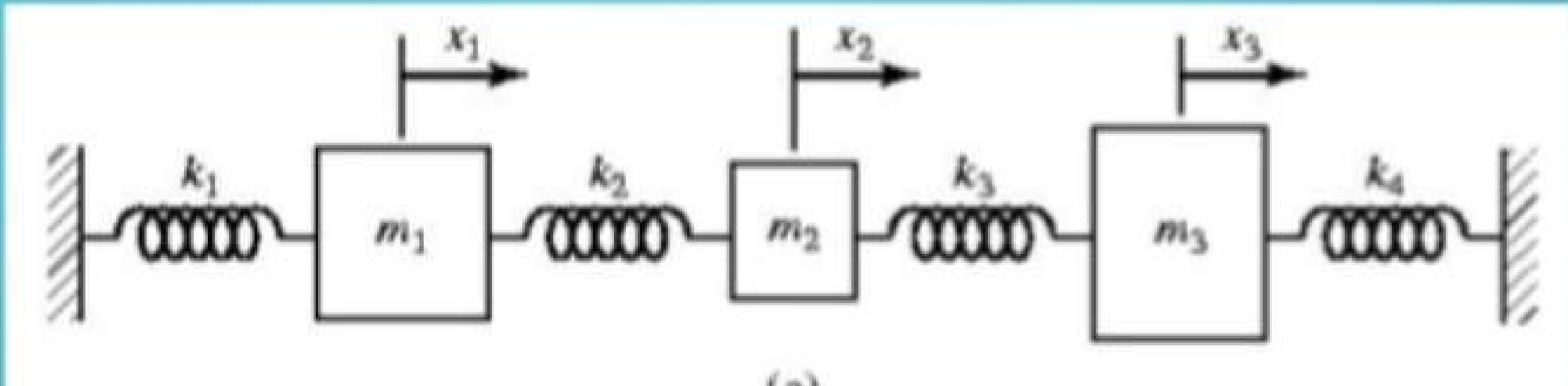


(b)

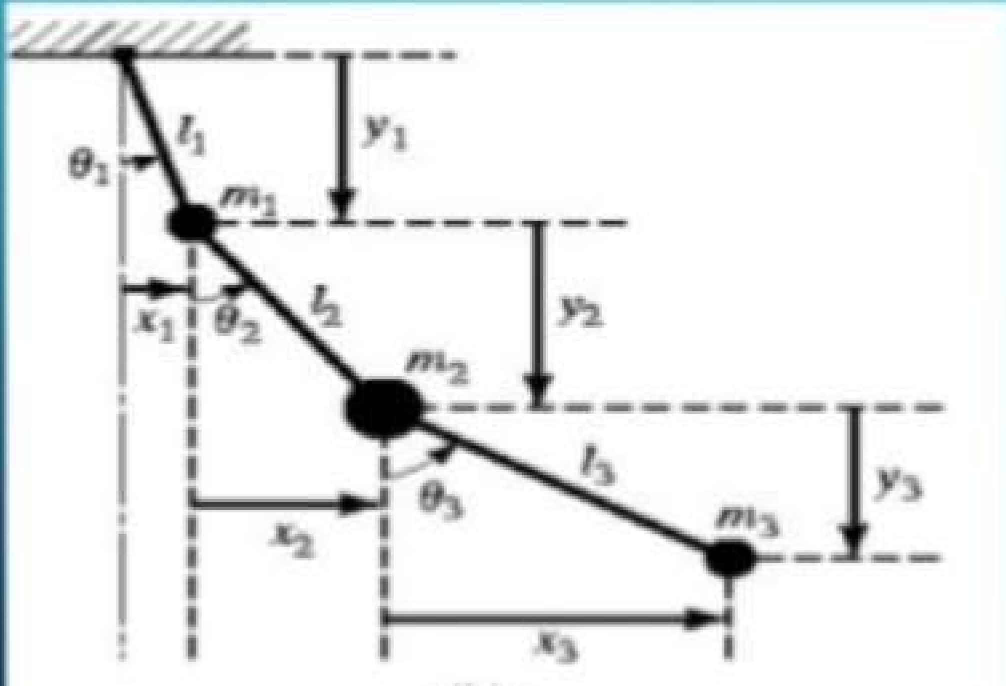


(c)

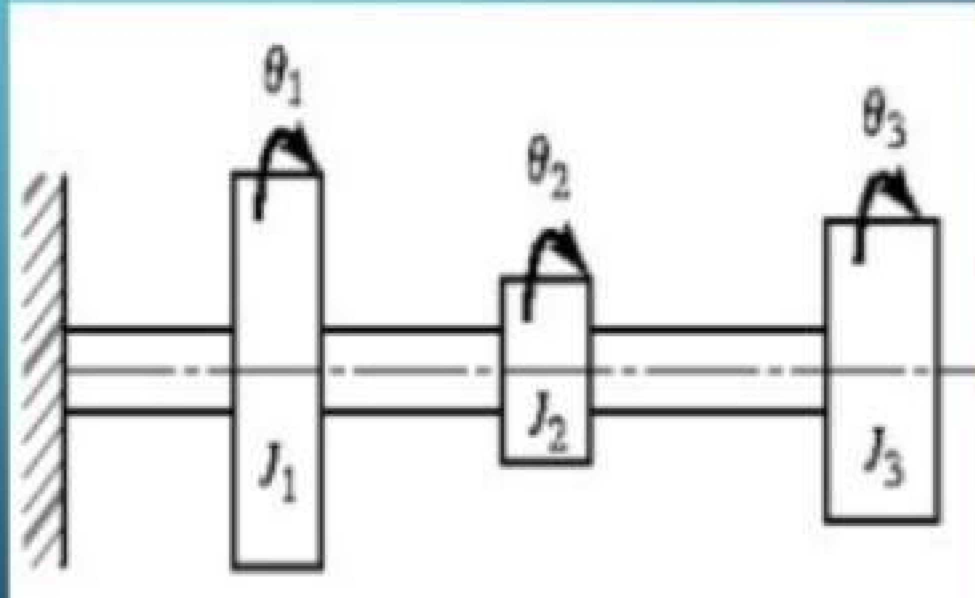
Examples of Three degree of freedom systems:



(a)



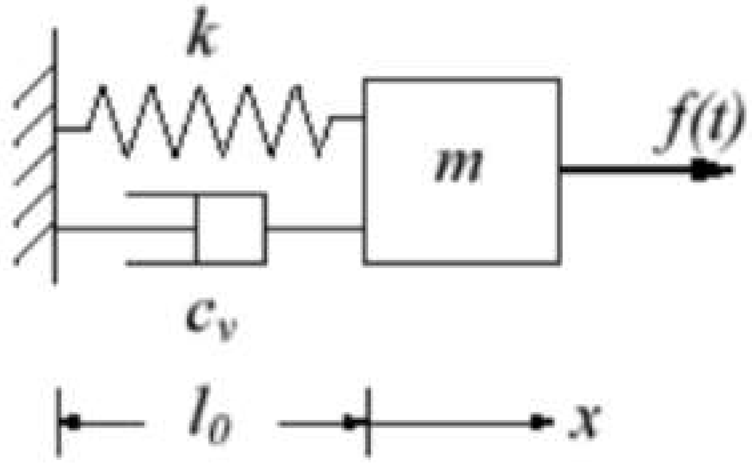
(b)



(c)

Definition

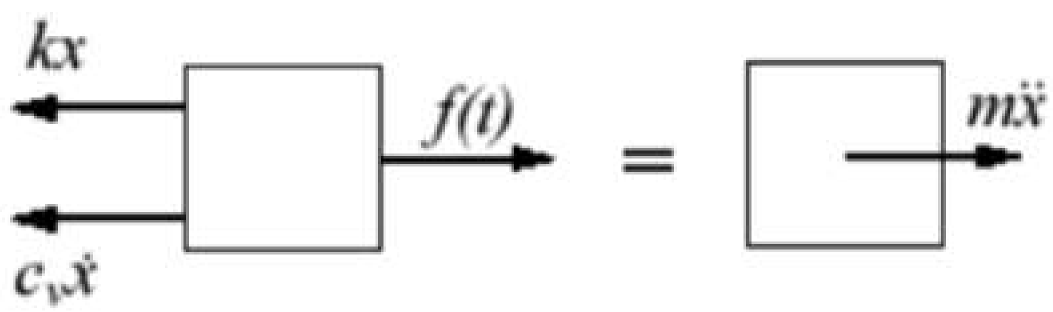
The simplest vibratory system can be described by a single mass connected to a spring (and possibly a dashpot). The mass is allowed to travel only along the spring elongation direction. Such systems are called *Single Degree-of-Freedom* (SDOF) systems and are shown in the following figure,



Equation of Motion for SDOF Systems

SDOF vibration can be analyzed by Newton's second law of motion, *F* = *m***a*. The analysis can be easily visualized with the aid of a free body diagram,

$$F = ma$$



$$-kx - c_v\dot{x} + f(t) = m\ddot{x}$$

The resulting equation of motion is a second order, non-homogeneous, ordinary differential equation:

$$m\ddot{x} + c_v\dot{x} + kx = f(t)$$

with the initial conditions,

$$\begin{cases} x(t=0) = x_0 \\ \dot{x}(t=0) = v_0 \end{cases}$$

The solution to the general SDOF equation of motion is shown in the damped SDOF discussion.

General Solution

1. Stretch-wrapping / Binding / Box pallet application

One of the effective ways to reduce the load turnover risk is to stabilize loads by stretch-wrapping, binding or roll box pallet application.

2. Fixed Stoppers at rack locations

Fixed stoppers installed in rack locations prevent pallet movement into crane aisle during earthquakes. They work effectively when combined with stretch-wrapping and binding.

3. Quake-absorbing Stoppers at rack locations

Quake-absorbing stoppers installed in rack locations absorb the collision energy and reduce load fall-off risk.

4. Quake-absorbing Rack

Dampers installed at the top of AS/RS racks reduce energy of seismic activity by up to 50%, reducing the potential for loads to fall of the racks during earthquakes.

