GOVERNMENT OF INDIA

SEISMIC SAFETY OF NUCLEAR POWER PLANTS

A MONOGRAPH

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1.0 PREAMBLE

Earthquakes have been one of the deadliest hazards to human civilization till date. Unlike hazard such as cyclone, earthquakes cannot be predicted with the short-term accuracy required for effective emergency management. Large earthquakes capable of causing significant impact on human life have a low probability of occurrence. However, once an earthquake has occurred, there is very little time for warning and action. The effect could be catastrophic.

Current seismic design methodology for conventional facilities like residential/office buildings, bridges etc. is based on the philosophy of resisting minor earthquakes without significant structural damage, moderate earthquakes with limited structural damage and a major earthquake without collapse or loss of life. Indian national standard, “Criteria for Earthquake Resistant Design of Structures”, IS 1893[1] divides the country into several seismic zones and specifies the maximum possible earthquake in each zone. It is recognized that forces, which the structure would be subjected during an earthquake, will be larger than those specified in the standard. At the same time, structures possess lot of reserve capacity that is not considered in the process of design. The experience of past earthquakes clearly indicates that if structures are engineered properly, following codes/standards in spirit and letter, effect of this devastating hazard can be reduced to the level of acceptable risk.

Seismic design requirements of an NPP are quite stringent than those for conventional structures. An NPP is generally designed for two levels of earthquake, namely the S1 level earthquake or Operating Basis Earthquake (OBE), and the S2 level earthquake or Safe Shutdown Earthquake (SSE) [2]. The OBE level earthquake corresponds to that level of earthquake which is expected to occur once during life of the plant. The SSE corresponds to the credible maximum seismic event expected at the site and is determined considering the local geology and seismology and specific characteristics of local sub-surface material. The structures systems and components (SSC) of the nuclear power plant required for safe shutdown of reactor, decay heat removal and maintaining the safe shutdown condition, are designed to remain functional during SSE.

Seismic safety of a nuclear facility is ensured not just by design for two levels of earthquake. There are various other design aspects that go into engineering to ensure seismic safety of an NPP. Approach of this engineering is different from that of conventional facilities. This monograph presents the general profile of earthquake engineering, philosophy and methodology adopted for seismic safety of both new and existing Indian NPP.

2.0 EARTHQUAKE AND ITS EFFECTS

2.1 Introduction [3, 4]

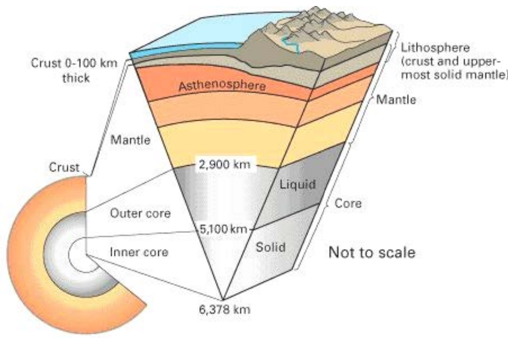


Figure 1 - Structure of earth (Source: Wikipedia Commons)

Earth is a layered planet as depicted in figure - 1. Its outermost layer, crust, varies to about 100 kilometers in depth, is made of rock and brittle in nature. Below it, is the mantle that extends from base of the crust up to a depth of nearly 2900 kilometers, beyond which the core begins. The core is mostly iron and nickel and consists of a liquid outer core and a solid inner core. Further the crust and the uppermost mantle up to a depth of 75 to 125 kilometers is called lithosphere. The lithosphere floats on the hot, plastic asthenosphere, which extends to 350 kilometers in depth. The entire layer of rock below the asthenosphere up to the core is called the mesosphere.

The theory of plate tectonics, presented in early 1960s [3], explains that the lithosphere is broken into seven large (and several smaller) segments called plates, figure - 2. The plates float on the layer below, the asthenosphere. As plate glides over the asthenosphere, the continents and oceans move with it. Most of the Earth's major geological activity occurs at plate boundaries, the zones where plates meet and interact.

The plates move slowly, at rates ranging from less than 1 to about 16 centimeters per year. Because the plates move in different directions, they knock against their neighbors at boundaries. The great forces thus generated at plate boundary build mountain ranges, cause volcanic eruptions and earthquakes. These processes and events are called tectonic activity. The earthquake that occurs at 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 intraplate earthquakes. The recurrence time for an intraplate earthquake is much longer than that of inter plate earthquakes.

2.2 Faults [5, 6]

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. A joint in rock mass is another tectonic feature similar to a fault. It is also a discontinuity in rock, except that in a joint, rocks on either side of the discontinuity have not moved. Lineaments are mappable linear surface features and may reflect subsurface phenomena. A lineament could be a fault, a joint or any other linear geological phenomena. Most faults produce repeated displacements over geologic time. Movement along a fault may be gradual, or sometime sudden generating an earthquake.

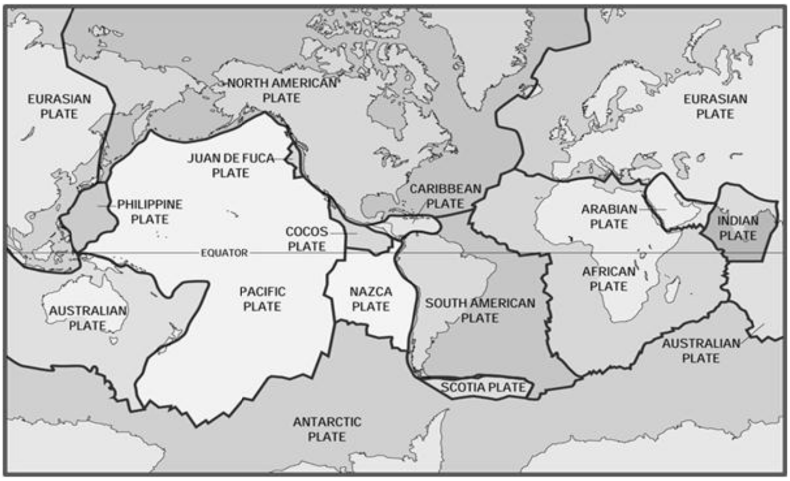


Figure 2 - Tectonic plate map of the world (Source: U. S. Geological Survey)

There are two important parameters associated with describing faults, namely, dip and strike, figure - 3. 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. Other terminologies of fault are shown in figure – 4. 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.

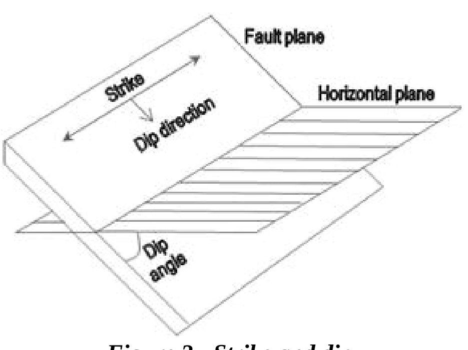


Figure 3 - Strike and dip

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. In a dip-slip fault, the slippage occurred along the dip of the fault, figure – 4(a) and (b). In case of a strike-slip fault, the movement, figure – 5 taken place along the strike, figure – 4(c) movement occurs diagonally across the fault plane in case of an oblique slip fault, figure – 4(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 – 4(a). In a reverse fault, the hanging wall has been displaced upward relative to the footwall, figure – 4(b). In a wrench fault, the foot or the hanging wall do not move up or down in relation to one another, figure - 4(c). Thrust faults, which are a subdivision of reverse faults, tend to cause severe earthquakes.

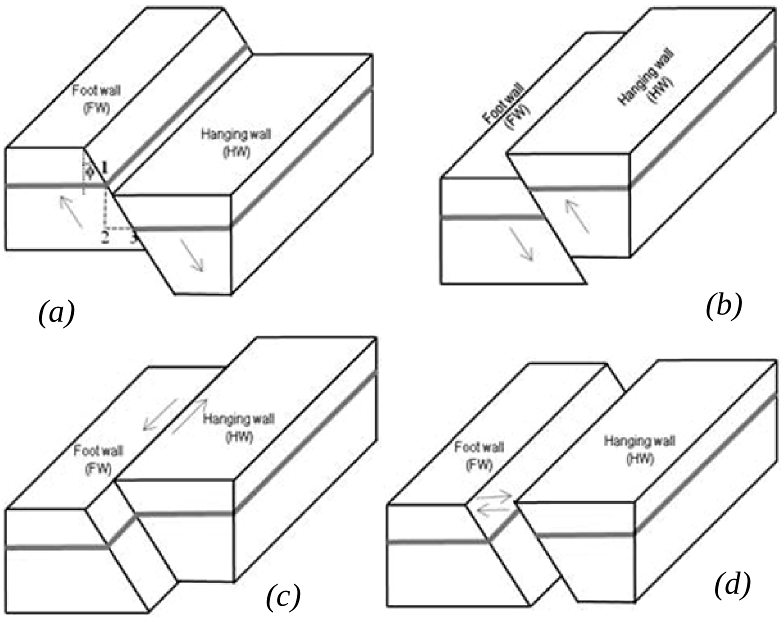


Figure 4 - Types of faults (Arrow shows direction of relative displacement)

(a) Normal fault; (b) Reverse fault;
(c) Strike-slip fault; (d) Oblique fault
(1-2)– Throw; (2-3) – Heave; ϕ - Angle of hade

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.

2.3 How do earthquakes occur?

Occurrence of earthquakes is explained generally by the elastic rebound theory. The two sides of an active fault are in slow but continuous movement relative to one another. This motion is accompanied by the gradual buildup of elastic strain energy within the rock along the fault. When the strain along fault

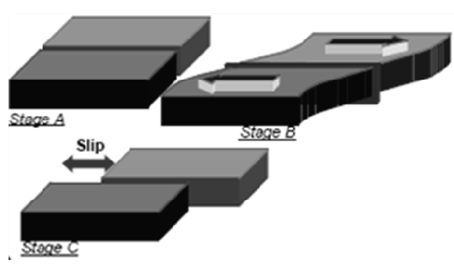


Figure 5 - Elastic strain buildup and strain rupture (Source: IITK BMTPC EQ Tips – 01)

exceeds threshold limit of the rocks, the fault ruptures. The rupture of the fault results in the sudden release of the strain energy that had been built up over the years. Outcome of this sudden release of energy is seismic waves. This is elastic rebound theory of generating earthquake. The theory is described pictorially in figure – 5.

Events associated with fault rupture are generally termed as seismic activities. The causes of fault rupture could be human induced or of natural origin. One of the example of seismic activity, which is due to human causes is reservoir-induced-seismicity (RIS). Koyna earthquake of 1967 in Maharashtra is an RIS.

2.4 Earthquake terminology [7]

The region on fault, where rupture takes place, is the focus or hypocenter of an earthquake, figure - 6. 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, as given in Table - 1. Most of the damaging earthquakes are shallow focus earthquakes.

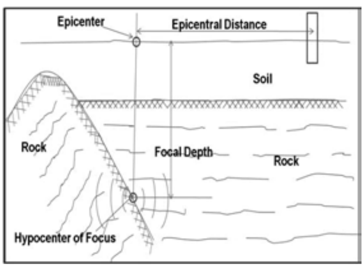


Figure 6 - Earthquake terminology [7]

Table – 1: Classification of earthquakes

Classification	Focal depth
Shallow focus	0 – 70 km
Intermediate focus	70 – 300 km
Deep focus	300 – 700 km

2.5 Seismic Waves [W1, W2]

The rupture of rock, along fault, during an earthquake, releases large quantum of energy, which propagates in the form of seismic waves. These waves travel outward from the focus of the earthquake. The waves travel along the surface of earth as well as through the interior of earth at varying speeds, depending on the material through which they move. Seismic waves are categorized in two types, based on their path of travel as body waves and surface waves, figure - 7.

Body waves travel through the interior of the Earth. Body waves transmit the first-arriving tremors of an earthquake, as well as many later arrivals. There are two kinds of body waves: primary and secondary.

Primary waves (also known as P-waves) are longitudinal or compressive waves, which mean that the ground is alternately compressed and dilated in the direction of propagation. P-waves are the fastest waves and the first to arrive at a station after an earthquake.

Secondary waves (S-waves) are transverse or shear waves, which mean that the ground is displaced perpendicularly to the direction of propagation. Shear waves can travel only through solids, as fluids cannot withstand shear stresses. Secondary waves or shear waves are several times larger in amplitude than the primary waves generated from an earthquake focus.

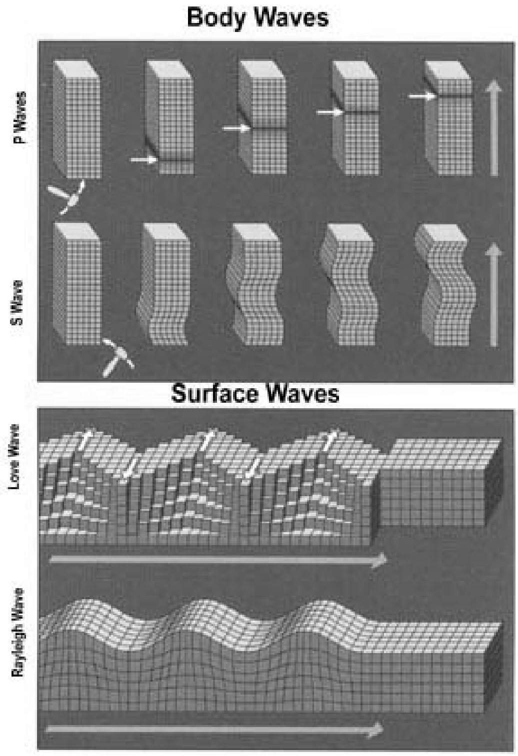


Figure 7 - Seismic waves [W2]

Surface waves travel through the earth crust and are of a lower frequency than body waves. The damage and destruction associated with earthquakes can be mainly attributed to surface waves. This damage potential and the strength of the surface waves reduce with increase in depth of earthquakes. There are two kinds of surface waves: Love waves and Rayleigh waves, Love waves are confined to the surface of the crust and produce entirely horizontal motion. Rayleigh waves make the particles oscillate in an elliptical motion. Most of the shaking felt from an earthquake is due to the Rayleigh waves.

The vibratory ground motion characterizing an earthquake is caused by the passage of seismic waves. Vibratory motion may repeat itself regularly, as in the balance wheel of a watch; or display considerable irregularity, as in earthquakes. When the vibratory motion is repeated in equal intervals of time, it is called periodic motion. The repetition time, T, is called the period of the vibration, and its reciprocal, 1/T, is called the frequency of vibration.

2.6 Recording earthquakes: [8]

Seismic waves are detected and recorded by instruments by measuring the movements of the ground due to earthquakes. Some instruments measure the ground displacements and are called seismographs, figure – 8. The record obtained from a seismograph is called a seismogram. To measure the ground accelerations, other type of device exist, called accelerographs. The accelerographs register the accelerations of the soil and the record obtained is called an accelerogram.

