Chapter 1
Earthquake Engineering and Technology

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Abstract Earthquake engineering and technology has been dealt with by introducing different sub-disciplines thereof, contemporary practices, and the latest developments. Upon giving background to open-up various topics, the expertise required and professionals involved in various disciplines have been systematically presented by giving information exchanges between the domains and sub-disciplines. Genesis of earthquakes from seismology viewpoint is given by explaining plate tectonics and its numerical modelling along with applications. Causes of earthquake in Himalayan subduction zone are described, upon describing seismic waves and their propagation from a medium. Gradually changing over the discussion from geology and seismology to geotechnical and structural earthquake engineering, seismic (dynamic) soil-structure interaction (SSI) has been introduced. The discussion on infrastructure resilience pave way for earthquake disaster management. Therefore, ongoing efforts being made in developing seismic design philosophy based on performance-based seismic analysis has then been elaborated. To this effect, static pushover analysis procedure has been explained. Subsequently, some of the advanced dynamic response modification devices have been presented in greater details, including different types of damping devices, tuned mass dampers, seismic base isolation systems, new seismic protection means, and their practical applications have then been shown. In case of the semi-active control devices, this chapter also has presented structural control algorithms and latest techniques thereof. Finally, the need for future research in earthquake engineering has been highlighted for benefit to students and researchers.

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1.1 Background and Opening Remarks

The frequency of minor to major earthquakes happening around the world is quite high as shown in Table 1.1. In so far as India is concerned, typically the northern part of India is severely affected because of the number of earthquakes happening every year, thus it mandates the civil infrastructure to be resistant to the earthquake-induced forces. From Table 1.1, it is clear that earthquakes with a magnitude of about five to six on the Richter scale happen almost daily somewhere or the other. The earthquakes occurring on the mainland have severe consequences on the manmade civil infrastructure (Fig. 1.1).

Therefore, civil engineers are naturally concerned with the safety of the infrastructure against earthquake-induced forces. As indicated in Fig. 1.2, a number of

Table 1.1 Frequency of occurrence of earthquakes [3]

Richter magnitude	About once every	Example
9.0+	20 Years	2011, Japan earthquake and tsunami
8.5–8.9	10 Years	2010, Chile earthquake and tsunami
8.0–8.4	1 Year	2008, China earthquake
7.5–7.9	3 Months	1906, San Francisco earthquake
7.0–7.4	1 Month	2010, Haiti earthquake
6.5–6.9	1 Week	1989, Loma Prieta, 1994, Northridge (LA)
6.0–6.4	3 Days	2014, Napa, 2011, New Zealand
5.5–5.9	1 Day	2007, Alum Rock (San Jose), 2011, Virginia

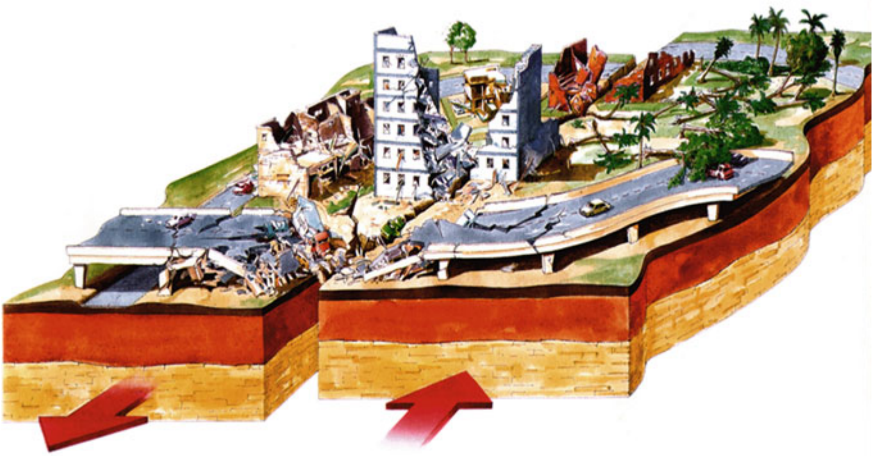


Fig. 1.1 Post-earthquake scenario [17]