5. Seismically Isolated Rack

A seismically isolated AS/RS base severs AS/RS from direct influence of earthquake tremors.

6. Seismograph / Seismometer

When the seismograph detects a significant tremor, the equipment controller sends cycle stop signals to all equipment to reduce the potential for damage.

7. Location control by stability

Ordinary random storage management places pallets in any available location. This means that tall, unstable loads may be stored on higher levels where the impact of earthquakes is greater.

Green's function

Green's function for simulating **earthquakes**

In linear elastodynamics, the displacement time field due to a unit impulsive point load, precisely defined in both space and time, is called **Green's function**.

Equivalent static force analysis

- The concept is a dynamic analysis into partly dynamic and partly static analyses for finding the maximum displacement.
- Is restricted only to a single mode of vibration of the structure.

Equivalent static lateral force analysis is based on the following assumptions,

1) Assume that structure is rigid.

2) Assume perfect fixity between structure and foundation.

3) During ground motion every point on the structure experience same accelerations

4)Dominant effect of earthquake is equivalent to horizontal force of varying magnitude over the height.

5) Approximately determines the total horizontal force (Base shear) on the structure.

Earthquake Lateral Force Analysis

- The design lateral force shall first be computed for the building as a whole.
- This design lateral force shall then be distributed to the various floor levels.
- There are two commonly used procedures for seismic design lateral forces:

1. Equivalent static force analysis

2. Dynamic analysis

Dynamic Analysis

Dynamic analysis is classified into two types, namely,

- *Response spectrum method and Time history method*

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.

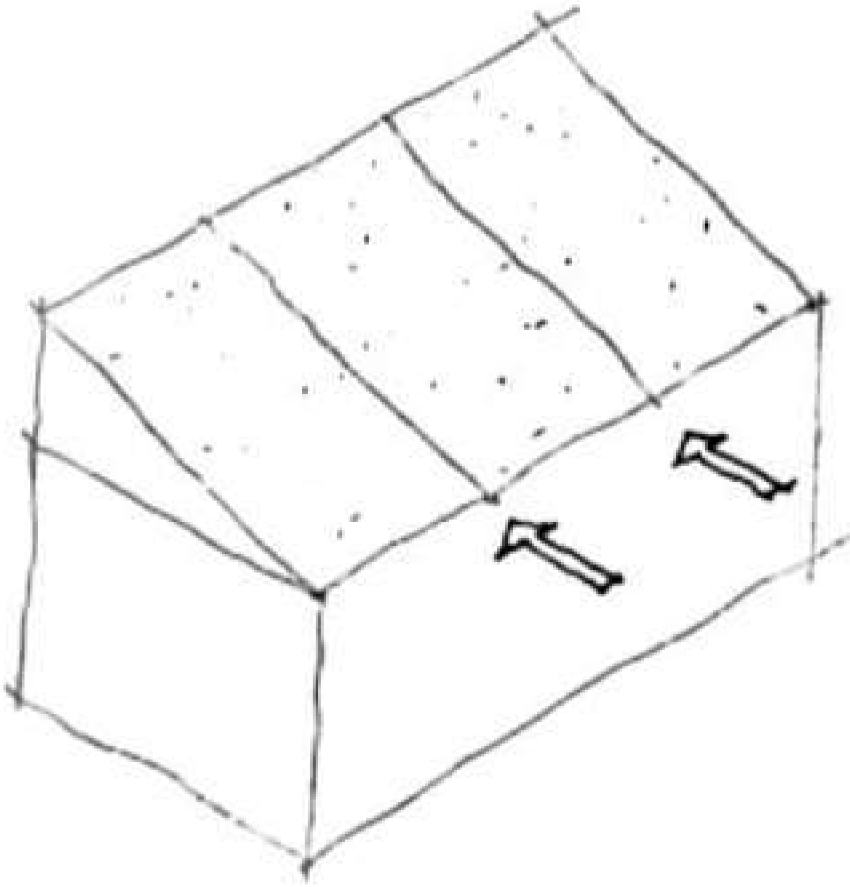
Response Spectrum Method: *Response spectrum method of analysis shall be* performed using the design spectrum specified in Clause 6.4.2 or by a site specific design, spectrum mentioned in Clause 6.4.6 of IS 1893 (2002)

When dynamic analysis is carried out either by the Time History Method or by the Response Spectrum Method, the design base shear computed from dynamic analysis (V_B) shall be compared with a base shear calculated using a fundamental period T_a ($\bar{V}_B$), where T_a is as per Clause 7.6. If base shear obtained from dynamic analysis (V_B) is less than base shear computed from equivalent static load method ($\bar{V}_B$ *i.e., using* T_a *as per Clause 7.6*), then as per Clause 7.8.2, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by ratio $\frac{\bar{V}_B}{V_B}$.

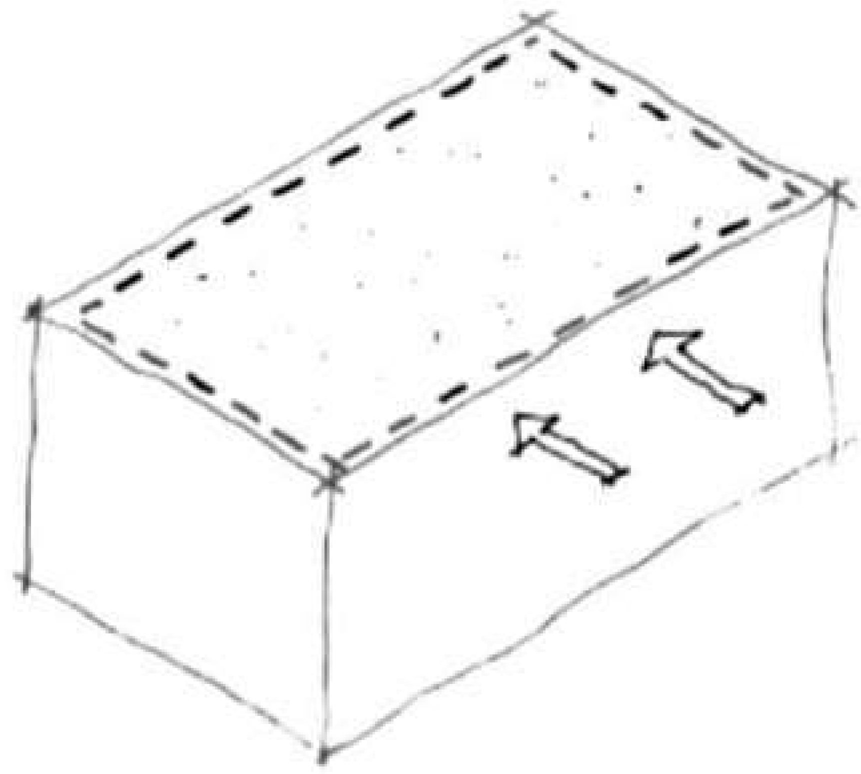
Horizontal diaphragms

Similar to a shear panel, a horizontal diaphragm is a horizontal truss (in a roof plane) or solid sheet element (in a floor). It is placed between vertical elements to transfer lateral loads to the vertical elements, such as shear panels, vertical trusses or moment frames.

In timber floors, for example, the floor sheeting carries the shear forces , while perimeter joists and plates carry the in-plane bending forces. The thickness of sheeting and size of joists will often be governed by vertical rather than lateral forces. Floor diaphragms in newer timber buildings are nearly always particleboard or plywood.



A roof plane diaphragm in a low-pitch timber roof



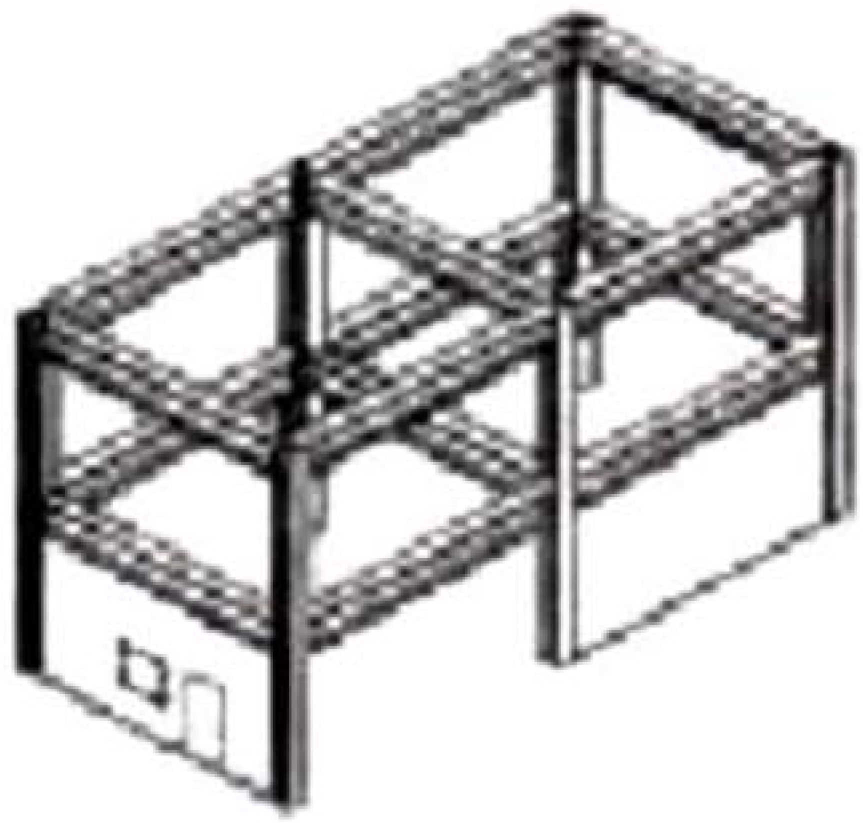
A concrete floor in a multi-storey building

Two examples of a horizontal diaphragm used to transfer lateral loads to the foundation during a seismic event.

Most building designs use some form of diaphragm at every floor and at roof level. A floor plate is an example of a horizontal diaphragm, and in many designs, a diaphragm will also serve as the building's floor or roof element.