The seismograph has three components – the sensor, the recorder and the timer. The principle on which it works is simple and is explicitly reflected in the early seismograph– a pen attached at the tip of an oscillating simple pendulum (a mass hung by a string from a support) marks on a chart paper that is held on a drum rotating at a constant speed. A magnet around the string provides required damping to control the amplitude of oscillations. The pendulum mass, string, magnet and support together constitute the sensor; the drum, pen and chart paper constitutes the recorder; and the motor that rotates the drum at constant speed forms the timer.

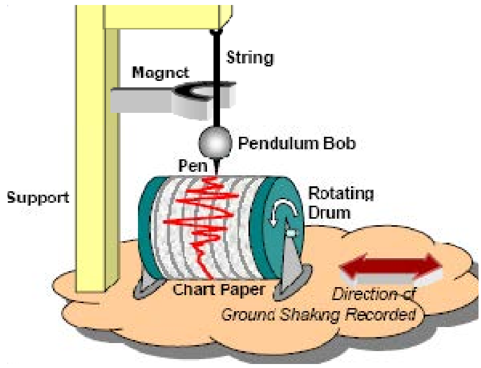


Figure 8 - Schematic of a seismograph
[Source: IIT-K BMTPC Eq tips – 02]

2.7 Measuring Earthquakes

The "size" of earthquake is generally measured by its magnitude and intensity. Magnitude measures the energy released at the source of the earthquake and is determined from the record of seismographs.
Intensity is a measure of the severity of shaking produced by the earthquake at a given location. Intensity is generally higher near the epicenter than far away. For an earthquake of certain magnitude, different locations experience different levels of intensity. Lines drawn on a map connecting points of equal intensity is known as isoseismals. figure – 9 shows the isoseismals for Bhuj earthquake of 2001.

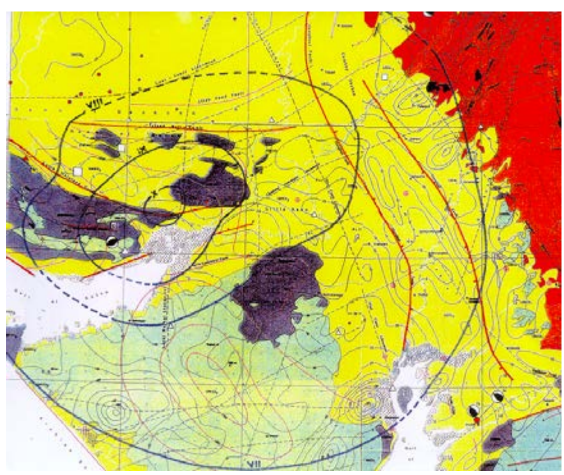


Figure 9 – Isoseismals for Bhuj earthquake 2001
[Source: www.gsi.gov.in (W3)]

2.7.1 Magnitude scales [9, 10]

There exists, a number of scales to represent earthquake magnitude; most common is the Richter scale. Richter magnitude, also known as local magnitude (M_L) is defined as the base-ten logarithm of the maximum ground motion amplitude (in millimeters) recorded on a Wood-Anderson short-period seismometer, located at a distance of one hundred kilometers from the earthquake epicenter. Richter magnitude scale is suitable for representing earthquake magnitude below 6.8 - 7. Earthquake magnitude is also represented in terms of body wave magnitudes (M_B) and surface wave magnitudes (M_S); they can measure earthquakes up to a magnitude of 8.5. The magnitude scales without saturation level are moment magnitude (M_W) and energy magnitude (M_E). The moment magnitude scale is a way of rating the seismic moment (estimate of the energy of an earthquake) of an earthquake with a simple, logarithmic numerical scale. The comparison of various scales is shown in the figure – 10 [10].

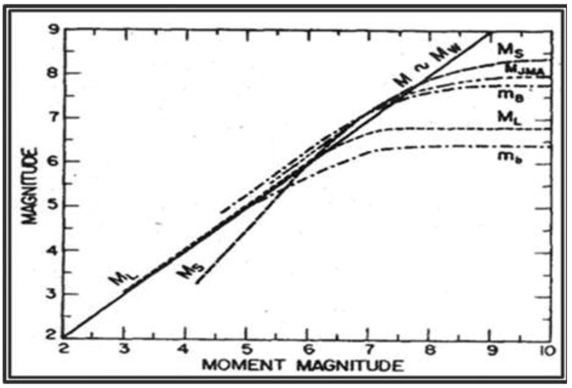


Figure 10 - Comparison of magnitude scales [10]

2.7.2 Intensity scales: [1, 9, W4]

The intensity scale consists of a series of certain key responses such as people awakening, movement of furniture, damage to chimneys, and finally - total destruction. 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. Another intensity scale is Comprehensive Intensity Scale (MSK 64). This scale is more comprehensive and describes the intensity of earthquake more precisely. Indian seismic zones were categorized on the basis of MSK 64 scale. Intensity and magnitude are correlated. Table – 2 compares, MMI and MSK intensity scales along with corresponding magnitudes and Indian seismic zones as per IS 1893.

Table - 2: Comparison between Intensity scales, magnitude and IS 1893 Seismic zone [Source: www.riskfrontiers.com(W4)]

MMI	MMI Scale description	MSK	MSK Scale description	Magnitude	Seismic zone IS 1893
I	Felt by almost no one.	1	Not Noticeable	2	II
II	Felt by very few persons at rest.	2	Scarcely Noticeable		
III	Felt quite noticeably by persons indoors. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck.	3	Weak, Partially observed	3	
IV	Felt indoors by many, outdoors by few. Sensation like heavy truck striking building.	4	Largely Observed	4	
V	Felt by nearly everyone; Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.	5	Awakening, Slight damage in clay buildings, Sometimes changes in flow springs		
VI	Felt by all, many. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.	6	Frightening, Felt by most indoors and outdoors, Fine cracks in plaster of ordinary brick buildings, In some cases cracks of width 1cm.	5	
VII	Damage negligible in buildings of good design and construction; considerable damage in poorly built, designed structures;	7	Damage in plaster of reinforced building		III
VIII	Damage slight in specially designed structures; considerable damage in ordinary buildings with partial collapse. Fall of chimneys, stacks, walls.	8	Destruction of building, Most buildings have small or deeper cracks	6	IV
IX	Damage considerable in specially designed structures; frame structures thrown out of plumb. Buildings shifted off foundations.	9	General damage of building, Most buildings have gap in wall, Ground cracks up to 10cm	7	V
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed. Rails bent.	10	General destruction of building, part of most of buildings were fallen		
XI	Few, (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.	11	Destruction, Severe damage to well built buildings, bridges, water dam and railway line.	8 and above	
XII	Damage total. Objects thrown into the air.	12	Landscape change, practically all structures above and below ground were damaged, surface of ground radically changed		

2.8 Earthquake Catalogues [11, 12]

Earthquake catalogues archive the information of past earthquakes. There are no established standards on the specific contents of the earthquake catalogues. General information that is archived for includes earthquake hypocenters, origin times, recorded travel times.etc Earthquake catalogues with high-quality data over large areas and long time periods are rare. The most common drawback in generation of regional and global earthquake catalogues has been that only few of them are based on original sources of information and rest relies on secondary evidence and a repetition of previous lists. Geological Survey of India (GSI) had compiled the catalogue of Indian earthquakes and contributed very substantially to the earthquake studies in India. AERB published a comprehensive earthquake catalogue containing earthquake data down to magnitude 3.0 for peninsular India in the year 1993 [12]. This comprehensive and enlarged database is useful to assess earthquake parameters for design of nuclear power plants and also of other critical structures.

2.9 Major Indian Earthquakes [13]

India has been subjected to some of the worst earthquakes in the past. Around the world, on an average about one earthquake of magnitude greater than 8.0 takes place every year as against about 96 incidents per year of magnitude range 6.0 to 8.0. Some of the greatest earthquakes of the world occurred in India. Table – 3 summarizes the major earthquakes which hit India over the years.

Table – 3: Major earthquakes in India

Earthquake	Year	Magnitude	Epicenter	
			Latitude	Longitude
Kutch earthquake	1819	8.3	23.60 N	69.60 E
Assam earthquake	1897	8.7	25.50 N	91.00 E
Bihar-Nepal earthquake	1934	8.4	26.21 N	86.21 E
Assam-Tibet earthquake	1950	8.7	24.60 N	92.94 E
Koyna Earthquake	1967	6.5	17.38 N	73.75 E
Uttarkashi earthquake	1991	6.6	30.78 N	78.77 E
Killari (Latur) earthquake	1993	6.4	18.07 N	76.62 E
Jabalpur earthquake	1997	6.0	23.08 N	80.09 E
Chamoli Earthquake	1999	6.8	30.11 N	79.35 E
Bhuj (Gujarat) earthquake	2001	7.6	23.44 N	70.31 E
Andaman Earthquake	2002	6.5	13.01 N	93.14 E
Kashmir Earthquake	2005	7.6	34.49 N	73.63 E
Sikkim Earthquake	2006	5.7	27.37 N	88.36 E

2.10 Effects of earthquakes

Main effect of an earthquake is vibration (ground excitation); there are other effects that are also important, and have to be addressed appropriately to ensure safety from earthquakes. In general, the major effects of earthquake are

- Vibration of structures
- Surface faulting
- Ground failure
  - Landslides
  - Subsidence
  - Liquefaction
- Water Waves
  - Tsunamis
  - Seiches

2.10.1 Vibration of structures: [14]

The hazard due to vibration commences when the ground motion interacts with natural and man-made structures. The resulting vibration induced loading effects in the structure can lead to various degrees of damage or to complete collapse. Figures – 11 depict the structural failures caused during the 2001 Bhuj earthquake. Effects of vibration are generally mitigated by engineering measures.

Figure 11 - Structural failures during Bhuj earthquake 2001
[Source: www.nicee.org (W6)]

2.10.2 Surface Faulting [15]

Large earthquakes generally produce a series of permanent effects on the ground consisting of scarps (steep slope), fractures, hollows and depressions that are expressions of slippage and permanent deformation occurring on seismogenic faults. Such features are known as surface or earthquake faults. Ground rupture is typically associated with

Figure 12 - Surface faulting

shallow earthquake magnitudes 6.2 and above . Such rupture can reach overall lengths of up to some kilometers and offsets up to several meters. Figure-12 shows an example of surface faulting. However, occasionally earthquake of lower magnitude can cover surface faulting occurs owing to local and unusual tectonic conditions.

2.10.3 Subsidence and uplift

Subsidence is the downward displacement of ground surface. The opposite of subsidence is uplift, which results in an increase in elevation of the surface. Subsidence can be caused by various phenomena, one of them being earthquake. Horizontal motions induced by shocks cause compaction as long as the cycles are relatively small. Vertical accelerations in excess of 1.0g are generally required to cause significant densification of sands. Figure- 13 shows adverse effect of subsidence.

Figure 13 - Subsided road [Source: Wikipedia commons (W5)]

2.10.4 Landslides

A landslide is a geological phenomenon which involves a wide range of ground movement, such as rock falls, deep failure of slopes and shallow debris flows that can occur in offshore, coastal and onshore environments. Earthquakes can induce landslide. A sudden shock, from an earthquake, can alter the configuration of a slope, causing the slipping of surface soil and rock and the collapse of cliffs.

2.10.5 Liquefaction

Liquefaction is a phenomenon which occurs primarily in the location having loosely deposited sands and silts with high ground water levels. The vibration caused by earthquake induced higher pore pressure (pressure exerted by ground water on surrounding soil particle). When the pore pressure crosses the strength of soil mass,

Figure 14 - Adverse effects of liquefaction during Nigata earthquake 1964 [Source: Wikipedia(W1)]

disintegration of soil mass took place. This phenomenon is known as liquefaction. Destructive effects of liquefaction can take many forms like flow failures of soil mass, lateral spreads, ground oscillation, loss of bearing strength, settlement etc. Figure – 14 depicts the adverse effect of liquefaction during the Nigatta earthquake, 1964 in Japan.

2.10.6 Tsunami

A tsunami is a series of sea waves created by displacement of sea bed caused by earthquake, volcanic eruption, man-made underground explosions, submarine landslides, hitting of meteoric objects etc. It is a class of long sea wave, which can reach great height when encountering shorelines, figure – 15. Earthquakes are often the cause of tsunami. An earthquake occurring near a body of water may generate a tsunami if (i) it occurs at shallow depth, (ii) it is of moderate or high magnitude, (iii) the fault rupture causes vertical movement of rock (sea bed) along the fault line, and (iv) water volume and depth is sufficient.

Figure 15 - Schematic of a tsunami [Source: Wikipedia]

If the first part of a tsunami to reach land is a trough (draw back) rather than a crest of the wave, the water along the shoreline may recede dramatically, exposing areas that are normally submerged. This can serve as an advance warning of the approaching tsunami which will rush in faster than it is possible to run.

2.10.7 Seiches

A seiche is similar to tsunami, but occurs in an enclosed or partially enclosed body of water, figure - 16. Seiches and seiche-related phenomena are observed in lakes, reservoirs and bays. It is caused by earthquakes, landslides and other non-seismic events like underwater volcanic eruption, underground man made explosion, meterological disturbances such as storms etc.

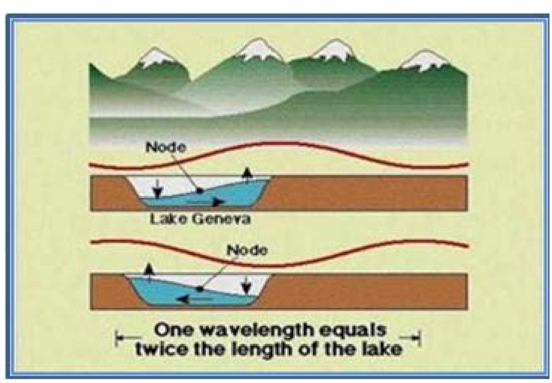


Figure 16 - Seiches [Source: U S. Geological Survey]

3.0 ASEISMIC DESIGN OF STRUCTURES

3.1 Introduction [8]

Earthquake causes ground motion, and a building founded on the ground experiences this motion at its base. However, the roof of the building has a tendency to remain at its original position. This tendency to remain in the original position is known as inertia. When ground experiences sudden motion, the roof of structure relatively moves backwards, as if a force is applied backwards, as shown in figure – 17. This force is called as inertia force. If the roof has a mass ‘*m*’ and acceleration of relative movement is ‘*a*’, then inertia force ‘*f*’ is mass times acceleration, *m* x *a*.