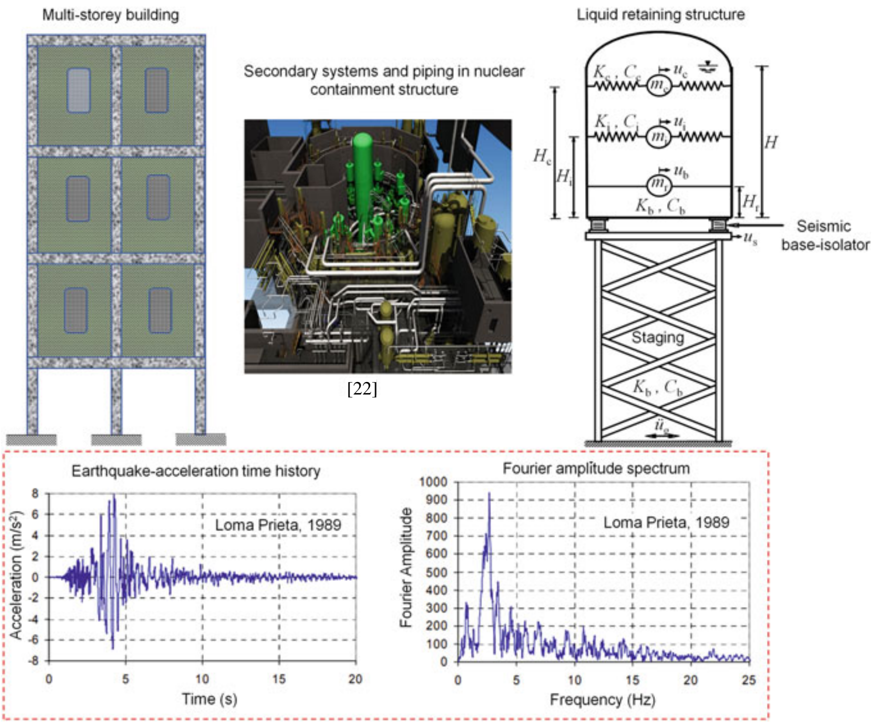


Fig. 1.2 Civil engineering infrastructure in day-to-day life

structures ranging from buildings to towers and various other types of structures are built for residential or commercial use. Further, nuclear power plants and wind turbine towers are vital for energy generation. The lifeline structures for water supply such as the liquid storage or containment tank and their corresponding pipeline and piping systems need to remain functional always. Hence, earthquake or seismic analysis and design of the civil infrastructure is an essential requirement. The analysis of structures subjected to earthquake ground motions is carried out either in the time domain, i.e., on a time scale, or in the frequency domain, i.e., on the frequency scale, where the loading function or the forcing function is the input given to the structures to determine their dynamic response.

Another important lifeline structure, as shown in Fig. 1.3, is the bridge, which is an essential component of the road and rail network for surface transportation. After any calamity, engineers must ensure that the bridges remain functional so that rehabilitation and rescue operations can be carried out effectively. Any loss of transportation infrastructure cannot be afforded during and after the earthquake. Any amount of downtime (out of service duration) for rehabilitation of these lifeline structures is highly undesirable because these lifeline structures play a vital role in the recovery of normalcy after the havoc caused by earthquakes. Thus, the earthquake

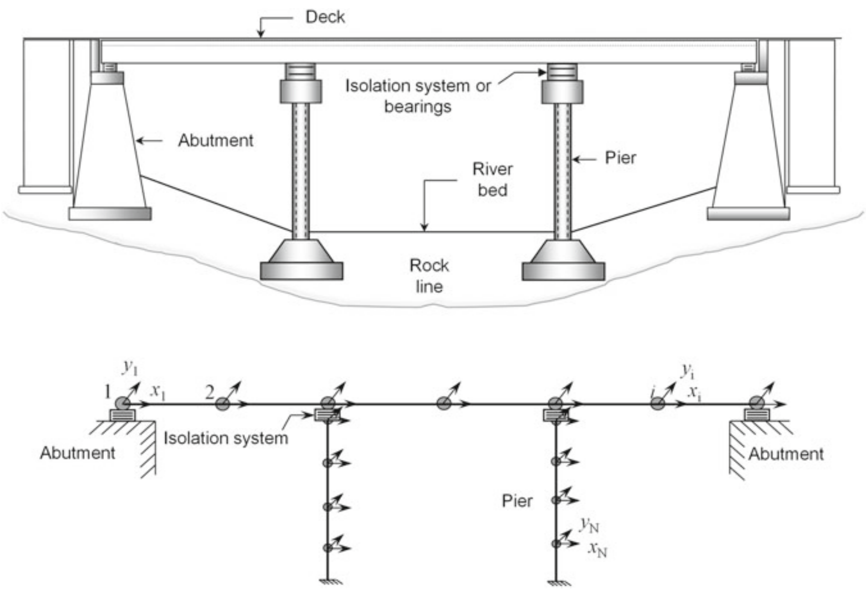


Fig. 1.3 Typical bridge structure [9]

resilience of bridges should be maximized under the anticipated earthquake-induced actions.