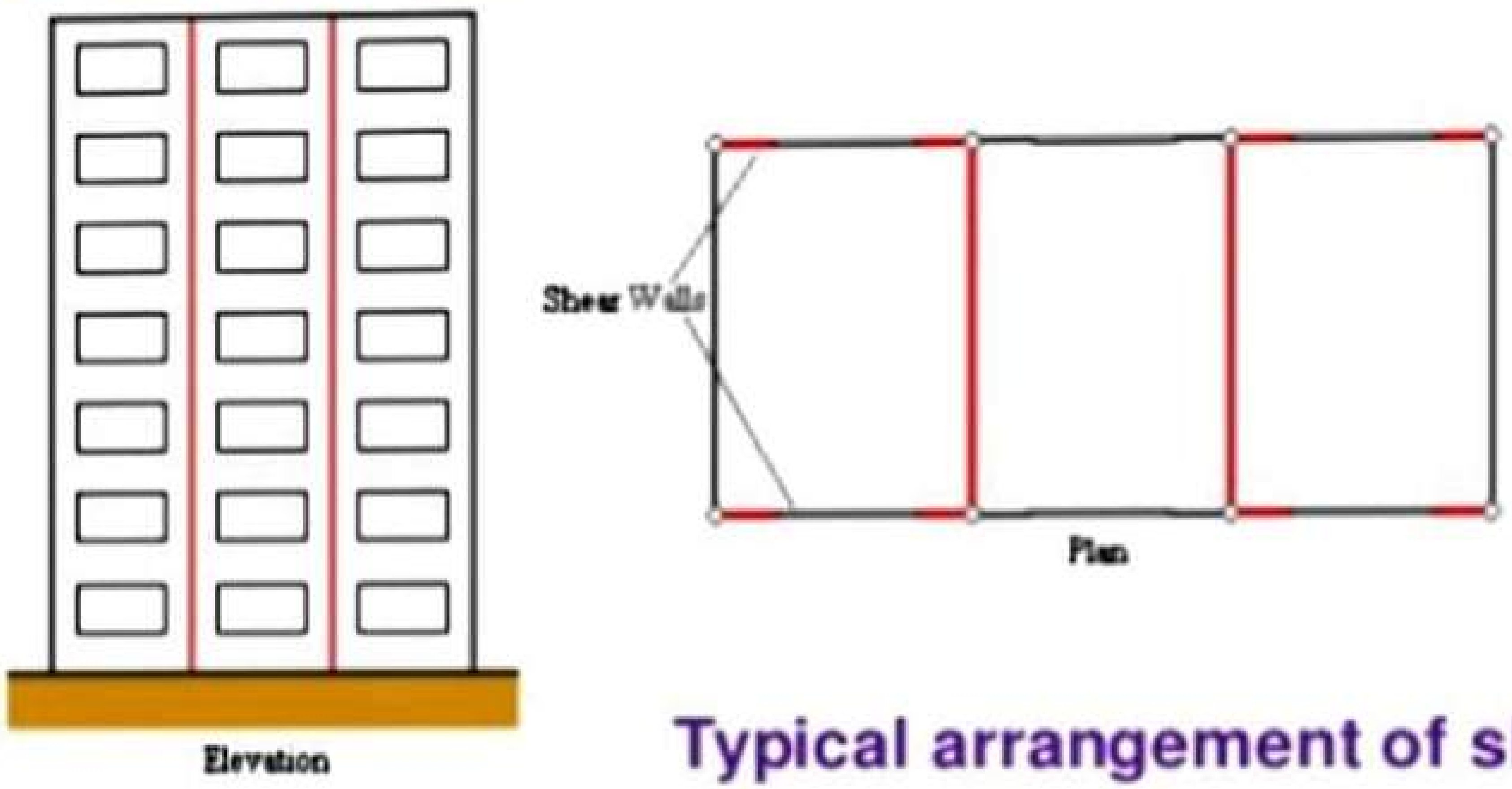
1. SHEAR WALLS

- First used in 1940, may be described as vertical, cantilevered beams, which resist lateral wind and seismic loads acting on a building transmitted to them by the floor diaphragms.



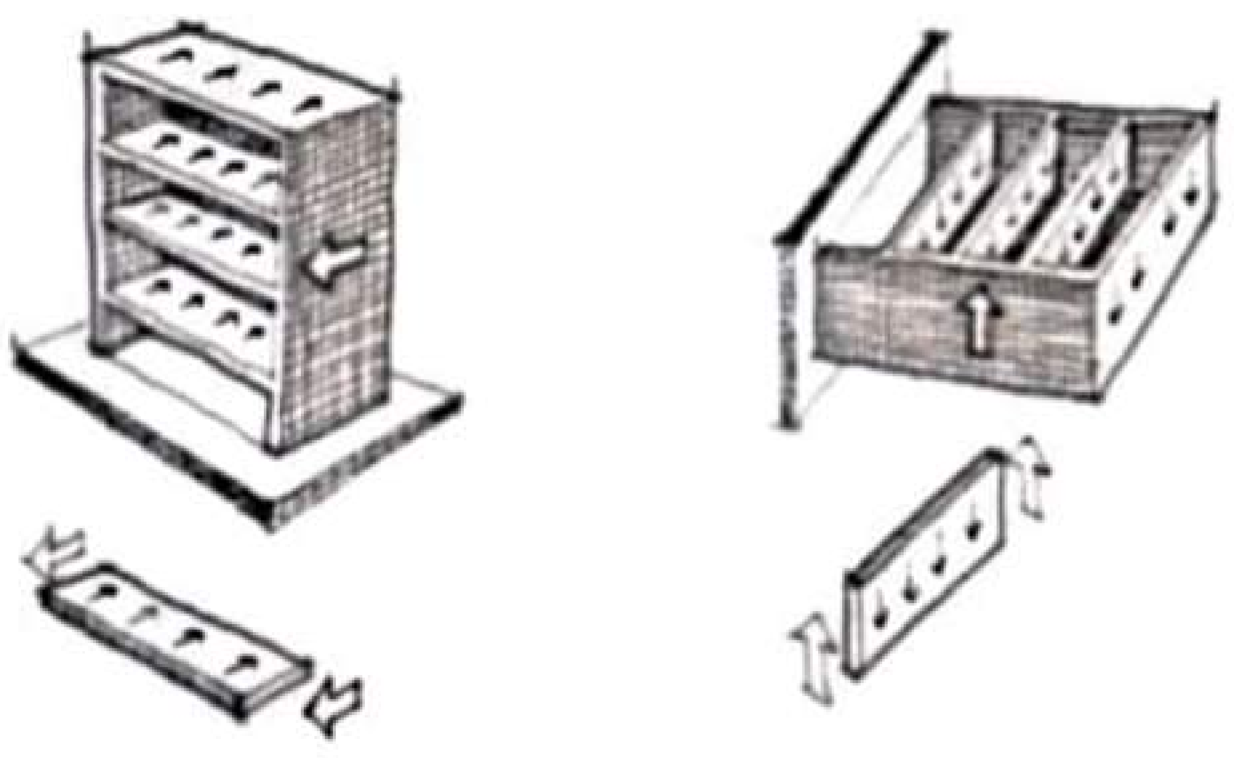
Building frame system with Shear Walls

EFFECTS IN EARTHQUAKE



Typical arrangement of shear walls

- A simple building with shear walls at its ends. Ground motion enters the building and creates inertial forces which move the floor diaphragms. This movement is resisted by the shear walls, and the forces are transmitted back down to the foundation.



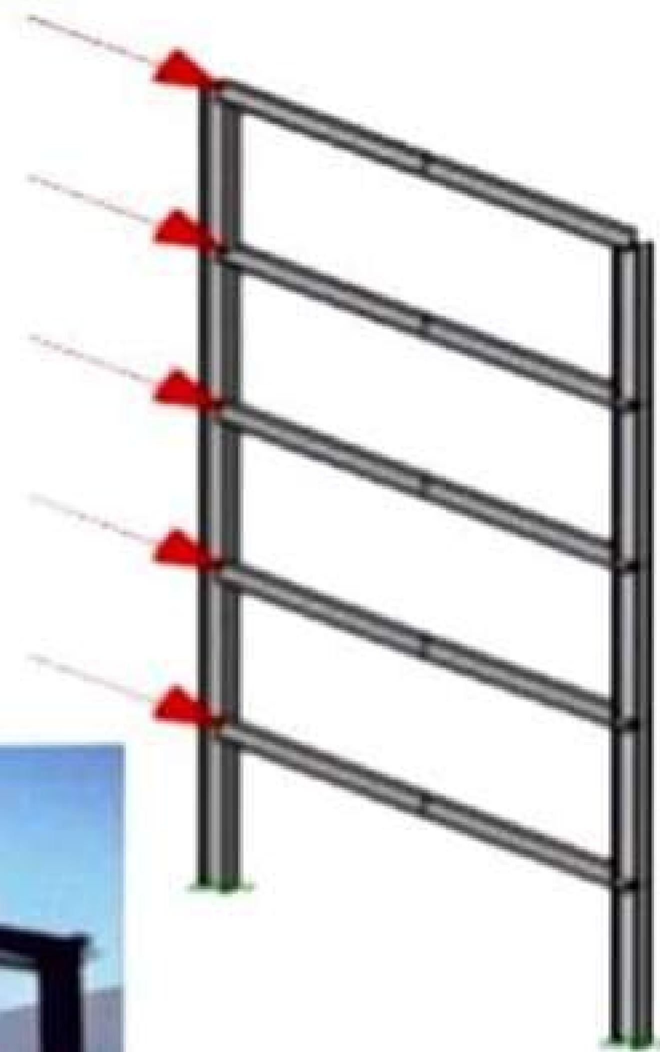
Shear wall: vertical analogy

2. MOMENT-RESISTING FRAMES

Moment-resisting frames are structures having the traditional beam-column framing. They carry the gravity loads that are imposed on the floor system. The floors also function as horizontal diaphragm elements that transfer lateral forces to the girders and columns. In addition, the girders resist high moments and shears at the ends of their lengths, which are, in turn, transferred to the column system. As a result, columns and beams can become quite large.