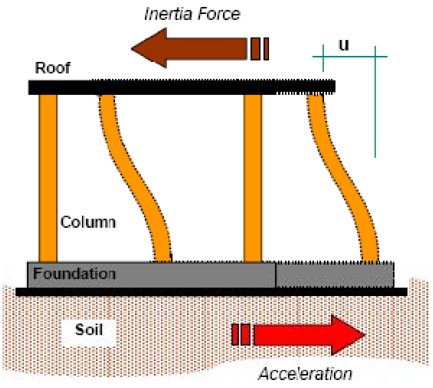


Figure 17 - Effect of inertia on a building when shaken at its base [Source: IIT K BMTPC eq tips - 5]

Under horizontal shaking of the ground, horizontal inertia forces are developed at floor level of a building, which are transferred to the foundation through slab, column and finally to the soil under foundation as shown in figure - 18. Each of these structural elements (i.e. floor slab, beam, column, wall and foundation) and connection between them must be so designed that they, as a structural system, can transfer the horizontal inertia forces safely to foundation. Walls or columns are the most critical elements in transferring the horizontal inertia force.

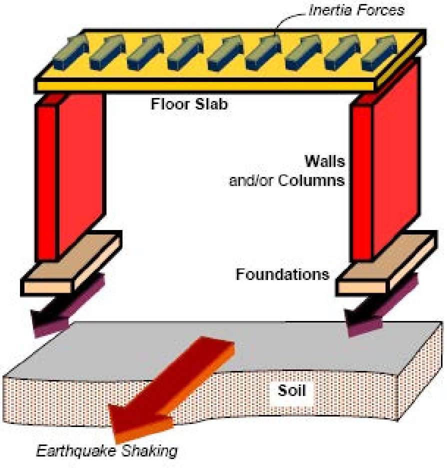


Figure 18 - Flow of seismic inertia force through structural components [Source: IIT K BMTPC eq tips - 5]

Objective of earthquake resistant design or aseismic design structure is to transfer the inertial force, caused by earthquake, safely to the foundation without causing undersigned damage to the structure.

3.2 Aseismic design philosophy [8]

Severity of ground shaking can be minor, moderate or strong. Table – 4 lists the grouping of earthquakes depending upon earthquake magnitude as minor, moderate and strong. Aseismic design philosophy is formulated based on the fact that minor shaking occurs frequently; moderate shaking occasionally and strong shaking rarely. Engineering of the structures is so performed that they resist minor shaking without any damage to load bearing members, moderate shaking with limited damage and strong shaking with acceptable damage but without collapse. Figure – 19 explains these performance objectives on the basis of reinforced cement concrete (RCC) framed structure with brick infill walls. RCC frame members, in figure – 19, are the load bearing members and brick infill the non-load bearing member.

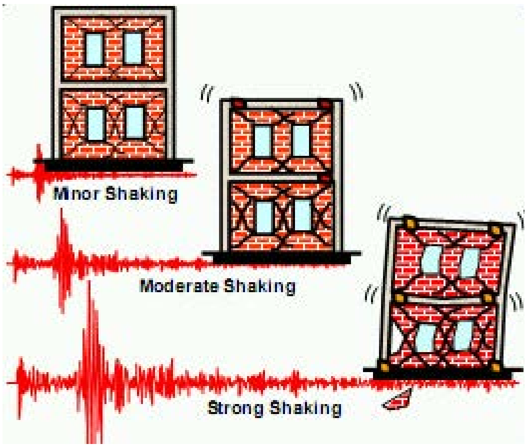


Figure 19 - Performance objectives under different intensities of earthquake shaking [Source: IIT K BMTPC eq tips - 8]

Four important steps for earthquake resistant design of structures are i) determination of loading effects of earthquake, ii) planning and design of structural configuration, iii) structural response analysis to determine forces induced in the elements like beams, columns, and iv) determination of cross sectional parameters to withstand the induced forces by adopting suitable design approach.

Table - 4: Classification of Earthquakes based on Magnitude

Classification	Magnitude
Minor	3 – 3.9
Light	4 – 4.9
Moderate	5 – 5.9
Strong	6 – 6.9

3.3 Loading effects of earthquake

Earthquakes motion can be recorded in terms of ground displacement, velocity or acceleration. Figure – 20 depicts the record of ground acceleration, velocity and displacement for an earthquake recorded at an observatory. For the purpose of determining the loading effects of earthquake, generally records in terms of ground acceleration are preferred. Loading effect of earthquake ground motion at a site is generally represented by three ground motion (GM) parameters viz. peak ground acceleration, response spectrum and acceleration time history.

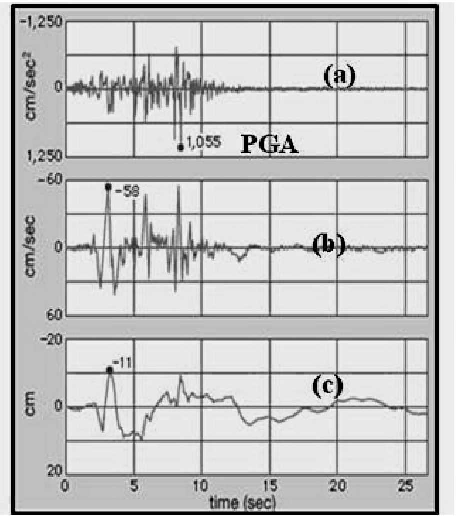


Figure 20 - Plot of (a) ground acceleration, (b) velocity and (c) displacement of an earthquake

Accelerogram gives the plot of earthquake ground acceleration with respect to time and is also known as acceleration time history. Acceleration time history provides information on ground motion characteristics that affect structural response viz. duration, frequency content and intensity. The amplitude (maximum ordinate) of this plot is known as the Peak Ground Acceleration (PGA). A typical acceleration time history and PGA is illustrated in figure – 21.

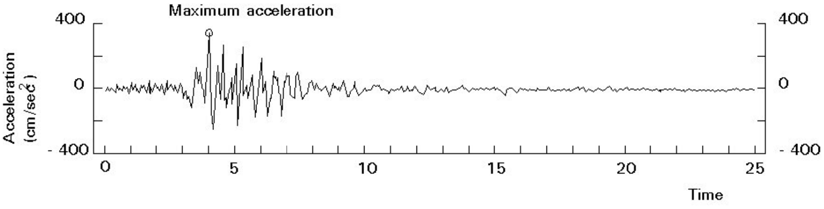


Figure 21 - Typical accelerogram

The combined influence of the amplitude of ground accelerations, their frequency contents and the duration of the ground shaking on different structures is represented by means of response spectrum. A plot showing the maximum response induced by ground motion in single degree of freedom oscillators of different fundamental periods having same damping is known as response spectrum, figure – 22.

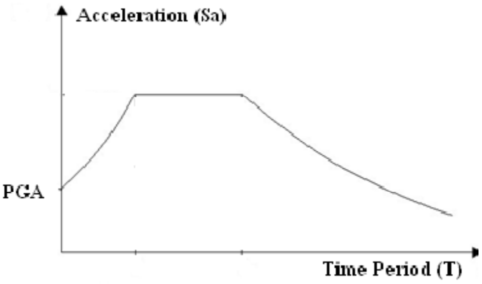


Figure 22 - Typical design response spectrum

The ground motion parameters i.e. PGA value, response spectrum and acceleration time history of a site, which are used in the design of structure are known as design ground motion parameters (DBGM). The DBGM parameters are specified at free field conditions. Ground motions that are not influenced by the presence of structures are referred to as free field motions.

3.4 Structural Configuration [9, 16]

During earthquake, the performance of structures depends upon magnitude of earthquake forces as well as layout and shape of the structure. Since the earthquake force is a function of mass, the layout and shape of building shall be such that it is as light as possible, and as symmetrical as achievable from the consideration of geometry, mass and stiffness distribution.

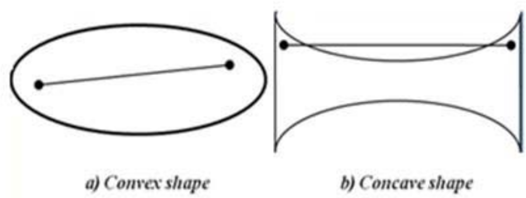


Figure 23 - Convex and concave shapes

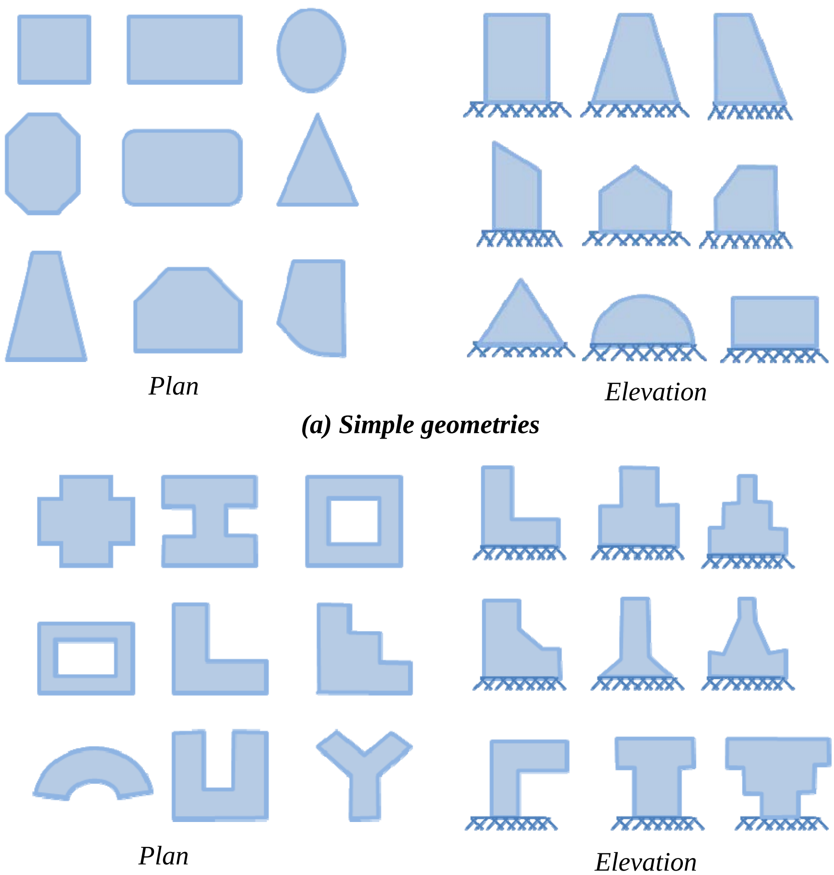
Building shapes are either convex or concave for the purpose of design in aseismic parlance. A convex shape is one where it is possible to join any two points within it by straight line without crossing the boundary. A concave shape is one, where a part of straight line may lie outside the shape, figure - 23. A building, convex in plan and elevation is considered as simple or regular building, figure – 24(a). If a building is concave in plan and elevation then it is considered as complex or irregular shape, figure – 24(b). Generally, buildings with simple geometry are less vulnerable to damage during strong earthquakes.

It is possible to split plans with complex geometries into simple geometries and thus make the structure more earthquake resistant. An example is breaking an L-shaped plan into two rectangular plan shapes using a separation joint at the junction. When two buildings are too close to each other, they may pound on each other during strong motion earthquakes. With increase in building height, this collision can be a greater problem. When building heights do not match, the roof of the shorter building may pound at the mid-height of the column of the taller one; this can be very dangerous.

3.5 Structural response analysis [17]

Responses of structure to an earthquake can be represented in terms of deflection, acceleration as well as induced forces such as bending moment, shear forces in the elements like beams and columns. This structural response is generally determined by analysis.

Determination of structural response for an earthquake is the subject of structural dynamics. Structural dynamics is a subset of structural analysis which covers the behavior of structures subjected to dynamic loading. A dynamic load changes with time while static load does not. Earthquake induces dynamic loading effects in the structures. There are two general classes of vibrations in structural dynamics- free and forced. Free vibration takes place when a system oscillates under the action of forces inherent in the system itself, and when external impressed forces are absent. Vibration that takes place under the excitation of external forces is called forced vibration. Vibratory effects during earthquakes fall under the category of forced vibration.



(a) Simple geometries

(b) Complex geometries

Figure 24 - Description of shapes of buildings [9]

All objects (including buildings and the ground) have a "natural period," or the time it takes to swing once back and forth, under free vibration. When a building and the ground sway or vibrate at the same rate, they are said to resonate. As the building and ground resonate, their vibrations are amplified or increased, and the building is subjected to higher earthquake forces.

Commonly used approach for seismic response analysis of structure for design is modal analysis. Response spectrum method and time history method are the two basic methods of modal analysis to determine structural response against earthquake excitation. For simple structures, equivalent static method of analysis may be used.

3.5.1 Analytical model:

Simulation of the structure to a mathematical model is an important step to analyze structural response against any load. In structural dynamics, the number of independent coordinates necessary to specify the configuration or position of a system at any time is referred to as the degrees of freedom. In general a structure may have infinite degrees of freedom. Idealization permits the reduction in the number of degrees of freedom to a discrete finite number.

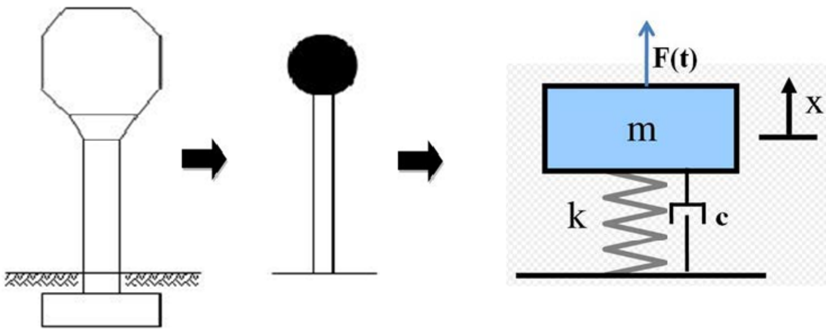


Figure 25 - Analytical model representing a water tank into a SDF system [Source: Wikipedia]

The basic idealization of a structure is single degree freedom (SDF) system. The conversion of a water tank to a SDF model is depicted in Figure – 25. It has a mass element (m) representing total mass and the structure (principal contributor – water); a spring element (k) representing the stiffness of the structure (generally the shaft); a damping element (c) representing frictional characteristics and energy losses of the structure; and an time dependent exciting force F(t) representing external forces acting on the structure. In case of earthquake, F(t) is derived from the PGA and response spectrum or acceleration time history of the vibratory ground motion.

For multi-degree freedom system (MDF), number of degree of freedom is more than one, figure-26. The mass of the structure is also distributed along the length and breadth of the structure. The mass of the structure is assumed concentrated at floor levels and subject to lateral displacement only. In figure-26, MDF system have structure mass lumped at nodal point with consideration of lateral displacement and rotational degree of freedom. To convert the mass degree of freedom to a discrete finite value, the masses are lumped at appropriate locations as shown in figure-26. In case of multi degree of freedom system, m, c and k are not single values but matrices.