1.2 Expertise and Professionals Involved

As illustrated in Fig. 1.4, let us recall one of the most well-known stories heard in childhood about how six people perceived an elephant when blindfolded. For example, the blindfolded person who feels the ears of the elephant assumes that it is a fan. Another person observes the legs of the elephant and concludes that it is probably the trunk of a tree. Another examines the tail and thinks that it is most likely a rope, and so on.

On a similar note, earthquake engineering has various facets corresponding to different disciplines, as shown in Fig. 1.5; fortunately, it is well understood by engineers. It begins with the seismology or Earth sciences disciplines dealing with the origin of the earthquakes and their propagation. Then the geotechnical earthquake engineering typically deals with soil dynamics and the foundation design against earthquakes and considers the soil-structure interaction (SSI). However, in structural engineering, there are four major fields: earthquake-resistant (analysis and) design of structures, structural (dynamic or vibration) response control, performance-based earthquake engineering, and structural dynamics (nonlinear vibrations), all of which will be discussed in the later sections of the book. Both geotechnical and structural

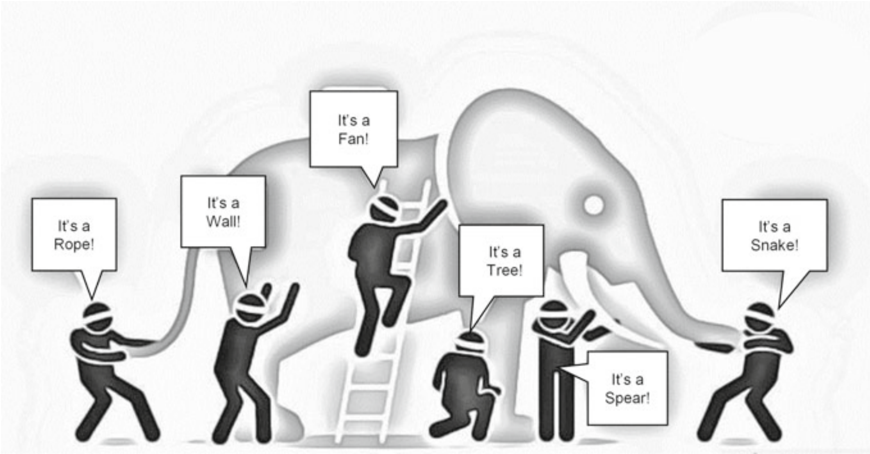


Fig. 1.4 An elephant and six blindfolded people

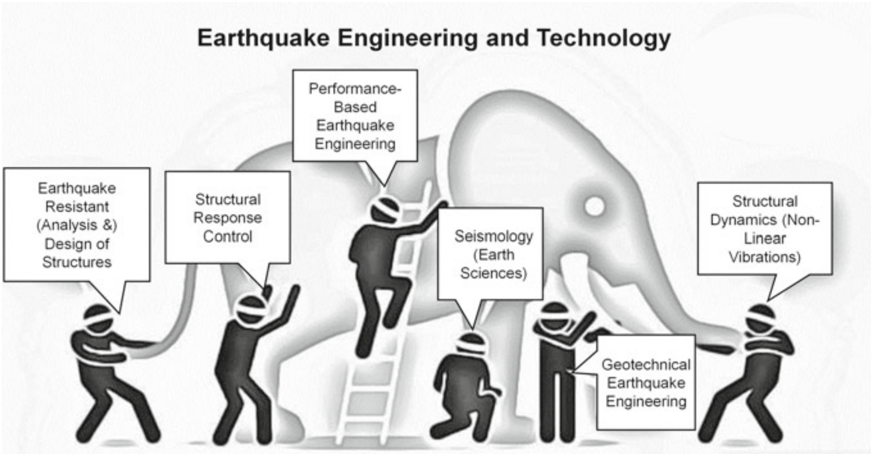


Fig. 1.5 Different disciplines in earthquake engineering and technology

engineers rely on the principles of structural dynamics (engineering vibrations) while solving the problems related to earthquake engineering.