MOMENT-RESISTING FRAMES

- Moment-resisting frames can be constructed ofsteel, concrete, or masonry.
- Consist of beams and columns in which bending of these members provides the resistance to lateral forces.
- There are two primary types of moment frames, ordinary and special.
- Special moment-resisting frames are detailed to ensure ductile behavior of the beam-to-column joints and are normally used in zones of higher seismicity.
- Steel moment-resisting frames have been under intensive study and testing.



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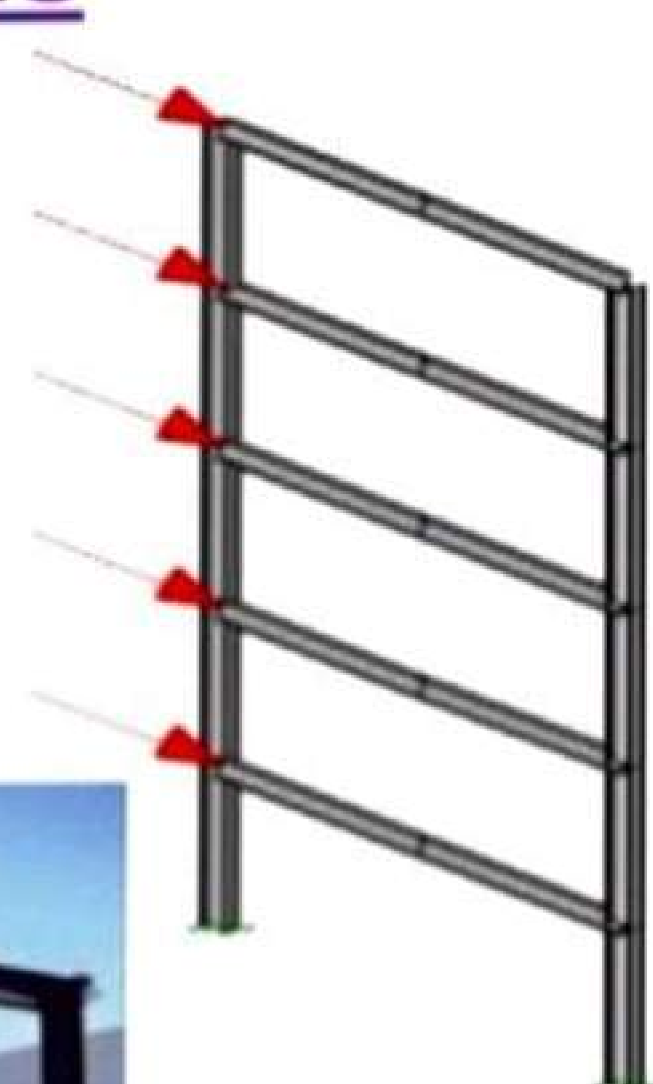


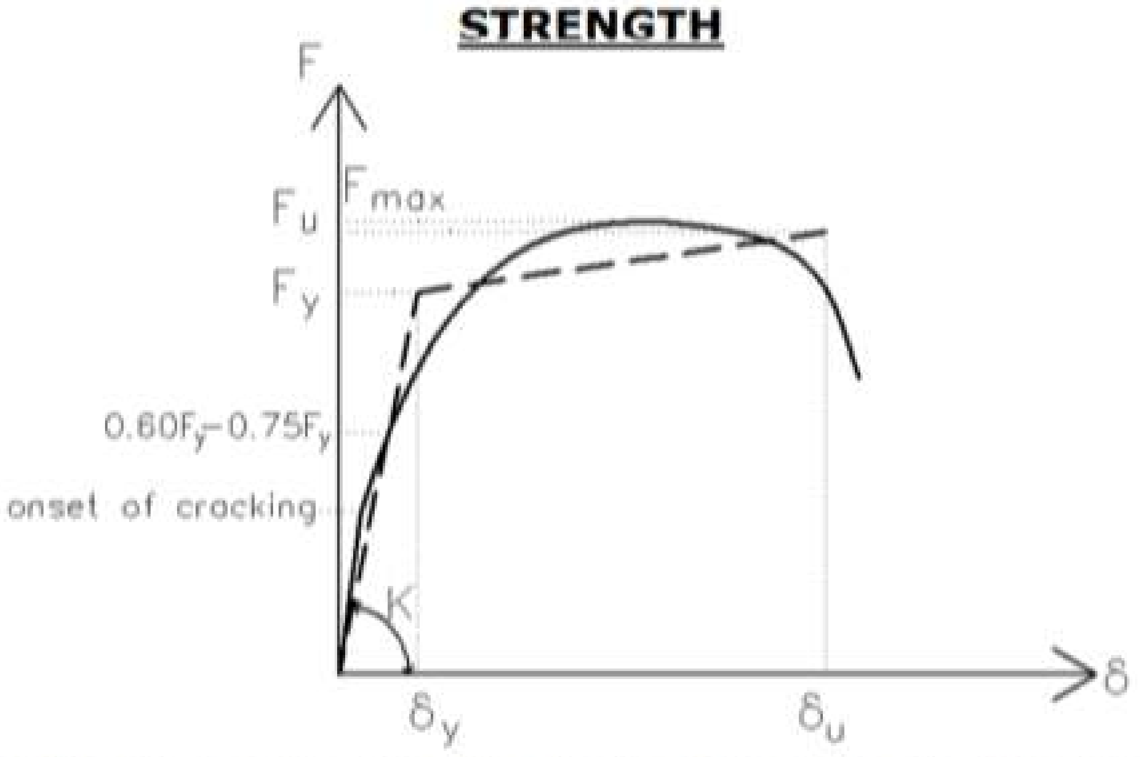
Seismic design concepts

A degree of seismic resilience can be achieved by applying a sound understanding of structural engineering and construction principles to the structural elements and system that make up the building. There are several fundamental concepts that the designer can draw on to achieve this goal.

Designing a building is a complex and specialist undertaking that is well beyond the scope of this guidance, but a few of the basic concepts are briefly introduced here. In each case, the nuances, complexities and engineering understanding that make these systems feasible in practice has been simplified. In any real-world situation, it is important to seek proper advice from a professional structural engineer before any work is undertaken.

Basic quantities of earthquake engineering

STRENGTH



In earthquake engineering the desired strength of a structure is usually expressed in terms of resistance to lateral forces.

Strength is quantified in terms of the yield force F_y and the ultimate force F_u of the idealized bilinear curve or sometimes of the maximum force of the real curve F_{max}

Basic quantities of earthquake engineering

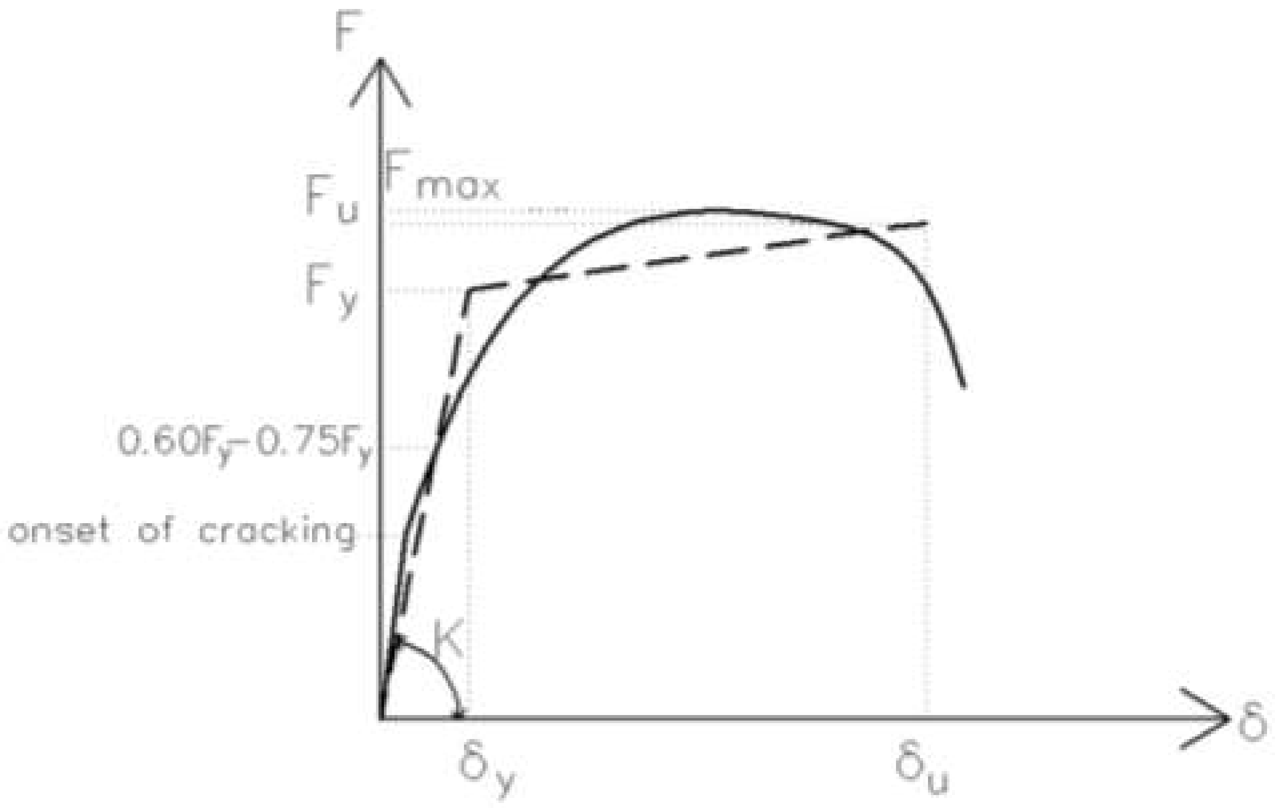
STRENGTH

For reinforced concrete buildings the increase of strength is usually achieved by:

- The increase of the elements' reinforcement.
- The use of higher quality materials, for example concrete C20 instead of C16 or steel S500 instead of S400 (the increase of the steel quality is usually more important for the increase of strength of a R/C element or structure)
- The increase of the elements' sections usually leads to an increase of strength as it is combined with the increase of the elements' reinforcement.