The dynamic equilibrium equation of an SDF can be written as

$$m\ddot{x} + c\dot{x} + kx = F(t) \qquad \text{...... (3.1)}$$

$\ddot{x}, \dot{x}$ *and* x represent the acceleration, velocity and displacement, of the structure. In case of seismic excitation, the external force F(t) is replaced by $(-m\ddot{x}_g)$, where $\ddot{x}_g$ is the ground acceleration due to seismic excitation. Equation (3.1) is the equation of motion of a SDF structural system. For MDF system, number of equations of motion is same as that of mass degree of freedom of the system. Solution of equation of motion can be achieved by classical method or by numerical method depending on the nature of equation. Solution of equation (3.1) results in determination of displaced shape of structure, or displacement of the nodes having mass. For SDF system, number of nodal displacement is one, for MDF system it is more than one, and equal to number of mass degree of freedom.

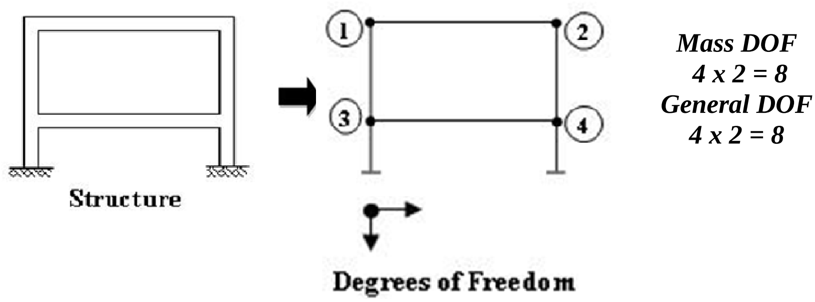


Figure 26 - Analytical model representing a MDF system

A structural system vibrates with particular frequencies depending on its stiffness and mass and these are referred to as natural frequencies of the system. These can be determined from the solution of the differential equation of 3.1 for free vibration condition, (F(t) = 0). Natural frequency, ω, for an undamped SDF system (c = 0) is given by

$$\omega = \sqrt{\frac{k}{m}} \qquad \text{...... (3.2)}$$

3.5.2 Modal analysis approach

Classical solution of the equilibrium equation of a MDF becomes difficult. Hence, we resort to either numerical techniques or a simplified method known as modal analysis. A mode of vibration is a characteristic pattern or shape in which a structural system vibrates. A structure with 'N' mass degree of freedom will have "N' modes of vibration as shown in figure – 27. Each mode will vibrate at a particular frequency called modal frequency and characteristic shape of a given mode is known as mode shape. The actual vibration of a structure under earthquake shaking is always a combination or mixture of different fundamental vibration modes.

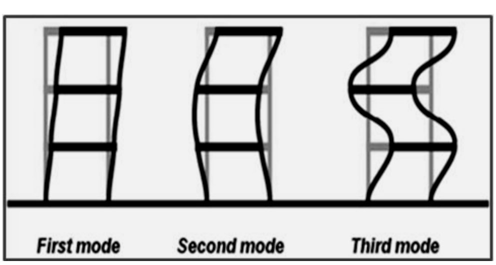


Figure 27 - Mode shapes of a three storey structure

Generally only a few modes are of interest for rational determination of structural response.

Modal analysis technique involves uncoupling an 'N' degree of freedom system to N-single degree of freedom systems. For those N single degree of freedom systems, the solution can be determined by classical methods. The system responses such as displacements and accelerations (forces) are obtained for each of the N-modes of vibration. The modal response is then combined to get the total response of the structure.

Methods of seismic response analysis provide the maximum response in each mode of vibration. The response parameters, such as the peak displacements, element stresses, element forces, and moments, are evaluated for each significant mode of vibration and then combined to obtain the total response of the structure. There are many rules available for combination of modal responses [18]. Widely used methods include square root of sum of squares (SRSS) and complete quadratic combination (CQC). The most common rule of modal combination is SRSS, which is based on assumption that the maximum values of response do not occur at the same time and is given by following expression.

$$R = \sqrt{\left(\sum_i R_i^{\ 2}\right)}$$

One of the exceptions of SRSS method arises when the responses are from modes with closely spaced frequencies. Other rules are to be used in such cases.

Generally modal analysis is performed separately for earthquakes in each of the orthogonal directions. It is necessary to combine further the responses from these three directions to obtain the total response of the structure. This is called spatial combination. SRSS method as well as 100:40:40 combination method is used for the spatial combination.

3.5.3 Response spectrum analysis method

Response spectrum method uses the seismic excitation represented in terms of a response spectrum as the exciting force. The free vibration analysis of the structure determines the natural frequencies of each mode (i.e. each single degree of freedom system). Peak response of the structure at each mode is determined from the response spectrum by knowing the natural frequency/time period corresponding to that mode. The responses are then combined using appropriate modal combination rule. This method is depicted in figure – 28 and is explained with illustrative example in Appendix - I.

3.5.4 Time History Analysis Method

In time history analysis method, the input motion is a prescribed function of time such as acceleration versus time, or displacement versus time. The analysis consists of a time integration of the equations of motion. Either the coupled equations of motion or the uncoupled equations of motion as is the case of modal analysis can be treated in this method. When uncoupled equations are treated the method is called modal superposition and when coupled equations are treated the method is called direct integration.

3.5.5 Equivalent Static Method

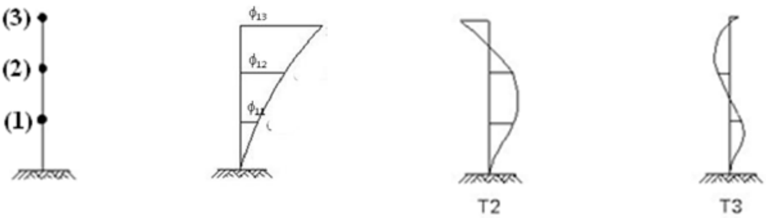
In equivalent static analysis, the total base shear is calculated as a product of horizontal seismic coefficient, and total weight of the structure. The value of horizontal seismic coefficient depends on the seismic zone, type of construction, foundation conditions and the importance of the structure. IS 1893, provides the methodology to calculate horizontal seismic-coefficients for different locations in India. The load generally has an inverted parabolic distribution along the height of the structure. A static analysis with these lateral loads yields the induced element forces.

3.5.6 Soil-structure interaction [19, 20, 21]

Civil engineering structures are founded on soil or rock. When a structure, founded on rock or stiff soil is subjected to an earthquake, the high stiffness of the rock maintaining the motion to be very close to the free-field motion. When a structure founded on soft soil is subjected to an earthquake, it interacts with the foundation and the soil, and thus changes the motion of the ground. Extent of this interaction depends on closeness of the stiffness of the structure and the foundation medium i.e. their relative stiffness. Soil-structure interaction broadly can be divided into two phenomena: a) kinematic interaction and b) inertial interaction. Earthquake ground motion causes soil displacement known as free-field motion. However, the foundation embedded into the soil will not follow the free field motion. This inability of the foundation to match the free field motion causes the kinematic interaction. On the other hand, the mass of the super-structure transmits the inertial force to the soil causing further deformation in the soil, which is termed as inertial interaction.

Soil structure interaction is addressed in dynamic analysis modelling by direct approach or impedance function approach. In direct approach, the foundation medium is represented as a finite element system and the earthquake input is defined as the base rock excitation. In impedance function approach, the earthquake input is specified by the free field ground motion at the soil structure interface. The soil system can be represented by means of finite elements or as a continuum – such as visco-elastic half space (semi infinite springs). If the foundation is rigid, the soil can be replaced by a set of equivalent springs and dashpots to represent the stiffness and damping characteristics of foundation. However, if the foundation is not rigid, then it becomes necessary to model the soil.

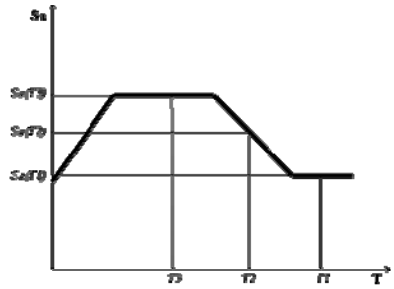
Step – 1: Computation of mode shapes and periods



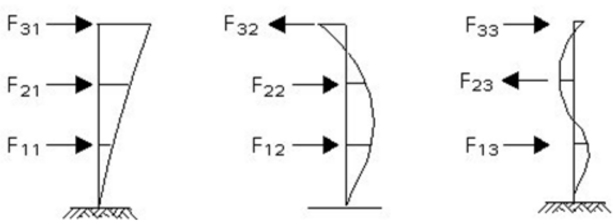
Model *Mode - 1* *Mode - 2* *Mode - 3*

Structural node numbers (1), (2) and (3) where masses are lumped.
T1, T2 and T3 are time periods of mode 1, 2 and 3 respectively.

Step – 2: Reading the response spectrum



Step – 3: Modal responses



Equivalent inertial force on i^{th} node due to j^{th} mode, $F_{ij} = Sa(T_j) . P_j . \phi_{ji} . m_i$
P_j is the participation factor of i^{th} node and ϕ_{ij} is the modal displacement of ith node for jth mode determined from modal analysis and m_i is the mass lumped at i^{th} node.

Step – 4: Determination of moment and shear in each mode
Example: Bending moment at column base

Step – 5: Combining responses from different modes
Example: SRSS method

$$Resulting\ Bending\ moment\ M = \sqrt{M_{1b}^2 + M_{2b}^2 + M_{3b}^2}$$

Figure 28 - Illustration of response spectrum method [Source: www.esdep.org(W7)]

3.5.7 Fluid-structure interaction [21, 22]

Fluid-structure interaction happens when structure like tanks, filled with fluid is subjected to earthquake excitation and results in sloshing. General approach is to separate the hydrodynamic pressures into impulsive and convective parts. The impulsive pressures are those associated with inertial forces produced by the accelerations of the walls of the container, and the pressures are directly proportional to the accelerations. The convective pressures are those produced by the oscillations of the fluid.

3.6 Design approach [8]

The structure is so designed that the elements are capable to withstand the forces induced by an earthquake. The induced forces are determined from structural response analysis. Primary requirement for an earthquake resistant structure is that their main structural elements are designed and constructed as ductile elements. This enables them to withstand earthquake effects with some damage, but without collapse. Earthquake-resistant design strives to predetermine the locations where damage takes place and then to provide good detailing at these locations to ensure ductile behavior of the structure.

Ductility is the property of certain materials to fail only after large deformations have occurred. Figure - 29 illustrates what is meant by ductility. Consider two bars of same length and cross sectional area - one made of a ductile material and another of a brittle material. When pulled, the ductile bar elongates by a large amount before it breaks, while the brittle bar breaks suddenly on reaching its maximum strength at a relatively small elongation. Amongst the materials used in building construction, steel is ductile, while masonry and concrete are brittle.

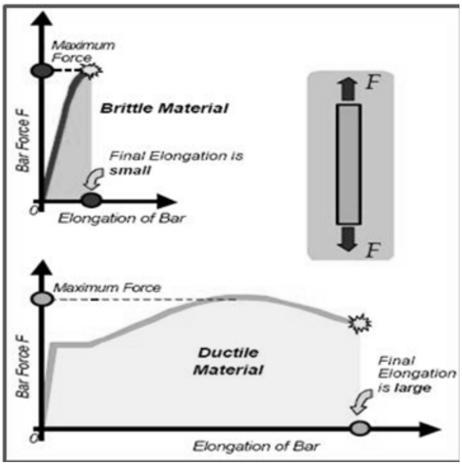


Figure 29 - Ductility

[Source: IIT K BMTPC eq tips - 9]

The building material most commonly used for construction of conventional structures is reinforced cement concrete (RCC), which is a composite material made of cement concrete and reinforcing steel. Concrete has a strong compression load carrying capacity but is weak in tension and brittle. Steel is strong in both compression and tension, and also ductile. In reinforced concrete structure, the quantity and location of steel bars has to be so engineered that the failure of the reinforced concrete member happens by steel reaching its strength in tension before concrete reaches its strength in compression. This type of failure is known as ductile failure. Ductile failure being a gradual failure is preferred during an earthquake, because it absorbs more energy caused by vibration and thus gives indication and long warning period prior to failure.

The failure of a column may affect the stability of the entire structure, but the failure of a beam will have only localized effect. Hence, in ductile design approach, it is considered appropriate to engineer the column as strong ductile links than the beams. This method of designing RCC buildings is called the strong-column weak-beam design method, figure - 30.

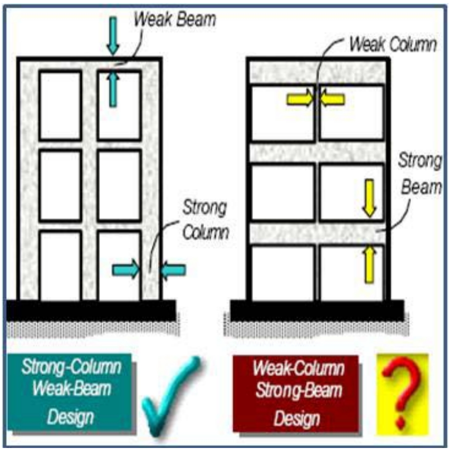


Figure 30 - Strong column weak beam concept in reinforced concrete design [Source: IIT K BMTPC eq tips - 9]

3.7 Seismic Standards of India:

3.7.1 IS 1893

IS 1893 is the main standard that provides the seismic zone map and specifies the seismic design force. For the purpose of determining seismic forces, IS 1893:2002 classifies the country into four seismic zones, Zones II, III, IV and V (figure – 31). The seismic force depends on the mass and seismic coefficient of the structure. The seismic coefficient of the structure depends on properties like the location of the structure (zone), importance of the structure, ductility of the structure etc.

The current revision (2002) has split the code into five parts. Part – 1 deals with general provisions and buildings, part – II with liquid retaining structures, part-III bridges and retaining walls, part – IV, Industrial structures including stack like structures and part – V dams and embankments.

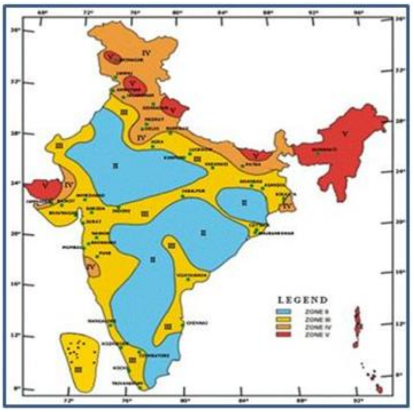


Figure 31 - Seismic zone map of India

IS 1893 refers to two levels of earthquake, the maximum considered earthquake (MCE) and the design basis earthquake (DBE). MCE represents the most severe earthquake effects considered by this standard and DBE is that earthquake effect that is reasonably expected to occur at least once during the life of the structure. The design approach adopted in this standard is to ensure that minor earthquakes (<DBE) are resisted without

any damage, moderate earthquake (DBE) with limited damage and major earthquakes (MCE) without collapse.

3.7.2 IS 4326

A standard that specifies the design and the required detailing for seismic construction of buildings, was published in 1967 (IS 4326: 1967). This standard deals with selection of materials, special features of design and construction for earthquake resistant buildings including masonry construction using rectangular masonry units, timber construction and buildings with prefabricated flooring/ roofing elements.