It is important here to understand the fine difference between earthquake engineering and earthquake technology! In earthquake engineering, sound theoretical basis is adopted for all the studies conducted, whereas in earthquake technology, real-life applications of those theories are pursued. Thus, earthquake engineering has a rather theoretical orientation as against the applied orientation in earthquake technology. Earthquake engineering works on the development of new analysis and design methods, which are then implemented to solve specific technical problems in

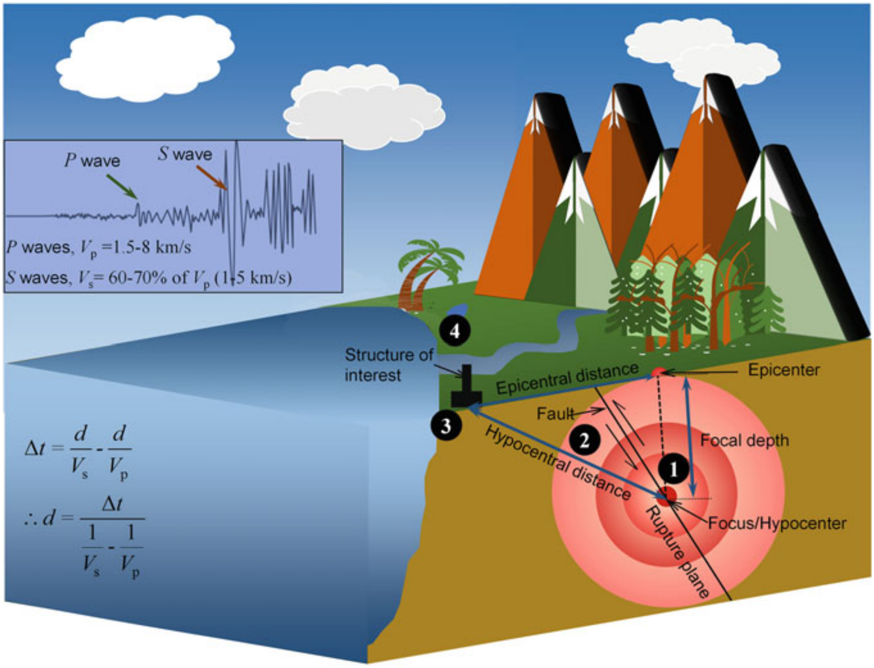


Fig. 1.6 Basic terminologies used in earthquake engineering and technology

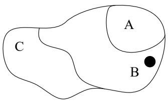
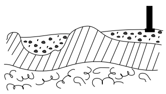
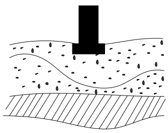
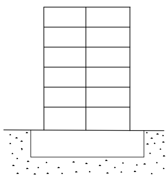
the technology field. Earthquake engineering focuses on research, development, and design; on the other hand, earthquake technology involves testing, construction, and field applications. Thus, technology is the practical application of engineering knowledge. In the present book, both aspects of earthquake engineering and technology have been construed with clarity.

Figure 1.6 shows the origin of earthquakes, where ① depicts the location of the fault where the fracture process initiates and rupture takes place, known as the focus or hypocenter. The seismic waves propagate in all directions from the focus. Eventually, the interest of structural engineers remains on how the structures of interest are affected due to the seismic activity. In Fig. 1.6, the focus of the earthquake is numbered as ① and the structure which is of interest is numbered as ④. Between those two points (① and ④), the competence of professionals in at least four different interdisciplinary fields is required, as indicated in Tables 1.2 and 1.3. Professional expertise in geology, seismology, geotechnical engineering, and structural engineering is required to deal with the various aspects of earthquake engineering and technology. Typically, the geotechnical and structural engineers work hand-in-hand in carrying out soil-structure interaction (SSI) studies. However, they require inputs from geologists and seismologists on the magnitudes and the site-specific characteristics (intensities) of the earthquake, how the seismic waves propagate to the considered site, and what kind of soil amplification occurs in the site. Once such information is delivered

Table 1.2 Professionals involved in various activities related to earthquake engineering and technology

Professional	Activities
Geologists	Tectonic movements
	Geological studies
Seismologists	Seismotectonics
	Microzonation
Geotechnical engineers	Soil dynamics/rock mechanics
	Geotechnical earthquake engineering
Structural engineers	Earthquake engineering and technology
	Civil built infrastructure