Basic quantities of earthquake engineering

STIFFNESS



The slope of the idealized linear elastic response $K=F_y/\delta_y$ is used to quantify stiffness

Basic quantities of earthquake engineering

STIFFNESS

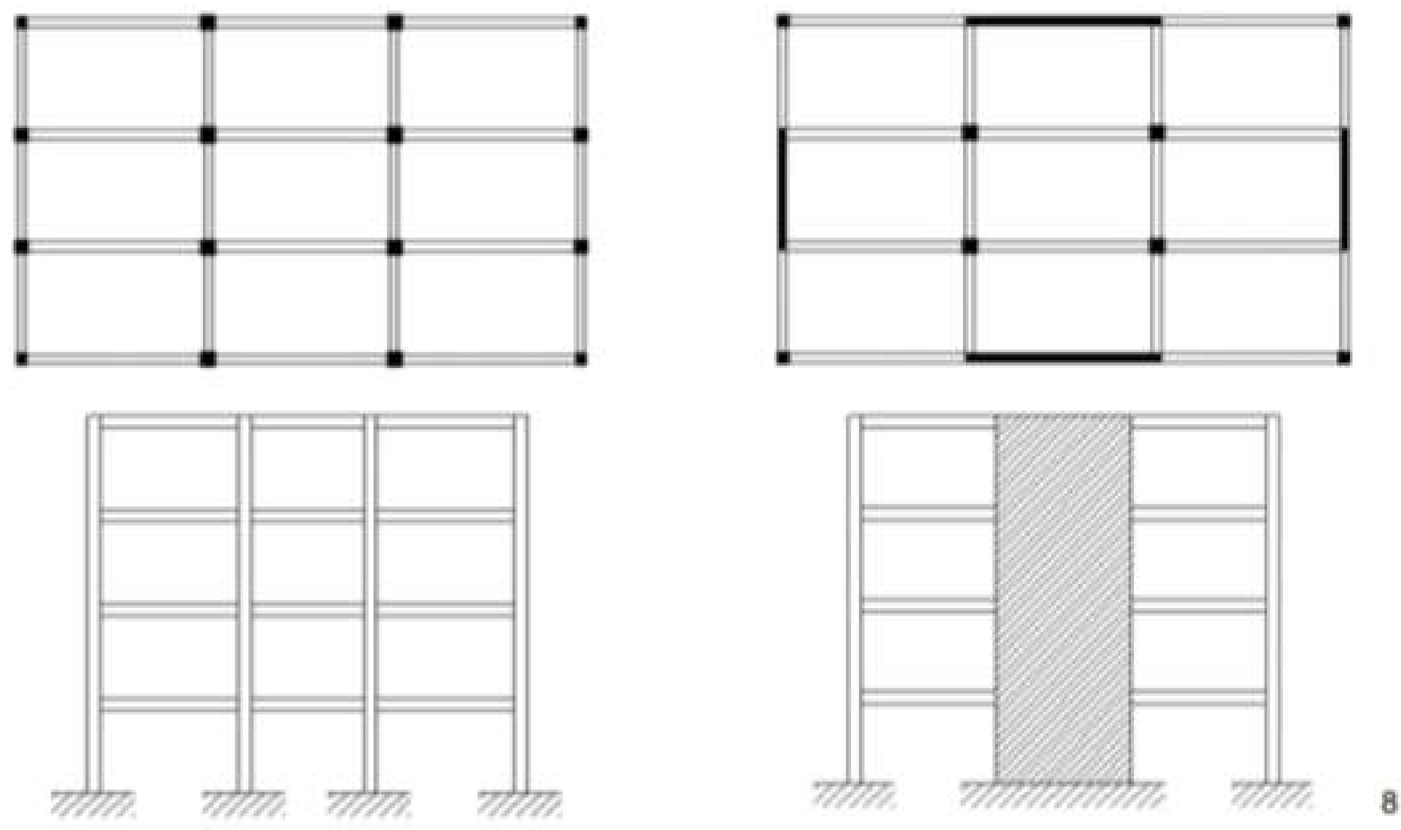
For reinforced concrete buildings the increase of stiffness is usually achieved by:

- The use of higher quality concrete as the modulus of elasticity is increasing when the concrete quality is higher (values of the modulus of elasticity for each concrete material can be found in EC2)
- The increase of the elements' sections
- The use of shear walls. This is the most common and the most effective way to increase the stiffness of a R/C building

Basic quantities of earthquake engineering

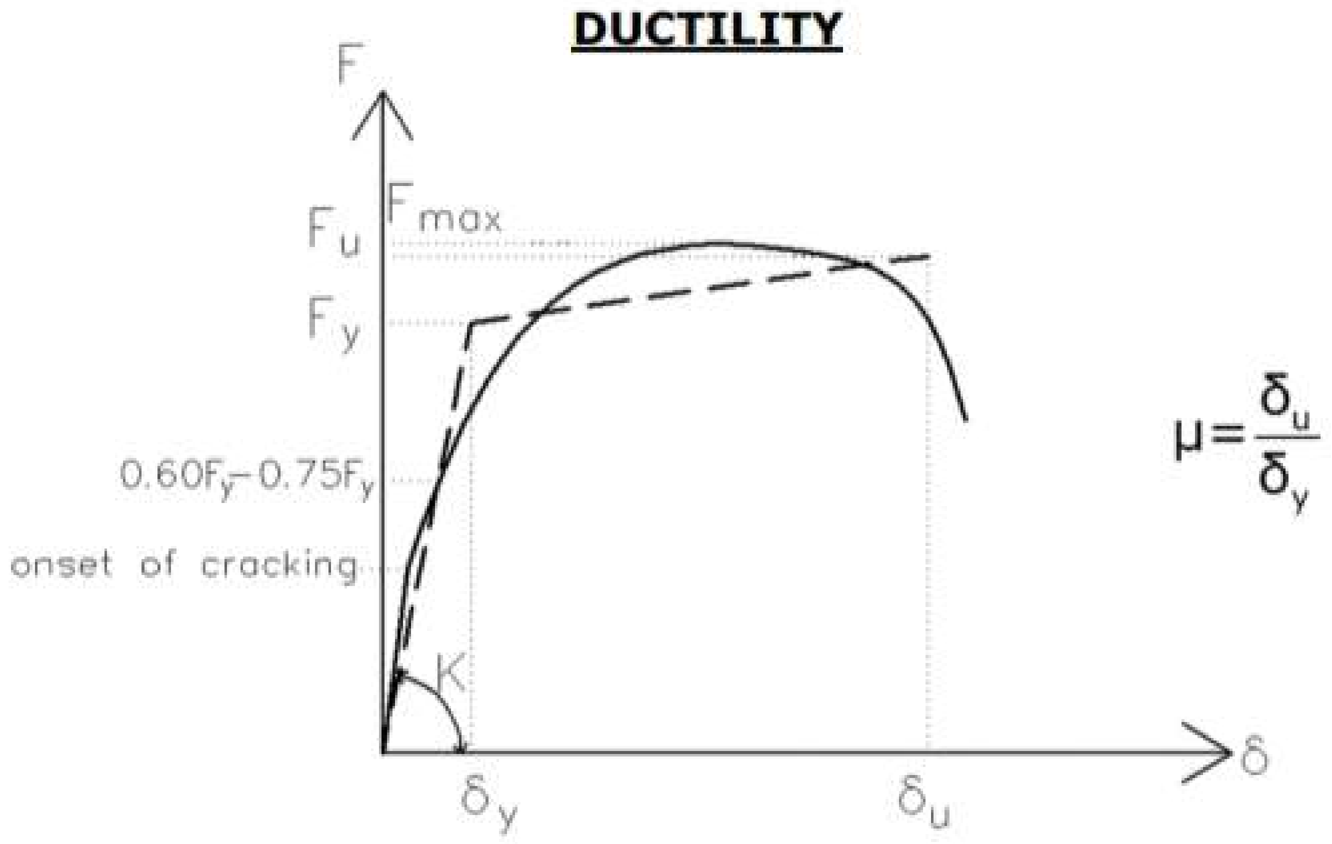
STIFFNESS

Flexible structures (moment frames) Stiff structures (dual systems)



Basic quantities of earthquake engineering

DUCTILITY



The ratio of the ultimate displacement δ_u to the yield displacement δ_y is called **ductility factor** and is used to quantify ductility

Basic quantities of earthquake engineering

DUCTILITY

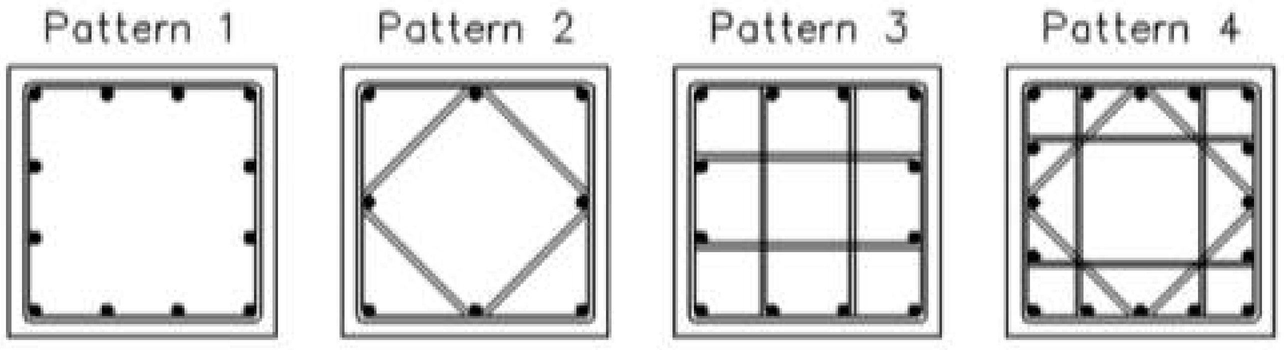
For reinforced concrete buildings the increase of stiffness is usually achieved by:

- Adequate confinement of critical zones with closely spaced hoops or ties
- Adequate shear reinforcement so that bending mode of failure is secured and shear failure is excluded (flexural failure if usually ductile while shear failure is brittle)
- Design of strong columns and weak beams, as the columns are more important for the global capacity of the building
- Adequate anchorage of longitudinal bars so that sliding is excluded

Basic quantities of earthquake engineering

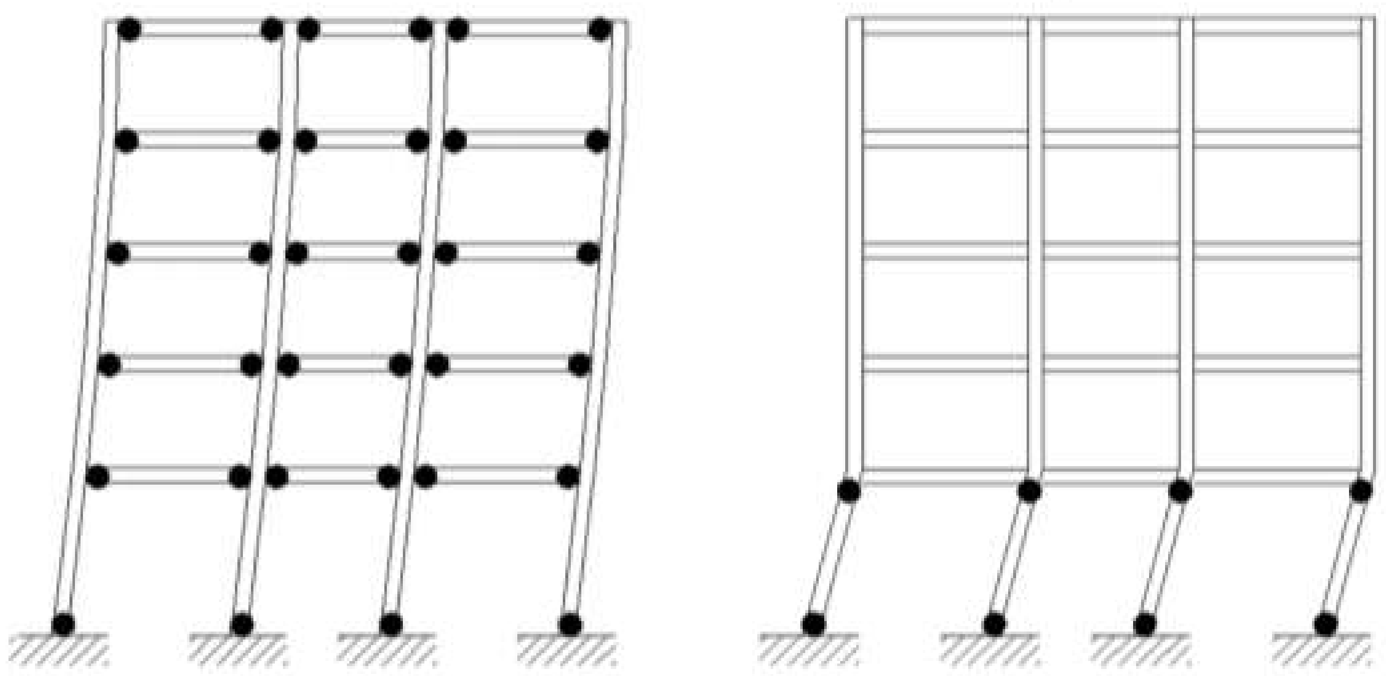
DUCTILITY

Typical hoop patterns for R/C columns that provide from poor (pattern 1) to very high (pattern 4) confinement



Basic quantities of earthquake engineering

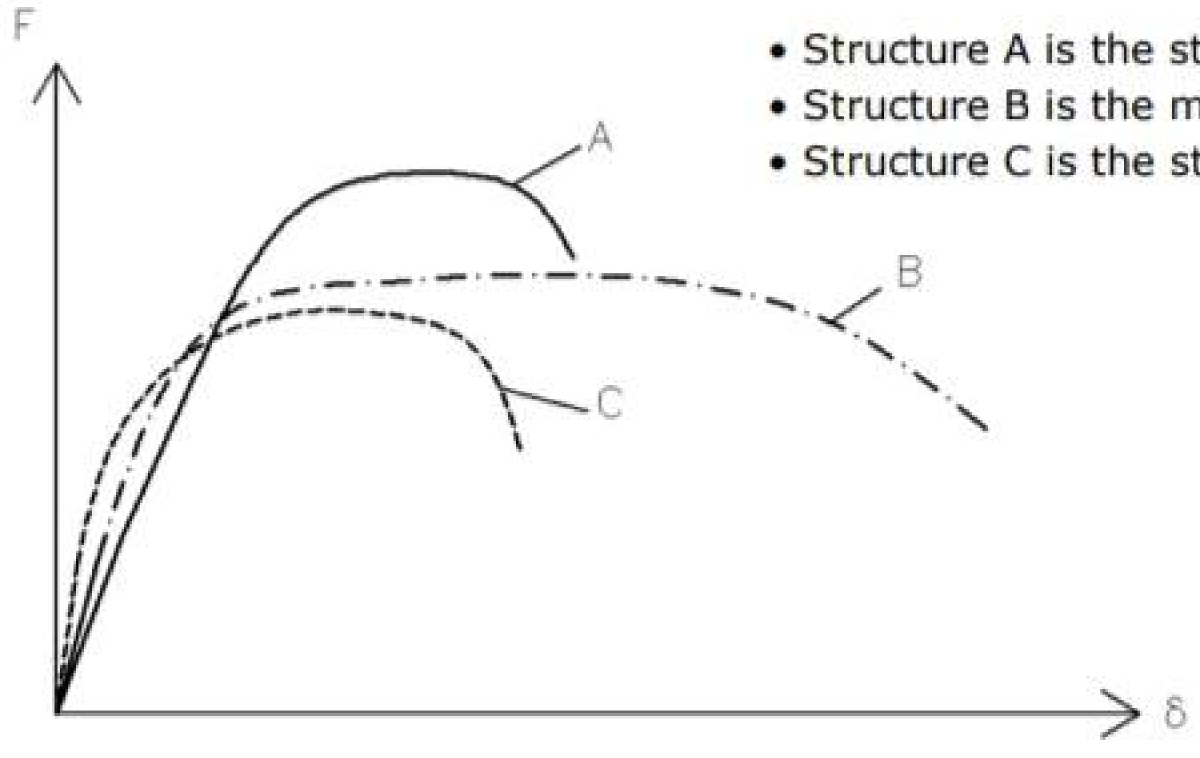
DUCTILITY



The first type of behaviour is achieved using the strong column – weak beam design rule

Basic quantities of earthquake engineering

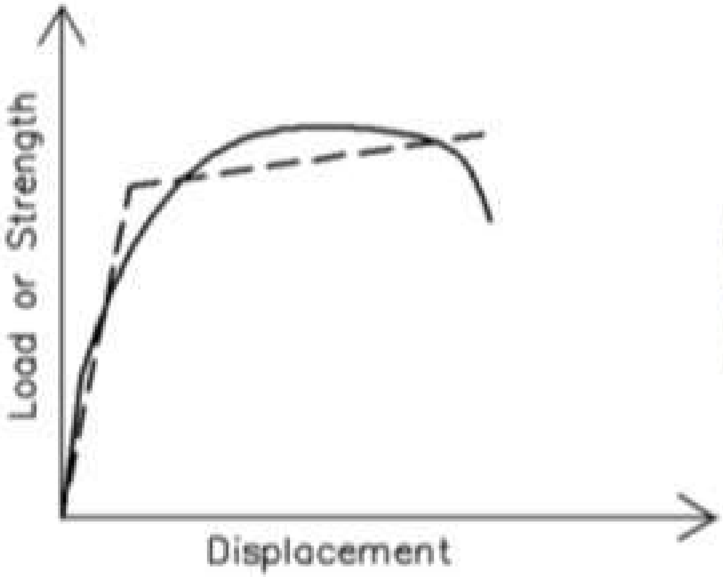
Comparison of strength, stiffness and ductility



- Structure A is the strongest
- Structure B is the most ductile
- Structure C is the stiffest

Basic quantities of earthquake engineering

Typical load-displacement relationship for a reinforced concrete element (or structure)



For design purposes a bilinear approximation of the real curve is usually used to simplify the calculations

This curve is usually in terms of:

- Base Shear – Roof displacement when refering to a structure
- Force – displacement or Moment – Curvature (or rotation) when refering to a single element's section

Basic quantities of earthquake engineering

Shear failure of R/C columns. Brittle failure with no warning.

Basic quantities of earthquake engineering

Shear failure of R/C columns. Brittle failure with no warning.

Basic quantities of earthquake engineering

Flexural damage. The drop of strength of the building is relatively small.

Basic quantities of earthquake engineering

Failure of a R/C column. Insufficient confinement reinforcement (almost no hoops)

Basic quantities of earthquake engineering

Soft-storey mechanism is one of the most common reason for the collapse of buildings during an earthquake.

Basic quantities of earthquake engineering

Soft-storey mechanism. This was a 3-storey building!

Review of Indian seismic code, IS 1893 (Part 1) : 2002

Sudhir K Jain

The Indian seismic code IS 1893 has now been split into a number of parts and the first part containing general provisions and those pertaining to buildings has been released in 2002. There has been a gap of 18 years since the previous edition in 1984. Considering the advancements in understanding of earthquake-resistant design during these years, the new edition is a major upgradation of the previous version. This paper reviews the new code; it contains a discussion on Clauses that are confusing or vague and need clarifications immediately. The typographical and editorial errors are pointed out. Suggestions are also included for next revision of the code.

With rapid strides in earthquake engineering in the last several decades, the seismic codes are becoming increasingly sophisticated. The first Indian seismic code (IS 1893) was published in 1962 and it has since been revised in 1966, 1970, 1975 and 1984[1]. More recently, it was decided to split this code into a number of parts, and Part 1 of the code containing general provisions (applicable to all structures) and specific provisions for buildings has been published[2].