3.7.3 IS 13920 [23]

Provisions for the ductile detailing of monolithic reinforced concrete frame and shear wall structures are specified in IS 13920 (1993). After the 2001 Bhuj earthquake, this standard has been made mandatory for all structures in zones III, IV and V.

3.7.4 IS 13827 and 13828 [24, 25]

Guidelines in IS 13827 deal with empirical design and construction aspects for improving earthquake resistance of earthen houses, and those in IS 13828 with general principles of design and special construction features for improving earthquake resistance of buildings of low-strength masonry.

3.7.5 IS 13935 [26]

Guidelines in IS 13935 cover general principles of seismic strengthening, selection of materials, and techniques for repair/seismic strengthening of masonry and wooden buildings. The standard provides a brief coverage for individual reinforced concrete members in such buildings, but does not cover reinforced concrete frame or shear wall buildings as a whole. Some guidelines are also laid down for non-structural and architectural components of buildings.

3.8 Seismic Isolators: [W8, W9]

Seismic isolation is a technology to protect the structure from the destructive effects of an earthquake by decoupling the structure from the ground and providing it with additional damping. This decoupling allows the structure to behave more flexibly which improves its response to an earthquake. The added damping allows the earthquake energy to be absorbed by the isolation system and therefore reduces the energy transferred to the structure. Seismic isolation is physically achieved by placing the structure on isolators.

The isolators are laterally flexible elements, yet they are able to carry the vertical loads of the structure. Since the isolators are more flexible than the structure, most of the lateral movements occur in the isolators. As a result the isolated structure experiences less motion and reduced forces. Figure – 32 depicts the comparison of the response felt by an normal structure and a structure on seismic isolators.

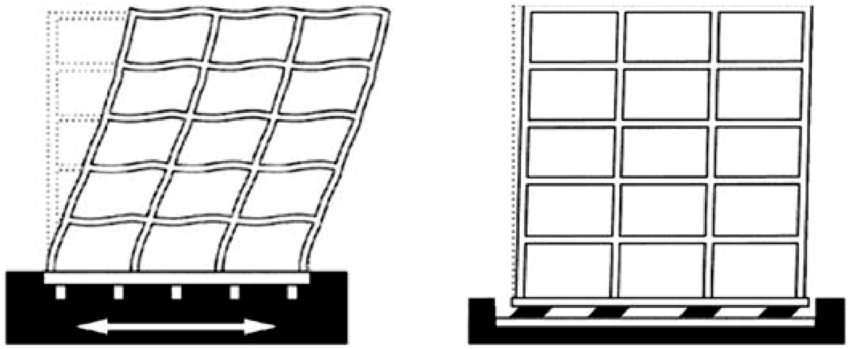


Figure 32 - Impact of base isolators on seismic response of structures [W8]

3.9 Seismic Retrofit: [27, 28, 29]

Seismic upgradation / retrofitting of structures aims at improving the aseismic performance of structures, which is in operation, in terms of their strength, stiffness and ductility, so that can resist seismic effects, stands for current standards and maintaining desired performance level. The principal stages in the complete seismic upgradation programme include, seismic re-evaluation, decision to upgrade, selection of upgradation strategy, design of upgradation measures, verification of upgradation, construction of upgradation measures and monitoring.

Structures may be upgraded adopting one or a combination of the following strategies:

- Local modification
- Removal or reduction of existing irregularities
- Structural stiffening
- Mass reduction
- Structural strengthening
- Enhancement of structural ductility
- Energy dissipation techniques.

Structural upgradation strategies are implemented by engineering appropriate upgradation measures. Some of the methods are

- Damage repair (Crack filling, jacketing, shotcreting)
- Addition of new structural members (shear walls, buttresses, bracings)
- Removal of weakness caused by openings
- Conventional strengthening using materials similar to concrete
- Fibre wrapping
- Energy absorption (Base isolation, supplementary energy dissipation)

4.0 SEISMIC SAFETY OF NPP

4.1 Introduction: [30]

The objective of seismic safety of an NPP is to ensure safety against radiological hazard to the plant, personal, public and environment in the event of design basis earthquake. Seismic safety of a nuclear facility covers five aspects,

- Determination of seismic input for the design and qualification of a nuclear facility at a given site,
- Seismic qualification, i.e. seismic design of a new nuclear facility following current codes/standards,
- Seismic design basis re-constitution, i.e. seismic safety assessment of an operating/existing nuclear installation following current codes/standards,
- Seismic re-evaluation, i.e. seismic safety assessment of an operating/existing nuclear installation and
- Seismic upgrading, i.e. enhancing seismic capacity of SSCs to newly determined seismic hazard loads.

4.2 Level of earthquakes

In accordance with present regulation, a new NPP is designed for two level of earthquakes - S1 level or operating basis earthquake (OBE) and S2 level or safe shutdown earthquake (SSE). OBE is defined as that earthquake which, considering the regional and local geology and seismology and specific characteristics of local sub-surface material, could reasonably be expected to affect the plant site during the operating life of the plant; it is that earthquake which produces the vibratory ground motion for which the features of NPP necessary for continued safe operation are designed to remain functional. If this level of ground motion is exceeded at the site, the plant is to be shut down, then be inspected to determine if any damage had occurred. The plant will be restarted only after it is certified fit for operation. The peak ground acceleration of this level earthquake in horizontal direction should not be less than 0.05g. The return period (mean recurrence interval) should not be less than 100 years when it is determined by a probabilistic method.

SSE is defined as that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local sub-surface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems and components (SSC) are designed to remain functional. These SSC are those necessary to ensure general safety requirements. The seismic event of this intensity has a very low probability of occurrence. PGA of this earthquake level is taken not less than 0.1g. It has a return period of not less than 10000 years.

4.3 Seismic classification of SSC of NPP: [30, 31]

Not all the structures, systems and components (SSC) of an NPP are required to be designed for SSE. Depending on the safety related importance in the event of an earthquake, the SSC of a NPP are categorized into three seismic categories.

i. Seismic category I include those SSC(s), whose failure could directly or indirectly cause accident conditions. Or in other words, this category includes those SSC(s) necessary to ensure general safety requirements. The components of this category are designed for both SSE and OBE.
ii. Seismic category – II includes those SSC(s), which are necessary for continuous operation of the plant and these components without undue risk to the operator, environmental and public are designed only for OBE.
iii. Components not falling under categories I and II, whose failure could not cause undue risk, are included as category – III and these components are designed as per the provisions of IS 1893.

4.4 Aseismic design approach of NPP:

The aseismic design of an NPP is a four step process, almost similar to seismic design of conventional structure. The first step is the determination of ground motion parameters; second step, related to structural layout and shape, which are principally governed by functional requirements and equipment lay out; third step is response analysis and the fourth one is design or qualification.

In contrast to the code based approach generally adopted in conventional structures, determination of ground motion parameters is one of the most important and involved processes for an NPP. For both SSE and OBE, the design basis ground motion parameters are specified in terms of

- Peak ground acceleration
- Response spectral shape
- Acceleration time history.

Next chapter broadly highlights the methods for determination of ground motion parameters.

Determination of structural response i.e. forces in structural elements and acceleration levels are another major step in aseismic design of NPP. In case of an NPP, the complexities of its structures warrant the performance of a detailed dynamic analysis. Either response spectrum or time history analysis method is adopted. The mathematical modeling is very involved and certain aspect of this is presented in Chapter – 6.

Next step in seismic design of an NPP is to combine the induced seismic forces with that of static forces. The rules for combination are specified by AERB safety standard AERB/SS/CSE-1 [32] for concrete structures and AERB/SS/CSE-2 [33] for steel structures. This step also involves qualification of SSC and equipment mounted on the building structure. Details of seismic qualification are given in Chapter – 6.

Seismic design of an NPP follows a much stringent but rational approach considering the risk associated with the failure of nuclear power plant structure. The basic differences in seismic design of a NPP from that of a conventional structure are

- Seismic input for a NPP is derived from site specific data, whereas the seismic input for conventional structures is based on the average parameters for the country given by seismic design codes.

- In contrast to that of a conventional structure, no reduction in seismic load on account of ductile detailing is considered for seismic design of a new NPP.
- An NPP structure is expected to perform elastically for loads corresponding to its design basis earthquake.

4.5 Geotechnical aspects related to seismic safety of NPP [34]

Geotechnical aspects that require due considerations are examined and include the following:

- Investigation of the possibility of liquefaction at the site, which can cause a complete loss of the soil's shear strength, resulting in a bearing capacity failure, excessive settlement, or slope movement.
- Calculation of the settlement of the structure caused by the anticipated earthquake.
- Determination and confirmation of the design parameters for the foundation, such as the bearing capacity and allowable soil bearing pressures.
- Investigation of slopes stability, including lateral deformation of the slope for the additional forces imposed during the design earthquake.
- Evaluation of the effect of earthquake on the stability of earth retaining structures.
- Development of site improvement techniques.
- Determining the type of foundation, such as a shallow or deep foundation, that is best suited for resisting the effects of the design earthquake.

4.6 Summary of seismic safety criteria of NPP: [30]

A number of features are associated with the seismic safety of an NPP. The criteria to achieve this are summarized as below:

- For aseismic design, SSC(s) can be grouped into three categories depending importance related to safety. All seismic Category-1 SSC(s) should withstand the effect of five OBE and one SSE. All seismic Category-2 SSC(s) shall demonstrate the capability to withstand the effect of OBE.
- Seismic ground motion parameter are so derived that they are site specific and the chance of exceeding the derived values of ground motion is extremely low.
- Structural responses due to seismic excitation are determined by analyzing the dynamic behavior of structures and dynamic characteristics of the excitation.
- Unlike conventional structure, methodology adopted in the aseismic design of NPP structures is based on the concept of resisting the entire earthquake load within the elastic range. Design of seismic Category-3 structures may be done in accordance with Indian Standard, "Criteria for Earthquake Resistance Design of Structures", IS 1893.
- Safety against hazard due to geotechnical failure caused by earthquake is ensured.

5.0 DERIVATION OF GROUND MOTION PARAMETERS FOR NPP

5.1 Introduction [2, 35]

To mitigate the seismic hazard, an NPP is designed to withstand the effects of vibratory ground motion arising from strong earthquakes. The design basis ground motion (DBGM) for this purpose is evaluated for each site. This can be determined by probabilistic method or deterministic method. Irrespective of the method, DBGM is characterized by PGA, response spectral shape and a time history compatible with response spectrum. PGA and response spectrum are derived based on site specific studies whereas spectrum compatible time history is generated from the response spectrum using analytical procedures. The DBGM parameters are evaluated for two levels of severity, S1 level earthquake or OBE and S2 level earthquake or SSE.

For estimating the DBGM parameters of a site, the earthquake sources (e.g. faults) around the site needs to be identified and maximum potential earthquake of each source need to be estimated. This is achieved by conducting a detailed investigation of geological and seismological environment of the site. The data on historical and pre-historical seismicity are also collected.

5.2 Geological and seismological investigations: [2, 35]

Geological investigations are carried out in three stages:

i. *Preliminary investigation:* For identifying type of the seismic hazards which need to be considered for the site and to organize detailed investigations in the subsequent stages.
ii. *Detailed investigations:* For evaluation of the seismotectonic status of the region and specification of design basis ground motion; and quantifying the potential of seismically induced flooding and ground failure.
iii. *Confirmatory investigation:* These investigations are required to confirm certain presumptions made and items identified during detailed investigation stage. Confirmatory investigations include those studies and investigations which are to be carried out for a longer period (2 to 3 years or more).

The investigations are conducted in four scales, regional (300km minimum), intermediate range (50km radius), local (5km radius), and site area (within plant boundary). Each set of study leads to progressively more detailed investigation resulting in large volume of data and information as it gets closer to the site. The site area investigations generally help in arriving at the foundation parameters and conducting stability check against liquefaction.

The areas are investigated through satellite imageries, aerial photographs, detailed maps to determine tectonic structures that could be considered as the sources for earthquakes. The historic earthquake data available in earthquake catalogues are also collected. Information on prehistoric seismicity can be obtained by paleoseismic studies [36].

Paleoseismology is the study of the timing, location, and size of prehistoric earthquakes. This focuses on instantaneous deformation of landforms and sediments during individual earthquakes. Paleoseismic history helps to understand aspects of earthquake geology such as regional patterns of tectonic deformation and the long-term behavior of specific faults. It can be used to supplement the calculation of seismic hazard.

Using the information obtained from the investigations, all regional geological and seismological information are compiled, and all related tectonic information within 300 kms of the site are plotted on a map. Epicenters of all known earthquakes greater than m = 3.0 are superimposed on the same, e.g. figure - 33.

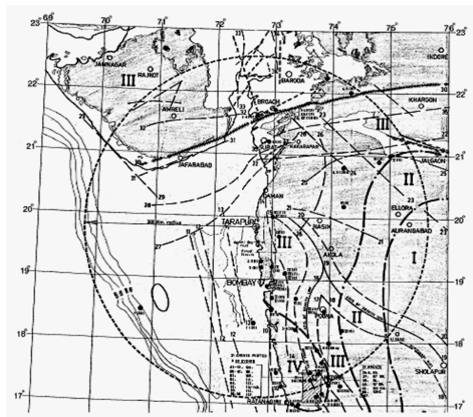


Figure 33 - Compilation of geological information around a site with epicenters mapped [37]

5.3 Determination of DBGM parameters: [2]

Based on the investigations, one would be able to identify seismogenic faults (faults that are capable of generating seismicity) and tectonic provinces (areas with diffused seismicity) in the region and the following information is compiled:

1. Size and shape of earthquake source and its distance from site
2. Seismicity, both historic and prehistoric, and maximum earthquake potential associated with each source. In absence of site specific data, the maximum earthquake potential can be estimated by increasing the maximum intensity of historical earthquake by unity.

Having estimated these parameters of each source, next step is the derivation of value of ground motion that can be produced by each source.

The acceleration produced by the earthquake is a function of earthquake magnitude and distance from the source. The attenuation of ground motion is represented by attenuation relationships. Generally, these empirical relationships have the following form:

$$Ln(y) = C_1 + C_2 m - C_3 R$$

Where 'm' is the magnitude of the earthquake and R is the distance from site. C_1, C_2 and C_3 are constants and are a function of the regional geology and soil conditions. 'y' is the acceleration at site due to earthquake of magnitude 'm' occurring at distance R.

The acceleration is directly proportional to magnitude and inversely proportional to distance. Hence, for estimating the maximum acceleration at a site, one needs to estimate the upper limit magnitude and lower limit of distance.

5.3.1 Deterministic approach for evaluation of DBGM parameters:

In deterministic approach, depending on the seismotectonic conditions and number of earthquake sources, one or more than one earthquakes will be postulated in the region and the PGA is estimated for each postulated earthquake using appropriate attenuation relationship. The parameters that are necessary for estimation are: size of the earthquake (magnitude or intensity) and distance from the site.