Table 1.3 Expertise in various disciplines of earthquake engineering and technology

Activity	Scheme	Authors	Information
Macrozonation		• Geologists	• Type of possible earthquakes
		• Seismologists	• Magnitude
			• Intensities
Microzonation		• Geologists	• Source position
		• Seismologists	• Magnitude
			• Intensities
			• Attenuation
			• Duration
Site condition		• Geologists	• Soil stratification
		• Geotechnical engineers	• Framing in soil type
			• Duration
			• Time history records
			• Design spectrum
Structure characteristics		• Geotechnical engineers	• Level of protection
		• Structural engineers	• General configuration
		• Mechanical engineers	• Materials
		• Architects	• Foundation type
		• Builders	• Structure type
		• Owner	

to the geotechnical and structural engineers, vibration responses of soil, foundation, and the structural system are evaluated, which facilitate the earthquake-resistant design of the structures. Further, structural engineers focus on how to effectively control or limit the structural response so that no catastrophic or devastating failure of the structure takes place. Finally, the aim is to ensure that the dynamic response of the structure meets all the specified requirements, such as functionality, safety, and serviceability requirements.

1.3 Seismology: Why Do Earthquakes Occur?

1.3.1 Genesis of Earthquakes

The genesis of earthquakes primarily involves the expertise of geologists to comprehend the phenomena associated. While seeking answers for why and how an earthquake happens, typically a comparison can be made with what happens in a hotpot during the preparation of tea! When tea is prepared inside the hotpot, convective currents are formed in the water, wherein the current flows from the bottom of the pot toward the top because of the difference in temperatures (temperature gradient) within the water. The heat is directly applied at the bottom of the hotpot, therefore the water in the lower portion gets heated and rises upwards; then the colder water toward the top gets displaced downwards. Following this, the temperature of the underlying water rises again, and the cycles continue to form the convective current. Almost similar to what happens inside the teapot, convective currents are formed within the Earth due to the different temperature zones of the Earth underneath the lithosphere. The temperature of the innermost liquid zone of the Earth is extremely high, while the solid outer soil surface which is in contact with the atmosphere is relatively colder, and semi-molten lava is present in between these two zones (Fig. 1.7). Due to the

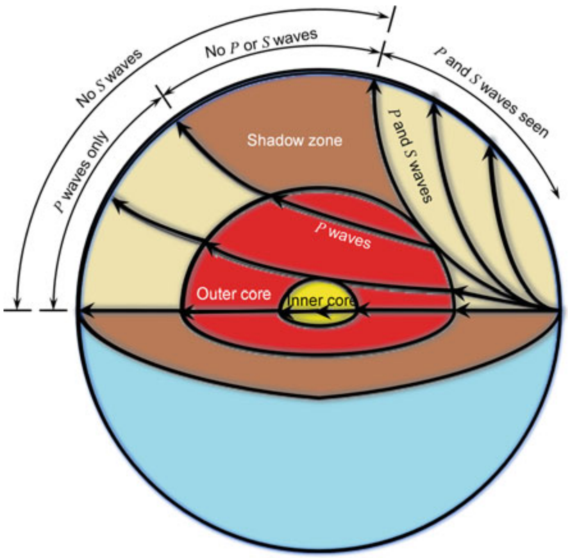


Fig. 1.7 Propagation of seismic waves

variations in the temperature, the convective currents are developed in the liquid and semi-molten zones, which in turn give rise to the various phenomena leading to the movement of the Earth's surface (in the form of movement of tectonic plates, as will be discussed later). Once these movements due to seismic activity take place at a point on the Earth's surface, different types of waves propagate on the Earth. Though in the next section the occurrence of an earthquake has been elaborated further, Fig. 1.7 shows how some of these primary and secondary waves are transmitted from the focus toward the different parts of the Earth's surface. The propagation of these waves depends on how far away the considered site is from the focus, and through which media it propagates. Interestingly, the shadow zone shown in Fig. 1.7 is formed due to the absence of any shear strength offered by liquid and semi-molten lava. Further discussion on the types of waves and the particle movements will be made subsequently to enhance the understanding of the origin of earthquakes.