Considerable advances have occurred in the knowledge related to earthquake resistant design of structures during the 18 years interval between the two editions of the code[3]. Some of these new developments have been incorporated in the 2002 version of the code, while many others have been left out so that the implementation of the code does not become too tedious for Indian professional engineers. For example, in the United States, the codes are revised every three years, and hence, a typical building code in the United States has acquired sophistication gradually over about six revisions during these 18 years. Since the Indian code has had to make a quantum jump with respect to many of the provisions, it still requires considerable effort for an average professional engineer to fully appreciate the new code and to be able to implement it correctly.

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

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

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

It is not uncommon to have errors or omissions in the codes. However, it is important to quickly correct these errors or omissions. This paper reviews the code and the suggestions for changes in the next revision are listed. Also listed are Clauses that are confusing or vague and need clarifications immediately. Finally, the typographical and editorial errors are pointed out.

Philosophical changes in the new code

As compared to the previous version, several major modifications have been incorporated in the new code. Some of the important modifications include the following.

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

high value of natural period and thus low seismic force.

Comments and suggestions

Other parts of IS 1893 (Foreword, page 2)

The fourth revision of the code (IS 1893 : 1984) covered buildings, water tanks, stacklike structures, bridges, dams and retaining walls. When carrying out the fifth revision, it was decided to split the code into five parts.

Part 1: General provisions and buildings

Part 2: Liquid retaining tanks – elevated and ground supported

Part 3: Bridges and retaining walls

Part 4: Industrial structures including stack like structures

Part 5: Dams and embankments

While Part 1 of the code has been released, the other parts are still in various stages of development. To address the situation that other parts of the code are not yet released, *Note* on page 2 of the code states:

"Pending finalization of Parts 2 to 5 of IS 1893, provisions of Part 1 will be read along with the relevant Clauses of IS 1893 : 1984 for structures other than buildings".

This is problematic in many situations, for instance, let us consider the case of overhead water tanks. In the 1984 code seismic design force for water tanks depends on parameters β, *I*, and α_o. In the new code, terms β and α_o do not exist. Instead, one now needs the response reduction factor (R) for water tanks which is supposed to be provided in Part 2 of code. Clearly, there is no way one can combine Part 1 of the 2002 version with the 1984 code for water tanks and there is bound to be dispute and possibly litigation in case of lump sum contracts for water tanks. Similarly, it is not possible to implement the new provisions for bridges, stacks and dams. However, the problem is most serious for water tanks: bridges are designed as per the provisions of IRC or Indian

Risk level

Para 5 on page 3 of the code states

"The seismic hazard level with respect to ZPA at 50 percent risk level and 100 years service life goes on progressively increasing ...".

This statement is made in the context of earthquake geology of the country. However, it may give a false impression that the values of ZPA (denoted by Z) given in the code are for 50 percent risk level and 100 years service life. Such a confusion needs to be avoided by modifying this statement as "The seismic hazard level goes on progressively increasing....".

Peak ground acceleration

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(*i*) what value of service life should be considered for his structure

(*ii*) if he is willing to reduce the service life of his structure say from 100 years to 50 years, how much reduction in the seismic design force would be allowed by the code.

The fact remains that the values of Z specified in the code were arrived at empirically based on engineering judgment and no explicit calculations were done or envisaged for service life. Hence, it is best to drop the mention of "service life". This suggestion is consistent with the fact that in the definition of Z in Clause 3.33 also, the code makes no mention of service life.

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It is best to mention later versions of the references. Further, a considerable part of the code is based on two published articles of the authors[7,8]. These two articles could provide additional background materials to the engineer and hence it is appropriate to add these to the list in the code.

Response spectrum (Clauses 3.5, 3.27, 3.30, 6.4, ...)

In the code, different terms are used for response spectrum, for example, "Design Acceleration Spectrum" (Clause 3.5); "Response Spectrum" (Clause 3.27); "Acceleration Response Spectrum" (used in Clause 3.30); "Design Spectrum" (title of Clause 6.4); "Structural Response Factor"; "Average response acceleration coefficient" (see terminology of S_a/g on p. 11), etc. It is best to use one single term consistently to avoid confusion. It is suggested that the term be "Design Acceleration Spectrum" for the plot of response spectrum with natural period, and the term be "Response Acceleration Coefficient" for the value of S_a/g for a given value of natural period.

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Both these definitions are quite incomplete and do not tell anything specific to the user. For instance, what is meant by "reasonable expectation"! Also, the design life of different structures may be different and yet the code specifies the same PGA value regardless of the design life of a structure.

Let us consider the use of these terms in the International Building Code (IBC). The IBC 2003 defines MCE as corresponding to 2 percent probability of being exceeded in 50 years (2,500 year return period), and the DBE as corresponding to 10 percent probability of being exceeded in 50 years (475 year return period). Clearly, there is no ambiguity in IBC on this account.

Since the seismic zone map in Indian code is not based on probabilistic hazard analysis, it is not possible to deduce the probability of occurrence of a certain level of shaking in a given zone based on this code. Therefore, use of terms such as MCE and DBE do not add any new information, and can sometimes cause confusion and disputes. For instance, someone may argue that the value of $Z=0.36$ for MCE in zone V of the code implies that the PGA value in zone V can not exceed $0.36g$, which is not the intention of the code. For instance, during 2001 Bhuj earthquake, ground acceleration $\sim 0.6g$ has been recorded at Anjar located at 44 km from epicentre.

Clause 6.1.3 implies that DBE relates to the "moderate shaking" and MCE relates to the "strong shaking". This is at variance with the definitions of MCE and DBE given in Clauses 3.19 and 3.6 as mentioned above. Again, it clearly shows that there is an element of confusion about the definition and implications of these two terms. Considering that these terms do not add any substantial value to the codal provisions, the two terms may be dropped from the code.

Centre of stiffness and centre of rigidity

In Clause 4.5, centre of stiffness is defined, but in Clause 4.21 while defining static eccentricity, the term centre of rigidity is used. Both centre of stiffness (CS) and centre of rigidity (CR) are the same terms for purposes of the code and hence to avoid confusion, it is best to use only one term consistently. It is proposed that centre of stiffness be replaced by the term centre of rigidity wherever it appears in the code.

Clause 4.5 defines centre of stiffness as "The point through which the resultant of the restoring forces of a system acts." This definition is incomplete. For single storey buildings it may be defined as:

> "If the building undergoes pure translation in the horizontal direction (that is, no rotation or twist or torsion about vertical axis), the point through which the resultant of the restoring forces acts is the centre of stiffness".

For multi-storeyed buildings, centre of rigidity (stiffness) can be defined in two ways.

> *All floor definition of centre of rigidity*: Centre of rigidities are the set of points located one on each floor, through which application of lateral load profile would cause no rotation in any floor, *Fig* 1(*a*). As per this definition, location of CR is dependent on building stiffness properties as well as on the applied lateral load profile.
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> *Single floor definition of centre of rigidity:* Centre of rigidity of a floor is defined as the point on the floor such that application of lateral load passing through that point does not cause any rotation of that particular floor, while the other floors may rotate *Fig* 1(*b*). This definition is independent of applied lateral load.

The two definitions for multi-storey buildings will give somewhat different values of design eccentricity but the

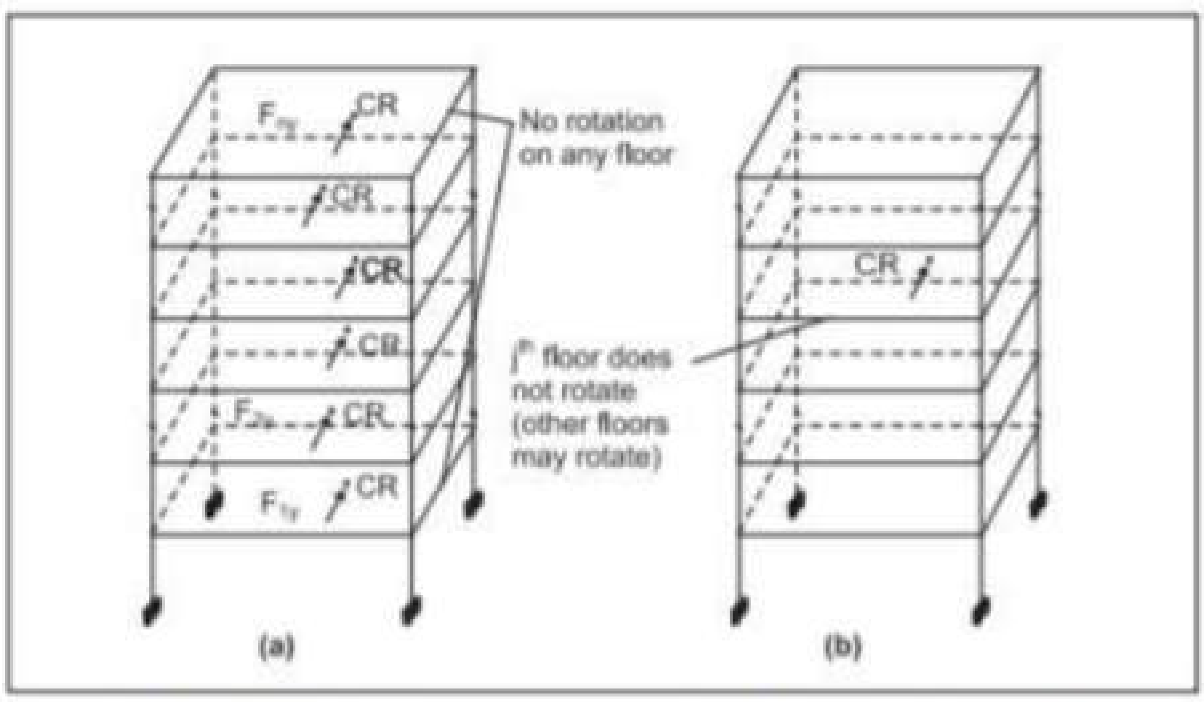

Fig 1 ***(a)*** **All floor definition of centre of rigidity** ***(b)*** **Single floor definition of centre of rigidity**

Equivalent static force analysis

- The concept is a dynamic analysis into partly dynamic and partly static analyses for finding the maximum displacement.
- Is restricted only to a single mode of vibration of the structure.