5.3.1.1 Evaluation of PGA for S2 level:

PGA of S2 level earthquake is evaluated on the basis of maximum earthquake potential (m) associated with a fault or tectonic province estimated with respect to minimum distance (R) from the NPP site, (figure – 34).

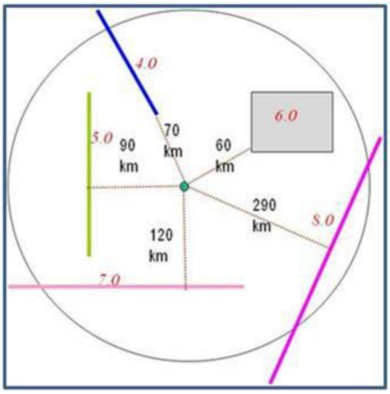


Figure 34 - Calculation model showing the sources around a site with associated maximum earthquake potential (m) and shortest distance (R)

Figure – 35 depicts PGA for different values of distance and maximum earthquake potential, estimated using attenuation relationship. Using this information, the acceleration at site due to earthquakes from each source (with associated maximum earthquake potential, m and minimum distance to site, R) is determined. Table – 5 tabulates the maximum acceleration at site due to each fault/tectonic province estimated from figure - 35. The maximum acceleration among these is considered as the PGA for S2 level earthquake. A minimum value of PGA of 0.1g is used as S2 level PGA and if the calculated PGA is less than 0.1g.

5.3.1.2 Evaluation of PGA for S1 level:

The S1 level motion is derived on the basis of historical earthquakes that have affected the area. In absence of detailed information, the S1 level motion can be specified as half of the S2 level motion where S2 level motion is fixed on the basis of seismotectonic approach elaborated above. The PGA value corresponding to DBE level earthquake should not be less than 0.05g.

5.3.1.3 Response spectra:

Response spectrum is specified in terms of displacement, velocity or acceleration. Response spectra used for NPP design can be Standard Response

Spectra as given in AERB/SG/S-11 or Site Specific Response Spectra derived from site specific study.

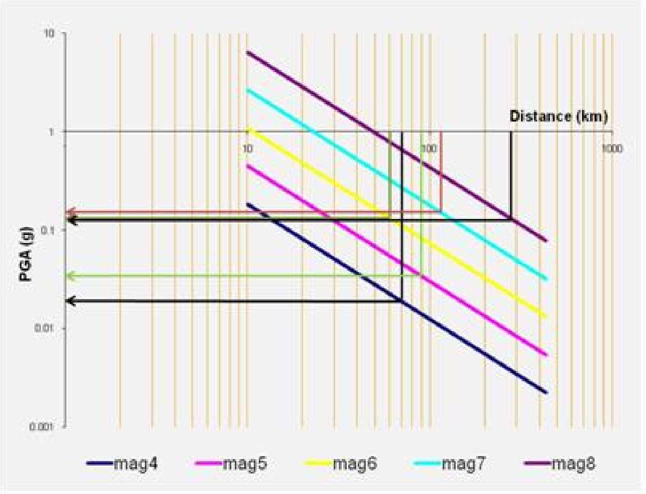


Figure 35 - Attenuation equation

Table – 5: Maximum acceleration at site due to different faults/tectonic provinces

Sl No.	Max potential of fault/tectonic province	Distance from site	PGA at site
1	4.0	70km	0.02g
2	5.0	90km	0.04g
3	6.0	60km	0.12g
4	7.0	120km	0.15g (Max PGA)
5	8.0	290km	0.11g

Site Specific Spectra is generally adopted in the design of NPP as due considerations can be given for size of the earthquake, source mechanism, distance from the source, transmission path characteristics and site characteristics. Response spectral shape for site specific spectra is generally derived from records of strong motion time histories at site. In case of non-availability of sufficient records, response spectral shapes derived for sites having seismic, geological and soil characteristics similar to that of the site under consideration can be used.

The method for derivation of site specific response spectra is as follows:

- Several strong motion accelerograms corresponding to horizontal direction of motion are collected from the site or from sites of similar geological and lithological features.
- These accelerograms are normalized to its PGA, i.e. all records are divided by corresponding PGA so that after normalization, maximum PGA is unity for all records, figure -36.

- Response spectra of these accelerograms are evaluated for different values of damping.
- Spectra corresponding to 84th percentile (mean + sigma) provide the design spectra, figure - 37.

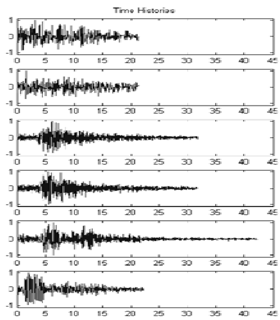


Figure 36 - Ensemble of accelerograms normalized to PGA and respective response spectrum corresponding to 5% damping

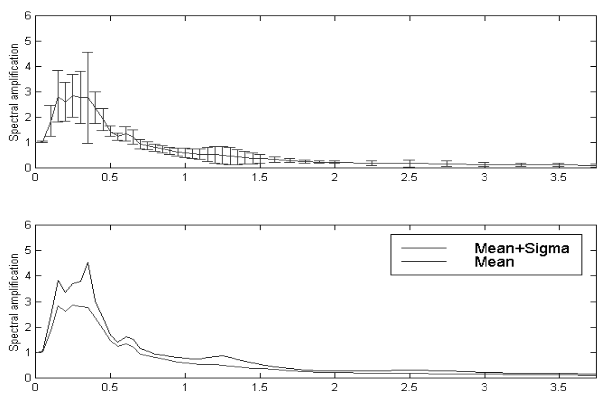


Figure 37 - Derivation of spectral shape from the ensemble of response spectrum

5.3.1.4 Time Histories

Time histories of vibratory ground motion are developed considering all the prescribed ground motion parameters and correspond to both S1 and S2 levels. Time histories are so derived that they are compatible with the design response spectra of 5% damping. In addition, it is ensured that the time history satisfies the constraints on specified values of peak ground acceleration, rise-time to peak acceleration, duration of strong motion etc.

5.3.2 DBGM in Vertical Direction

The peak ground acceleration, response spectra and spectra compatible time history for vertical direction are evaluated separately using the same procedure as for horizontal motion. If sufficient data are not available, it is taken as $2/3^{rd}$ of the corresponding value along horizontal direction. The same spectral shape and time histories generated for horizontal direction of motion are generally used for vertical motion.

5.3.3 DBGM in Two Orthogonal Horizontal Directions

Peak accelerations in two orthogonal horizontal directions are not same. It can be quantified by the ratio between the two orthogonal horizontal directions. In the absence of data, the peak accelerations in two directions can be considered equal. The spectral shape and time history are same in both directions.

5.3.4 DBGM parameters for Indian NPP sites

Design basis ground motion parameters for the Indian NPP sites have been till date established by deterministic approach. Table - 6 tabulates the PGA value for S1 and S2 level earthquakes at all Indian NPP sites.

Table – 6: PGA values for Indian NPP sites

Site	PGA – S1 level	PGA – S2 level
Tarapur (Maharashtra)	0.100g	0.200g
Kota (Rajasthan)	0.050g	0.100g
Kalpakkam (Tamil Nadu)	0.078g	0.156g
Narora (UP)	0.150g	0.300g
Kakrappar (Gujrat)	0.100g	0.200g
Kaiga (Karnataka)	0.100g	0.200g
Kudankulam (Tamil Nadu)	0.050g	0.150g

5.3.5 Probabilistic Approach for Evaluation of DBGM Parameters [10]

Determination of ground motion parameters by probabilistic method is accomplished by performing a probabilistic seismic hazard analysis (PSHA) [10]. Unlike maximisation of single valued earthquake events as in deterministic approach, probabilistic approach takes into account the probable distribution of earthquake magnitudes in each source, probable distances within that source where earthquakes could originate and dispersion of acceleration estimated using attenuation equations. In PSHA methodology, occurrence of earthquakes is usually considered as Poisson process. This means that the events have an average occurrence rate and could occur independent of the time elapsed since last event.

PSHA involves four steps (figure – 38):

- Specification of the seismic-hazard source model(s) (zonation);
- Specification of earthquake recurrence relationships which reflect earthquake activity in the source
- Specification of the ground motion model(s) (attenuation relationship(s)); and
- The probabilistic calculation.

The outcome of PSHA is a hazard curve which depicts the annual probability of exceedence of different values acceleration, ‘Y’, figure-39. The probability of exceedence of a particular value of acceleration is the product of (1) probability of occurrence of an earthquake with magnitude ‘m’ at a distance ‘R’. (2) The probability of exceedence of acceleration above the value 'Y' given 'm' and 'R'. By summing the scenarios for all possible ranges of sources, magnitudes and distances, one would get total probability of exceedence beyond acceleration 'Y'. By repeating this exceedence for different values of 'Y', one can estimate the seismic hazard curve of the site.

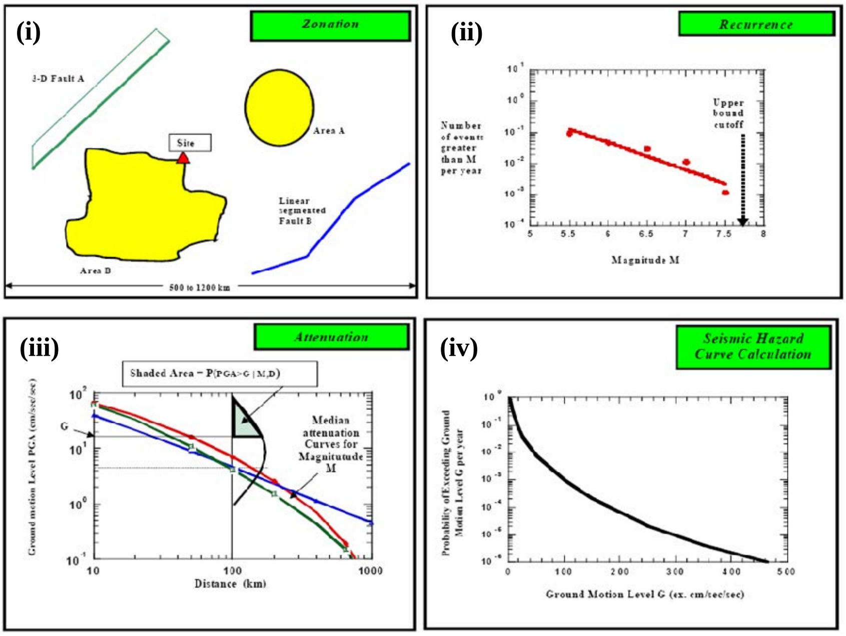


Figure 38 - Elements of PSHA [38]

One of the major advantages in this method is the possibility for incorporation of uncertainties. Uncertainties are introduced by lack of data and/or lack of knowledge, inadequate modeling, etc. These uncertainties can be taken into account by developing alternate scenarios and models.

For detailed treatment of PSHA, readers are requested to refer [10].

Figure 39 - Typical hazard curve

6.0 SEISMIC DESIGN AND QUALIFICATION OF NEW NPP

6.1 Introduction [39]

All structures, systems and components (SSCs) important to safety of an NPP are designed or qualified to ensure safe performance against DBGM. As explained earlier, aseismic design involves three basic steps, viz. selection of an appropriate structural configuration, determination of seismic response i.e. earthquake induced forces in structural elements by analysis and determination of elemental cross section properties (cross section dimensions and reinforcements for RCC structures, diameter and thickness for piping etc) following appropriate codal provisions. In case of seismic qualification, the cross section properties of structural elements are known. The earthquake induced forces in structural elements can be determined by analysis. The structural adequacy of the elemental cross section for the induced forces is then checked following appropriate codal provisions.

Plant specific SSCs are designed, while generic items like pumps, motors, heat exchangers are qualified to ensure their safe performance during design basis earthquake. For this purpose the safety related components of a nuclear facility are categorized into three seismic categories as explained earlier. Seismic qualification of SSCs can be performed by the use of one or more of the following approaches [39]:

- Analysis;
- Testing;
- Earthquake experience;
- Comparison with already qualified items (similarity).

It is also possible to use combinations of these methods. Qualification generally includes qualification of structural integrity as well as qualification for operability or functionality. Analysis is generally main tool for qualification, especially where structural integrity is of main concern and that are of a size or scale to preclude their qualification by testing. Civil engineering structures, tanks, distribution systems and large items of equipment are usually qualified by analytical methods. On the other hand, testing is adopted to qualify those components which are rationally not amenable to analysis. These components are small, sensitive devices used in plant safeguards equipment, where measured acceleration and malfunction levels (functionality) are the failure criteria.

6.2 Qualification by analysis [22, 31]

Qualification by analysis includes the following major tasks:

- Mathematical modeling
- Analysis including main system and subsystem analyses

Modeling task includes derivation of the seismic excitation in terms of DBGM parameters as well as modeling of the stiffness, mass and damping characteristics of SSC. The energy of a vibrating system is dissipated by various mechanisms, which is known as damping. It is difficult to identify and describe each of these energy dissipating mechanism of a SSC. Hence, damping is modeled in a highly idealized manner, generally in the form of equivalent viscous damping.

Analytical model can be different according to the structural characteristics of the components e.g. lumped mass models, one dimensional models, axisymmetric models, two or three dimensional finite element models. Ideally one would like to model the structure incorporating all its complexity in a full three dimensional model in which the structure is idealized using solid elements or shell elements depending on the configuration. But, during dynamic analysis, these models produce local modes with little participation of mass and are of no consequence to the global behaviour of the structure. Also a 3-D model requires large computational effort. Hence for simplicity, an alternate approach of lumped model which provides insight into the global behavior of the structure is used. During the later part of the analysis, the 3-D model is utilized for obtaining a better approximation of stress resultants near discontinuities. Both methods are equally accepted for dynamic analysis of NPP components. The lumped mass approach was adopted for seismic analysis during earlier days. Currently with the advancement of high speed computing, 3-D models are also used for complete dynamic analysis of NPP. Figure – 40 depicts the lumped mass model of reactor building including containment structure [40] and as well 3-D model of the containment structure of an NPP.

Seismic analysis of an NPP comprises of main system analysis and subsystem analysis [41]. Main systems include those civil engineering structures which house the mechanical, electrical and instrumentation systems. These mechanical, electrical and instrumentation systems are referred to as the subsystems. Analysis of main system and subsystem shall ideally be carried out together and such an analysis is termed as coupled analysis. However, the coupled model may sometimes be too cumbersome or possibly ill conditioned for analysis. In such case, the main systems and subsystems are analyzed separately by decoupling them from each other. Major structures that are considered in conjunction with foundation media constitute the main system. Other SSCs attached to the main system constitute the subsystems. There are defined criteria of decoupling the main system and subsystem for analysis depending on their mass ratio and frequency ratio. Main system analysis is carried out to obtain the structural response of main system components or civil engineering structures. One of the outputs of main system analysis is floor response spectrum, which is used as input for subsystem analysis.