Now, due to the cooling down of the Earth's crust, shrinkage takes place, thus the tectonic plates are formed, which are shown in Fig. 1.8. The tectonic plates keep on moving due to the formation of the convective currents within the molten or semi-molten lava underneath. Owing to the movement of the tectonic plates, particularly the movement of one plate across another along the plate boundaries, there are a number of earthquakes taking place round the year. If the attention is shifted to the Indian plate, as shown in Fig. 1.9, the Indian plate is continuously moving toward the Eurasian plate. This is the reason why a lot of earthquakes occur in the Himalayan region, and hence additional attention needs to be given across this region in the Indian peninsula.

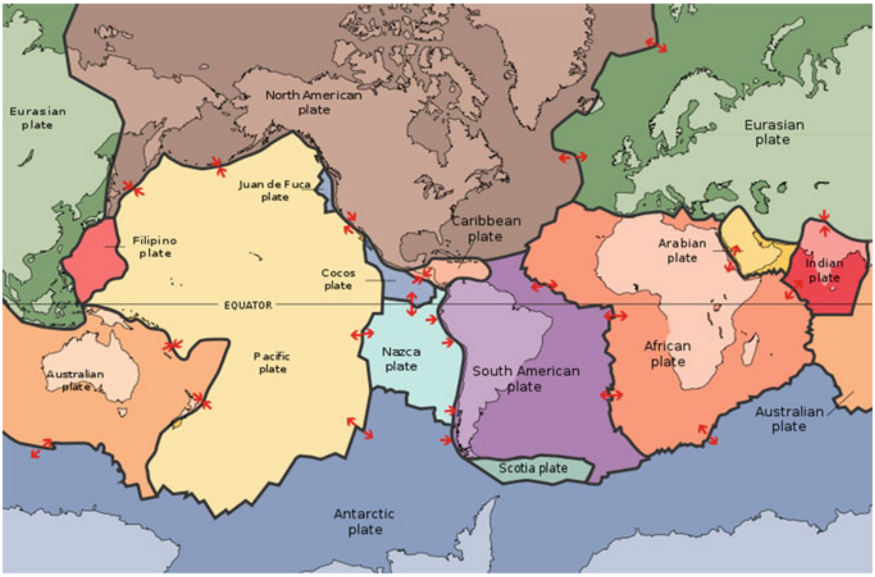


Fig. 1.8 Major tectonic plates on the Earth's surface [11]

Fig. 1.9 Relative movement of the Indian plate [6]

1.3.2 Finite Element Model for Seismic Activity in Indian Plate

Figure 1.10 shows a finite element (FE) model developed by researchers at the Indian Institute of Technology (IIT) Madras to study the seismic activity in the Indian plate. Researchers at different institutes in India, such as National Geophysical Research Institute (NGRI), are similarly investigating the plate tectonics and seismic activities caused in the Himalayan region due to the Indo-Eurasian plate interaction. All these researchers are trying to model how the Indian plate is moving toward the Eurasian plate and thus obtain the effects of this movement on the seismicity of the region. The FE analysis is carried out wherein the entire plate is modeled with certain specific support and boundary conditions. The FE simulations are able to accurately predict the movement of the Indian tectonic plate, which is found to be approximately 12 mm per year. Thus, the FE simulations could successfully give an idea about how

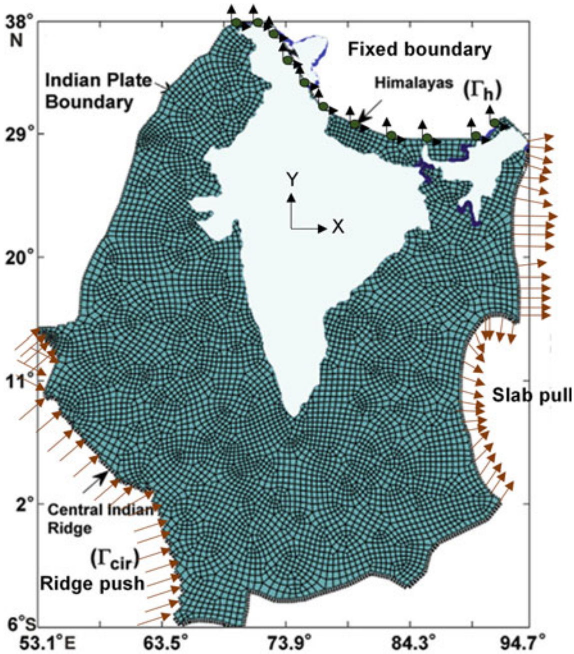


Fig. 1.10 Finite element (FE) model for seismic activity in Indian plate (modified from [8])

the tectonic plates are moving, and hence the nature of the earthquakes is anticipated. Subsequently, the same knowledge base is utilized to model the earthquake phenomenon, i.e., the ground motion is simulated. These days such simulations are being carried out by using advanced data analytics tools like machine learning or deep learning approaches. Either specific source modeling or blind source modeling approaches are followed for predicting the ground motions at a particular location.