Equivalent static lateral force analysis is based on the following assumptions,

1) Assume that structure is rigid.

2) Assume perfect fixity between structure and foundation.

3) During ground motion every point on the structure experience same accelerations

4)Dominant effect of earthquake is equivalent to horizontal force of varying magnitude over the height.

5) Approximately determines the total horizontal force (Base shear) on the structure.

Earthquake Lateral Force Analysis

- The design lateral force shall first be computed for the building as a whole.
- This design lateral force shall then be distributed to the various floor levels.
- There are two commonly used procedures for seismic design lateral forces:

1. Equivalent static force analysis

2. Dynamic analysis

Dynamic Analysis

Dynamic analysis is classified into two types, namely,

- *Response spectrum method and Time history method*

***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 Clause 6.4.2 or by a site specific design, spectrum mentioned in Clause 6.4.6 of IS 1893 (2002)

When dynamic analysis is carried out either by the Time History Method or by the Response Spectrum Method, the design base shear computed from dynamic analysis (V_B) shall be compared with a base shear calculated using a fundamental period T_a ($\bar{V}_B$), where T_a is as per Clause 7.6. If base shear obtained from dynamic analysis (V_B) is less than base shear computed from equivalent static load method ($\bar{V}_B$ *i.e., using* T_a *as per Clause 7.6*), then as per Clause 7.8.2, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by ratio $\frac{\bar{V}_B}{V_B}$.

Review of Indian seismic code, IS 1893 (Part 1) : 2002

Sudhir K Jain

The Indian seismic code IS 1893 has now been split into a number of parts and the first part containing general provisions and those pertaining to buildings has been released in 2002. There has been a gap of 18 years since the previous edition in 1984. Considering the advancements in understanding of earthquake-resistant design during these years, the new edition is a major upgradation of the previous version. This paper reviews the new code; it contains a discussion on Clauses that are confusing or vague and need clarifications immediately. The typographical and editorial errors are pointed out. Suggestions are also included for next revision of the code.

With rapid strides in earthquake engineering in the last several decades, the seismic codes are becoming increasingly sophisticated. The first Indian seismic code (IS 1893) was published in 1962 and it has since been revised in 1966, 1970, 1975 and 1984[1]. More recently, it was decided to split this code into a number of parts, and Part 1 of the code containing general provisions (applicable to all structures) and specific provisions for buildings has been published[2].

Considerable advances have occurred in the knowledge related to earthquake resistant design of structures during the 18 years interval between the two editions of the code[3]. Some of these new developments have been incorporated in the 2002 version of the code, while many others have been left out so that the implementation of the code does not become too tedious for Indian professional engineers. For example, in the United States, the codes are revised every three years, and hence, a typical building code in the United States has acquired sophistication gradually over about six revisions during these 18 years. Since the Indian code has had to make a quantum jump with respect to many of the provisions, it still requires considerable effort for an average professional engineer to fully appreciate the new code and to be able to implement it correctly.

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

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

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

It is not uncommon to have errors or omissions in the codes. However, it is important to quickly correct these errors or omissions. This paper reviews the code and the suggestions for changes in the next revision are listed. Also listed are Clauses that are confusing or vague and need clarifications immediately. Finally, the typographical and editorial errors are pointed out.

Philosophical changes in the new code

As compared to the previous version, several major modifications have been incorporated in the new code. Some of the important modifications include the following.

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

high value of natural period and thus low seismic force.

Comments and suggestions

Other parts of IS 1893 (Foreword, page 2)

The fourth revision of the code (IS 1893 : 1984) covered buildings, water tanks, stacklike structures, bridges, dams and retaining walls. When carrying out the fifth revision, it was decided to split the code into five parts.

Part 1: General provisions and buildings

Part 2: Liquid retaining tanks – elevated and ground supported

Part 3: Bridges and retaining walls

Part 4: Industrial structures including stack like structures

Part 5: Dams and embankments

While Part 1 of the code has been released, the other parts are still in various stages of development. To address the situation that other parts of the code are not yet released, Note on page 2 of the code states:

"Pending finalization of Parts 2 to 5 of IS 1893, provisions of Part 1 will be read along with the relevant Clauses of IS 1893 : 1984 for structures other than buildings".

This is problematic in many situations, for instance, let us consider the case of overhead water tanks. In the 1984 code seismic design force for water tanks depends on parameters β, I, and α_0. In the new code, terms β and α_0 do not exist. Instead, one now needs the response reduction factor (R) for water tanks which is supposed to be provided in Part 2 of code. Clearly, there is no way one can combine Part 1 of the 2002 version with the 1984 code for water tanks and there is bound to be dispute and possibly litigation in case of lump sum contracts for water tanks. Similarly, it is not possible to implement the new provisions for bridges, stacks and dams. However, the problem is most serious for water tanks: bridges are designed as per the provisions of IRC or Indian

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Railway codes, and the design of industrial structures is usually done by established structural consultants.

To obviate this situation, the process for finalisation of remaining parts of the code must be completed at the earliest. In the meanwhile, a *model code* is urgently needed for the water tanks which could be adopted by the government departments for their contracts.

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Risk level

Para 5 on page 3 of the code states

"The seismic hazard level with respect to ZPA at 50 percent risk level and 100 years service life goes on progressively increasing ...".

This statement is made in the context of earthquake geology of the country. However, it may give a false impression that the values of ZPA (denoted by Z) given in the code are for 50 percent risk level and 100 years service life. Such a confusion needs to be avoided by modifying this statement as "The seismic hazard level goes on progressively increasing....".

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Fig 1 *(a)* All floor definition of centre of rigidity *(b)* Single floor definition of centre of rigidity

IS 13920-1993

- Applicable for structures located in
  - Zones IV and V
  - Zone III and $I > 1$
  - Zone III and is an industrial structure
  - Zone III and more than five stories

Critical zones in R.C. Frames

Where plastic hinge can form and requires proper confinement:

- Ends of beams upto length of 2d
  - Large negative moments and shears
- Moment reversal is possible
- Ends of columns
  - – about 1/6 of the clear height
- Beam column joints
  - Reversible local shear
  - Causes diagonal cracking

Detailing of Beams

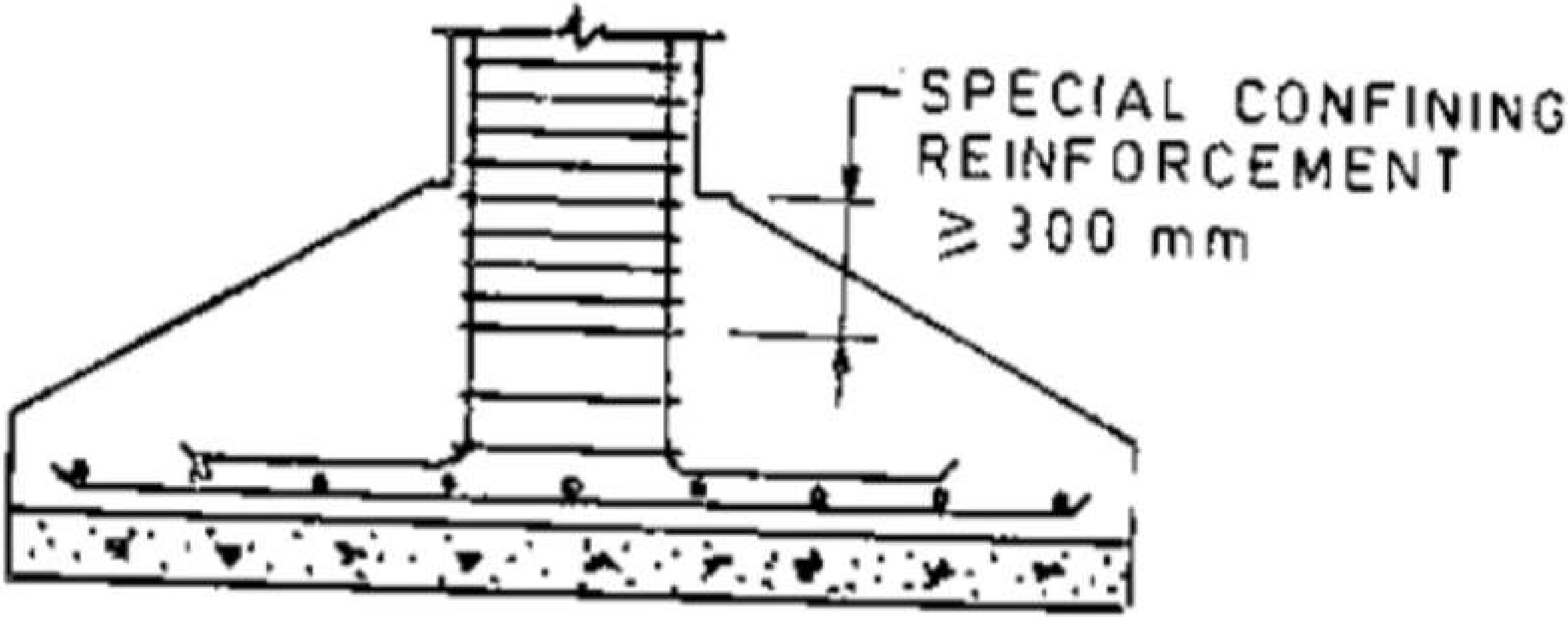
- Member size proportions
  - **Web width $\geq$ 200mm –**
    - For proper detailing and confinement
  - **Overall depth D $\leq$ 0.25 of clear span**
- Longitudinal reinforcement
  - **Minimum longitudinal steel = 0.24 ($\sqrt{fck}$)/fy**
    - Equals .00259 for M20 and F415
  - **Maximum long steel on any face, 0.025**

Columns

- Minimum dimension not less than 200mm
- - do - not less than 300mm

  for span > 5m or height > 4m
- Footing stirrup shall continue 300mm into footing

Fig 10 Provision of Special Confining Reinforcement in Footing



Conclusions

- India has a well developed code
- Problem lies in compliance
- Introduce earthquake engineering in curriculum
- Update knowledge
- Registration of engineers