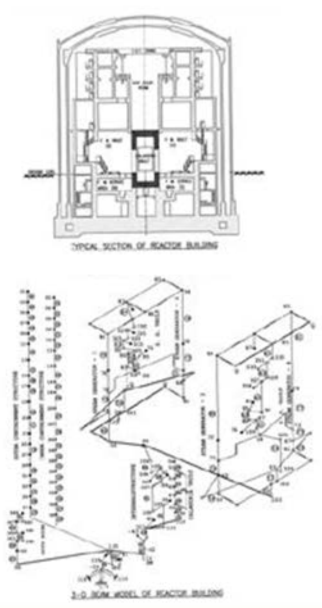
Acceptable methods of seismic response analysis of nuclear facilities include:

- The time-history method
- The response-spectrum method
- Equivalent Static method

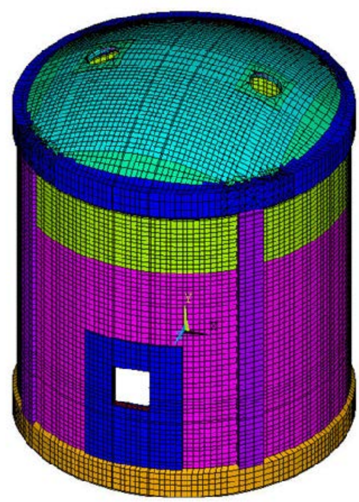
Generally, to determine the structural response of main system components, is carried out by response spectrum analysis method for the purpose of design. Time history analysis method is adopted to derive the acceleration response time history at different elevations/floors of the main systems, from which the floor response spectrum is derived. Subsystem analysis is also carried out by response spectrum method using the floor response spectrum as input. In some cases time history analysis is carried out, especially when operability of a system is to be qualified following stringent acceptance criteria.

6.3 Seismic qualification by testing [39, 42]

Testing of the actual item or prototype is a method of direct seismic qualification. Seismic qualification by testing is generally conducted on shake table. A component is subjected to input motion equivalent or similar to that of DBGM. The motion experienced by the component being tested is measured using the instrumentation available in the shake table and the response spectrum corresponding to this motion is called the Test Response Spectrum (TRS). The requirements to be met are specified by the Required Response Spectrum (RRS). RRS could be specified by the floor response spectra at the location where the component is mounted, when a site specific test is being carried out. The component is qualified, if it continues to perform its intended function when the TRS envelops the RRS.



(a) Lumped mass model of reactor building



(a) 3-D finite element model of reactor building

Figure 40 - Seismic analysis model of reactor building of an NPP

Types of testing include:

- Type approval test (fragility test);
- Acceptance test (proof test);
- Code verification test;
- Low impedance test (dynamic characteristic test).

Direct qualification by testing makes use of type approval and acceptance tests. The type approval (fragility) test is generally used for standard electrical components and mechanical components when design margins to failure, damage or non-linear response and identification of the lower bound failure mode have to be evaluated. Such testing is typically carried out by means of a shake table.

The acceptance (proof) test is also used for electrical and mechanical components to demonstrate their seismic adequacy. It is typically performed by manufacturers to demonstrate compliance with procurement specifications. Such testing is typically carried out by means of a shake table.

The code verification test is important for reliable analytical work. Computer codes should be verified before their application by means of analyses made using an adequate number of test results or results obtained from other appropriate computer codes or analytical procedures. Low impedance (dynamic characteristic) tests are limited to identify similarity or to verify analytical models.

6.4 Seismic qualification based on earthquake experience

Seismic qualification of SSCs by means of the use of experience [43] from strong motion seismic events is having a growing application.

The principal requirements of this method include that the level of seismic excitation experienced during a real earthquake by an item identical to the one under qualification exercise should effectively envelop the seismic design motion at the point of installation in the plant building. The item being qualified and the item that has seen the strong motion should have the similar characteristics, support or anchorage arrangement. This method of qualification is widely used for seismic evaluation of existing facilities.

6.5 Seismic qualification based on similarity [39]

In this method, particular equipment is compared with another similar equipment which has already been qualified. The similarity principle is evoked to show that the two pieces of equipment respond essentially the same and that the difference between the two can only help maintain or improve the functional and structural integrity of the equipment.

Similar appearance or geometric size does not establish a basis for dynamic similarity. Similarity requires the documentation of related mass, stiffness and damping characteristics. Structural laws relating to dynamic response determines the similarity.

7.0 SEISMIC EVALUATION OF EXISTING NPP

7.1 Introduction: [44]

Seismic evaluation of an existing nuclear facility is prompted by from the following considerations:

(a) Evidence of greater seismic hazard at site than expected before, owing to new or additional data and/or to new methods.

(b) Regulatory requirements, such as periodic safety reviews, to ensure that the plant has adequate margins for seismic loads.

(c) Lack of anti-seismic design or poor anti-seismic design.

(d) New technical findings such as vulnerability of some structure (e.g., masonry walls) or equipment (e.g., relays), other feedback and new experience from real earthquakes.

7.2 Principles of Seismic re-evaluation [41, 44]

Seismic re-evaluation (or seismic evaluation) is distinguished from seismic qualification primarily in that seismic qualification is intended to be performed at the design stage of a plant, whereas seismic re-evaluation is intended to be applied after a plant has been put in operation. Primary objective of seismic re-evaluation is to review the seismic capacity of safety related SSCs of the plant required to achieve a set of safety objectives. This review exercise is conducted with respect to the ground motion, termed as review basis ground motion (RBGM). The RBGM parameters are derived following same criteria of S_2 level earthquakes or SSE.

Objective of seismic re-evaluation is to assess the capability to perform the following safety functions of an existing plant, in the event of RBGM,

- ❑ Safe shutdown of the plant;
- ❑ Maintaining the plant in safe shutdown condition;
- ❑ Long-term decay heat removal;
- ❑ Containment/confinement of radioactive inventory.

Seismic re-evaluation aims at re-assessing the safety of the plant, with respect to the above four functions, against RBGM parameters with consequent upgrading, if found necessary. General approach to seismic re-evaluation is outlined below:

- Evaluation of seismic hazard of the site,
- The re-evaluation focuses on those SSCs essential to achieve the desired safety objectives without compromising the defense in depth[1].
- The additional capacity of the SSC required to withstand an earthquake is evaluated considering inherent conservatism of the original design, taking into account certain limiting assumptions in terms of operational status, probability of other external events, material behavior.

[1] Defense in depth means provision of multiple levels of protection for ensuring safety of workers, the public or the environment.

- The seismic safety assessment uses conservatism carefully and employs the best available techniques to evaluate capacity of the plant in terms of RBGM parameters, and possibly resulting in upgradation.

Safety analysis is carried out to identify the structures systems and components (SSC) required to perform safety functions satisfying the limiting operating conditions and including those necessary to guarantee the existence of defense in depth in the event of RBGM. Safety analysis adopts an event tree / fault tree approach to identify the accident sequences and to list the Structures, Systems and Components (SSC) required to ensure the safety functions. The steps involved in safety analysis are divided into four major activities:

- Postulation of seismic induced initiating events,
- Formulation of event trees for each of the postulated events to accomplish the required safety functions,
- Formulation of fault trees for each of the frontline systems appearing on the event trees, and
- Determination of list of SSCs by a minimal cut set evaluation of the fault trees.

Only those SSCs which are required to perform the safety functions satisfying the limiting operating conditions and including those necessary to guarantee the existence of defense in depth in the event of RBGM are evaluated. The list of these SSCs is known as seismic structures, systems and components (SSSCs) list (SSSCL) of the plant.

Comparison of seismic qualification and seismic evaluation is given in figure – 41. In seismic evaluation of an existing plant, seismic capacity is assured following current seismic criteria and considering identical SSSC(s) required to successfully perform the safety functions mentioned above.

7.3 Seismic Capacity Assessment [41, 45]

There are two main approaches for assessing the seismic capacity of an existing nuclear facility:

- ❑ The seismic margin assessment (SMA) and
- ❑ The seismic probabilistic safety assessment (SPSA).

7.3.1 Seismic Margin Assessment:

Seismic margin is generally expressed in terms of the earthquake level that compromises plant safety, specifically leading to melting of the reactor core. The measure of seismic capacity adopted in seismic margin reviews is the so-called "High Confidence, Low Probability of Failure" (HCLPF) capacity, usually given in unit of peak ground acceleration. This is a conservative representation of capacity, and in simple terms, corresponds to the earthquake level at which it is extremely unlikely that failure of the component will occur. From the mathematical perspective, the HCLPF capacity values are approximately equal to a 95% confidence (probability) of not exceeding about a 5% probability of failure. Using the HCLPF concept, the search for the seismic margin shifts to determining the plant-level HCLPF capacity and comparing it with the review basis earthquake.

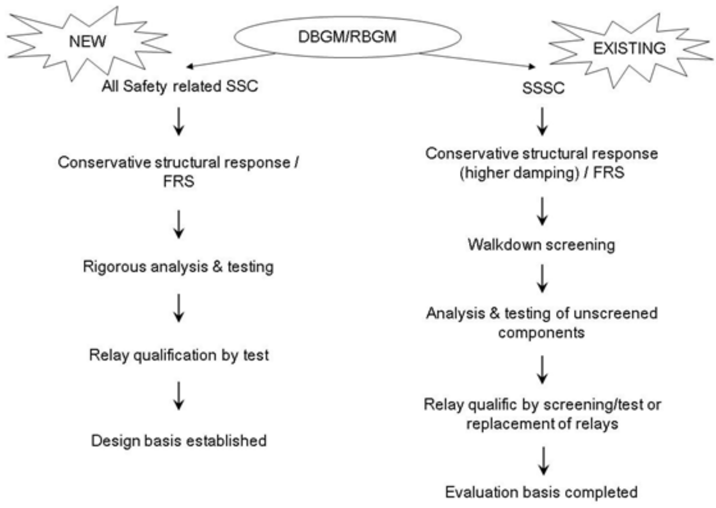


Figure 41 - Seismic re-evaluation vs seismic qualification [43]

7.3.2 Seismic Probabilistic Safety Assessment: [41, 46]

Objective of seismic probabilistic safety assessment (SPSA) is involved the following major activities:

- Probabilistic Seismic Hazard Analysis to derive RBGM parameters
- Structures and components fragility analysis.
- Establishing plant fragility using plant logic analysis
- Risk quantification.

Seismic Fragility is the conditional probability of failure for a given value of seismic input parameter e.g. PGA.

SPSA differs from probabilistic safety analysis with internal events, such that in SPSA instead of dealing with random equipment failures, earthquake is considered as a cause for failure; and the frequency of failure of a particular component is computed from seismic hazard of the site and fragility of the component. SPSA results in identification of accident sequences leading to core damage and frequency of each of those. Principal difference between SPSA and SMA is that SMA, instead of looking for a core damage frequency as is the case of SPSA, looks for the level of earthquake below which core damage is unlikely.

7.4 Tasks for seismic evaluation [41]

Even though the two methods, SMA and SPSA, differ in many respects, the major activities to be undertaken to accomplish the final goal by both these methods are almost similar. Hence, it is generally noticed that performing SPSA along with SMA is beneficial as most of the tasks and associated activities are common. Figure – 42 depicts typical flow diagram of seismic re-evaluation procedure of an existing NPP.

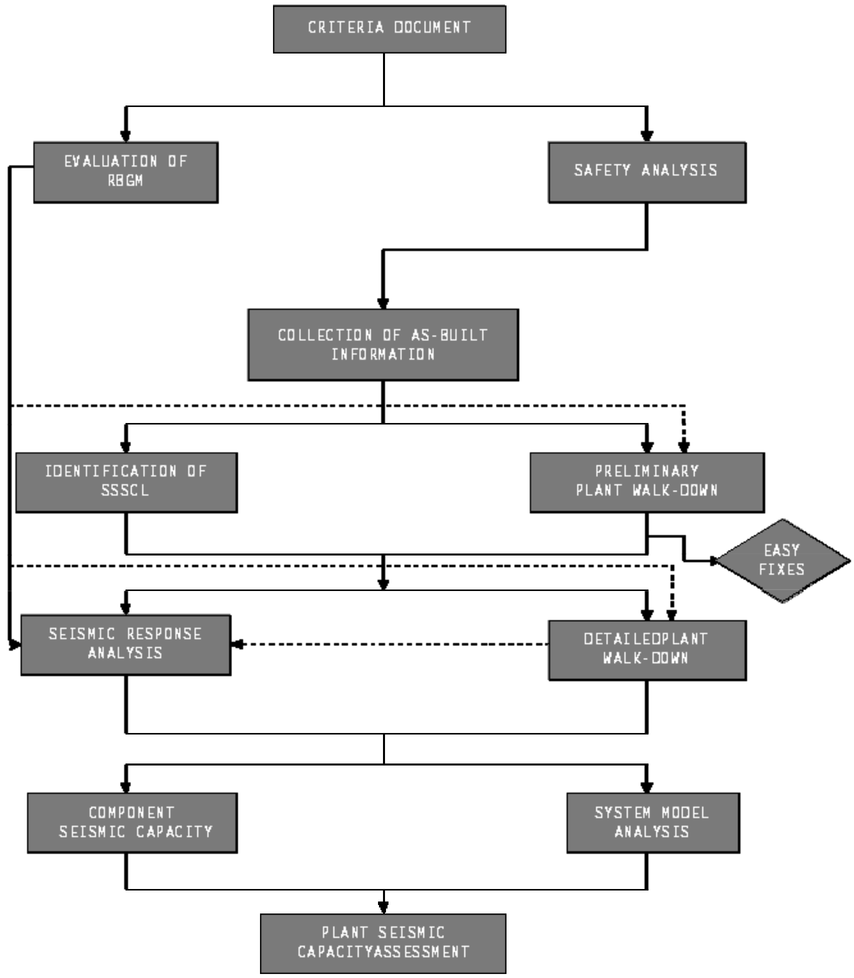


Figure 42 - Task flow for seismic re-evaluation [41]

Plant walk-down is an important task for both SMA and SPSA while carrying out seismic re-evaluation of existing nuclear facilities. The objectives of walk down include:

- Confirm the completeness of SSSCL, their required functions, their possible failure modes, to screen out the SSSCs which feature a seismically robust construction
- Collection of as-built data and assessment of seismic capacity of components in SSSCL.

- Identification the easy-fix solutions/upgrades that can be carried out regardless of any analysis.
- To define representative configurations for further evaluations.

The main focus of walk-downs are on:

- Equipment characteristics and inherent seismic capabilities.
- Anchorage of equipment
- Load path from the anchorage through the equipment
- Spatial and other types of interaction.

The plant walk-down is generally carried out in two stages, preliminary and detailed. Preliminary plant walk-down will be carried out by the plant operating personnel to obtain the necessary information for generating the SSSCL. The main objective of this walk-down is the identification of those obvious seismically robust SSCs, which can be considered as having adequate seismic capacity and, therefore, are screened out of further evaluations. Those SSCs, which require a modification or whose seismic capacity is uncertain, are further evaluated in detail during detailed plant walk-down.

8.0 SEISMIC INSTRUMENTATION [39, 47]

8.1 Introduction

The main objective of the seismic instrumentation is to record the ground motion arising due to natural and manmade disturbances and dynamic behavior of SSC inside NPP, during an earthquake. Information recorded in seismic instrumentation helps in:

i) Assessing safety of the plant after an seismic event
ii) Validating the aseismic design
iii) Improvement in aseismic design technique.

There are some instruments which could initiate the shutting down process of an NPP in the event of an earthquake having high magnitude.

Seismic instrumentation is installed at nuclear power plants for the following reasons [28]:

- For structural monitoring:
  - To collect data on the dynamic behaviour of SSCs of the nuclear power plant and to assess the degree of validity of the analytical methods used in the seismic design and qualification of the buildings and equipment.
- For seismic monitoring:
  - To provide alarms for alerting operators of the potential need for a plant shutdown depending on post-earthquake inspections.
- For automatic scram systems:
  - To provide triggering mechanisms for the automatic shutdown of the plant.

A variety of instruments are available for measurement of earthquake response (acceleration/velocity/displacement). These instruments try to capture the earthquake response at the point of attachment to the structure. The range of instruments available includes accelerographs, structural response recorders (SRRs), peak accelerographs and seismic switches. Accelerographs record the full history of vibratory motion during the occurrence of earthquake. SRRs record spectral accelerations at specified frequencies and peak accelerographs record the maximum acceleration observed at that location.

Data for immediate decision making process will not be available from SRRs and peak accelerographs, as the recorded data require post processing. This can be achieved by switches (both for PGA and response spectral values), which instantly conveys information on exceedance of a set point of acceleration. With the advances made in digital electronics and signal processing, it has now become feasible to conduct the real time analysis of data from accelerographs also.

8.2 Selection of instruments

The choice of the instruments is done by specialists in the field considering the dynamic range, trigger level, frequency band, damping, recording speed etc needed to specifically assess acceleration time history, structural response etc. specific to seismic environment.

8.3 Location of seismic instruments

As a minimum, the accelerographs are located in free-field, foundation of containment structure, two elevations (excluding the foundation) on the internal structure within the containment of a reactor as well as on foundation and at an elevation of an independent Seismic Category I structure. If seismic isolators are used, instrumentation is placed on both the rigid and isolated portions of the same and an adjacent structure, as appropriate, at approximately the same elevations. In addition, behavior of a representative piping equipment and their supports are also monitored with the help of seismic instrumentation.

8.4 Multiunit sites:

Instrumentation in addition to that installed for a single unit is not required, if essentially the same seismic response is expected at the other units based on the seismic analysis used in the seismic design of the plant. However, if there are separate control rooms, annunciation for exceedance of set parameters should be provided to both control rooms.

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APPENDIX – I: ILLUSTRATIVE EXAMPLE OF RESPONSE SPECTRUM METHOD

Consider the idealized model of the three storey building shown in figure AI-1 below. Assume that the building will vibrate in the lateral direction and vibrations in other two perpendicular directions are constrained. Therefore, this building will have 3 dynamic degree of freedom and 3 modes of vibration.

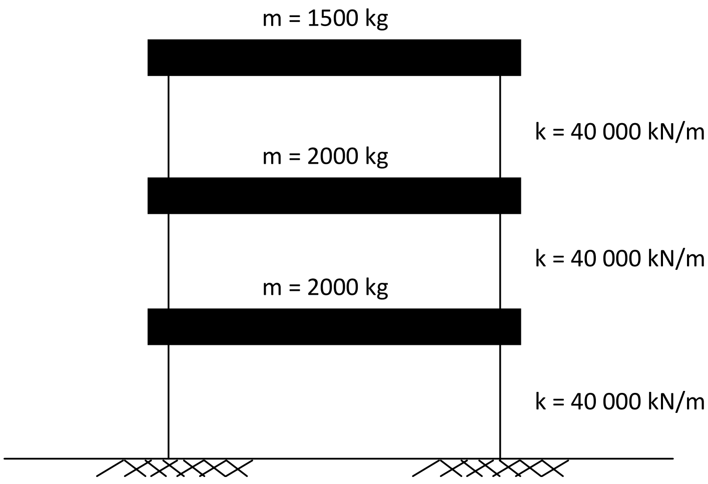

Figure AI-1 : Idealized model of three storey building

The basic dynamic equilibrium equation of multi degree of freedom system can be written as

$$M\ddot{x} + C\dot{x} + Kx = F(t) \qquad \text{(I-1)}$$

Where, M = mass matrix; C = damping matrix; K = stiffness matrix

Consider the solution of this equation to be harmonic and of the form

$$x = \nu \sin\omega t \qquad \text{(I-2)}$$

Substituting for x, in equation (I-1), the equation is transformed to the following form for a undamped free vibration system

$$K\,v - \omega^2 M\,v = \{0\} \qquad \text{(I-3)}$$

For the above MDOF system,

Mass matrix $M = 1000 \begin{bmatrix} 2 & 0 & 0 \\ 0 & 2 & 0 \\ 0 & 0 & 1.5 \end{bmatrix}$ kg and

Stiffness matrix $K = 1000 \begin{bmatrix} 80 & -40 & 0 \\ -40 & 80 & -40 \\ 0 & -40 & 40 \end{bmatrix} N/m$

The eigen solution of equation (I-3) will result in eigen values (ω) i.e natural frequency and eigen vector which are the mode shapes [9].

Substituting M and K in equation (I-3) and taking an eigen solution we get

$$\begin{vmatrix} 80 - 2\omega^2 & -40 & 0 \\ -40 & 80 - 2\omega^2 & -40 \\ 0 & -40 & 40 - 1.5\omega^2 \end{vmatrix} = 0$$

Solving, we get the natural frequency and period as,

$\omega_1 = 2.138\ radians/sec$	T_1 = 2.939 secs.
$\omega_2 = 5.877\ radians/sec$	T_2 = 1.069 secs.
$\omega_3 = 8.219\ radians/sec$	T_3 = 0.764 secs.

The eigen vectors corresponding to each of the eigen values can be determined by substituting the value of ω in equation (I-3) and solving it. These eigen vectors are called the mode shapes and is represented by {ϕ}. Mode shapes are nothing but a sort of scaled displaced shape of the structure for that mode of vibration. Figure AI-2 below shows the displaced shape for first mode.

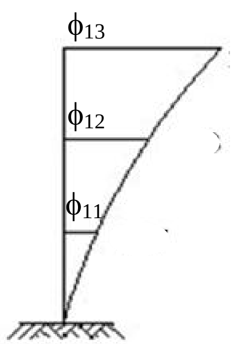

Figure AI-2 : Displaced shape for first mode

For First mode, $\{\emptyset\}_1 = \begin{Bmatrix} 0.47 \\ 0.83 \\ 1.00 \end{Bmatrix}$; For Second mode, $\{\emptyset\}_2 = \begin{Bmatrix} -1.08 \\ -0.30 \\ 1.00 \end{Bmatrix}$;

And for Third mode, $\{\emptyset\}_3 = \begin{Bmatrix} 1.11 \\ -1.53 \\ 1.00 \end{Bmatrix}$

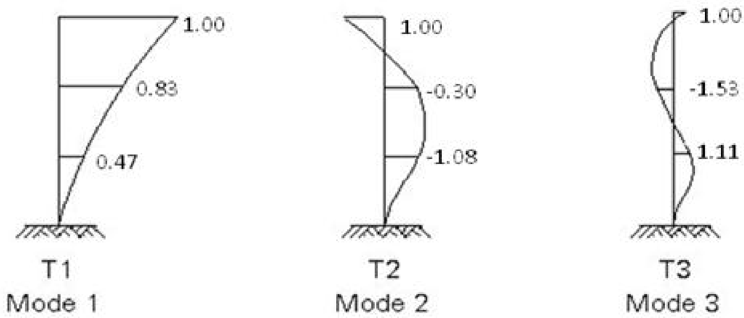

Figure AI-3 : Modal Deformation

For a MDOF system subjected to seismic excitation, F(t) in equation (I-1) will be

$$F(t) = -\mathrm{M.r.}\ddot{\mathrm{x}}_{\mathrm{g}}$$

Here r is called the excitation influence vector. It consists of 1's corresponding to translational degrees of freedom along the direction of ground motion and 0's corresponding to other degrees of freedom.

Therefore equation (I-1) can be rewritten as

$$\mathrm{M}\ddot{\mathrm{x}} + \mathrm{C}\dot{\mathrm{x}} + \mathrm{Kx} = -\mathrm{M.r.}\ddot{\mathrm{x}}_{\mathrm{g}}$$

This equation can be decoupled into

$$\ddot{\mathrm{y}} + 2\omega_{\mathrm{n}}\zeta_{\mathrm{n}}\dot{\mathrm{y}} + \omega_{\mathrm{n}}^2\mathrm{y} = P_n.\ddot{\mathrm{x}}_{\mathrm{g}} \quad \mathrm{n} = 1,2,3,\ldots..\mathrm{N}$$

Where,

$$P_n = \frac{\{\emptyset\}_n^T\, M\, r}{\{\emptyset\}^T\, M\, \{\emptyset\}_n} \text{ and}$$

$$y = [\emptyset]\ \mathrm{x}$$

P_n is called the earthquake mode participation factor [9] of mode 'n' for the direction of the ground motion described by $\ddot{x}_g$. It denotes how much each mode participates in the vibration of the building when subjected to base excitation.

For the example problem the earthquake participation factor for first mode is given by

$$P_1 = \frac{\begin{Bmatrix}0.47\\0.83\\1.00\end{Bmatrix}^T 1000 \begin{bmatrix}2 & 0 & 0\\0 & 2 & 0\\0 & 0 & 1.5\end{bmatrix}\begin{Bmatrix}1\\1\\1\end{Bmatrix}}{\begin{Bmatrix}0.47\\0.83\\1.00\end{Bmatrix}^T 1000 \begin{bmatrix}2 & 0 & 0\\0 & 2 & 0\\0 & 0 & 1.5\end{bmatrix}\begin{Bmatrix}0.47\\0.83\\1.00\end{Bmatrix}} = 1.235$$

Similarly for modes 2 and 3
P_2 = -0.314 and P_3 = 0.076

Assume that the site has a PGA of 0.15g, and the design is to be as per the IS-1893 response spectrum (Critical damping ratio (ξ_n) = 5%), for rock sites. Then the spectral amplification factor corresponding to the time period of the three modes are derived as shown in figure – AI-4 below.

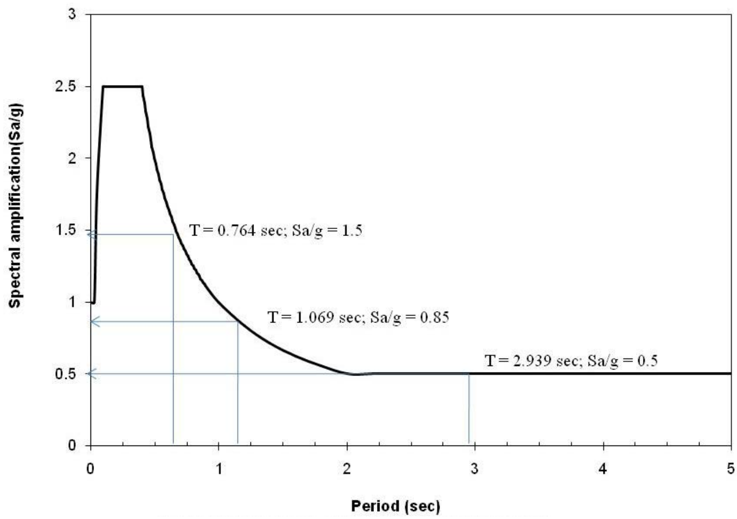

Figure AI-4 : Spectral amplification factor

Table- AI-1: Spectral amplification factor corresponding to time period

Mode	*Period*	*Sa/g*	*Sa = Sa/g * PGA*
1	2.939	0.5	0.5*.15g = 0.075g
2	1.069	0.85	0.1275g
3	0.764	1.5	0.225g

Maximum acceleration in mode 'n' at floor 'i' is given by

$$A_{ni,max} = \emptyset_{ni} P_n S_{an} \quad \text{(I-6)}$$

If m_i is the mass at floor 'i', the maximum lateral force at floor 'i' in mode 'n' is

$$F_{ni,max} = m_i A_{ni,max} = m_i \emptyset_{ni} P_n S_{an}$$

Calculation of maximum acceleration and maximum lateral force for floor 3 in mode 1 is illustrated below:

$$A_{13,max} = \emptyset_{13} P_1 S_{a1} = 1.00 \times 1.235 \times 0.075 \times 9.81 = 0.91\ m/sec^2$$

$$F_{13,max} = m_i A_{ni,max} = m_i \emptyset_{ni} P_n S_{an} = 1500 \times 1.00 \times 1.235 \times 0.075 \times 9.81$$
$$= 1362\ N$$

The maximum lateral forces at each floor calculated in similar manner for each mode is tabulated below:

Table- AI-2: Maximum lateral forces at each floor for each mode

Flr	**Mass**	ϕ_{1x}	ϕ_{2x}	ϕ_{3x}	A_{1x}	A_{2x}	A_{3x}	F_{1x}	F_{2x}	F_{3x}
	(kg)				**(m/s²)**	**(m/s²)**	**(m/s²)**	**(N)**	**(N)**	**(N)**
1	2000	0.47	-1.08	1.11	0.43	0.42	0.19	854.1	848.3	372.4
2	2000	0.83	-0.3	-1.53	0.75	0.12	-0.26	1508.3	235.6	-513.3
3	1500	1	1	1	0.91	-0.39	0.17	1362.9	-589.1	251.6

Note: Subscript 'x' represents floor number

The maximum storey shear $V_{ni,max}$ in mode 'n' within storey 'i' are obtained by summing up the maximum lateral forces $F_{ni,max}$ of all floors above storey 'i'.

Hence,

$$V_{ni,max} = \sum_{j=i}^{n} F_{nj,max} \qquad \text{(I-7)}$$

For storey 1 in mode 1,

$$V_{11,max} = \sum_{j=1}^{3} F_{1j,max} = 854.13 + 1508.36 + 1362.98 = 3725.47$$

Similarly for other storey,

Table- AI-3: Maximum shear forces at each floor for each mode

	V_{1x}	V_{2x}	V_{3x}
	(N)	**(N)**	**(N)**
Floor - 1	3725.47	494.86	110.72
Floor - 2	2871.34	-353.47	-261.69
Floor - 3	1362.98	-589.12	251.63

Adopting SRSS method of modal combination, maximum storey shear at floor 'i' due to all modes of vibration

$$V_i = \sqrt{\sum_{r=1}^{N} V_{ni}^2} \qquad \text{(I-8)}$$

Therefore, for floor 1,

$$V_1 = \sqrt{(3725.47)^2 + (494.86)^2 + (110.72)^2} = 3759.8\ N$$

Similarly V_2 = 2603.1 N;   and V_3= 1506.1 N