1.3.3 Cause of Earthquake in Himalayan Subduction Zone

The primary cause of the earthquakes in the Himalayan and the Trans-Himalayan region of India is the formation of the subduction zone between the Indo-Australian plate and the Eurasian plate, as illustrated in Fig. 1.11. The Indo-Australian plate is continuously subsiding beneath the Eurasian plate, thus raising the heights of the Himalayan mountains in this process and causing earthquakes due to the abrupt motion at the boundaries of the two plates.

A number of other geotectonic features can also contribute to earthquake events. For example, there are some regions where an oceanic spreading ridge exists, or even there are some active volcanic areas, where earthquakes occur as a result of some of these geotectonic features, as shown in Fig. 1.12. At the plate boundaries, there

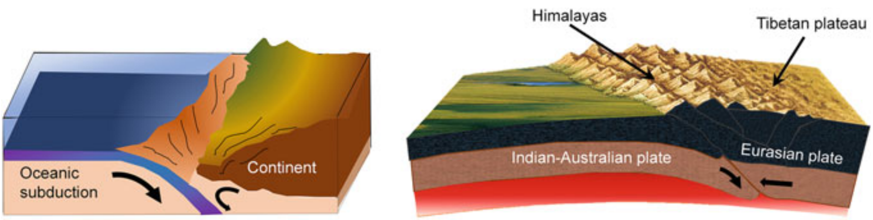


Fig. 1.11 Subduction zone between Indo-Australian and Eurasian plates

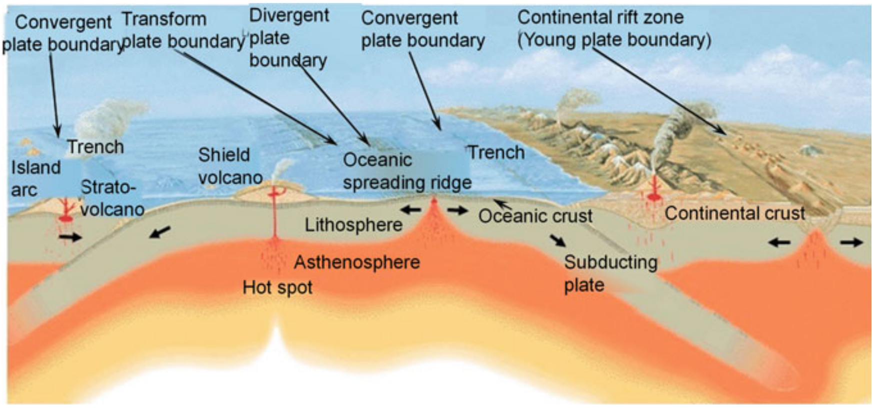


Fig. 1.12 Seismotectonic features [24]

are different kinds of movements taking place relative to each other, which are categorized as convergent, divergent, and transform boundaries. The movement of the plate boundaries is convergent when the plates are approaching toward each other. The movement of the plate boundaries is divergent when the plates are moving away from each other. When seen from the top, if the plate boundaries are moving in the transverse direction relative to each other, they are known as transform boundaries. All these seismotectonic features give rise to earthquakes having different characteristics based on how the accumulated energy is released in the process. Typically, seismologists study such features and determine the characteristics of earthquake ground motion and also determine the features of the wave propagation. Further, macro- and microzonation of the region facilitate the collection of ground motion data and its assimilation for strong motion studies.

As mentioned earlier, earthquakes originate due to the rupture at active fault locations when the tectonic plates move relative to each other. The tectonic movements build up strains in the rock masses. Typically, during these continuous tectonic movements, when the strain developed in the rocks exceeds the strain capacity (ultimate strength) at any fault location, the rocks break in a brittle manner, causing a sudden rupture and release of the accumulated energy. In this process, a large amount of energy is released. This is explained in geology using the elastic rebound theory. The exact location at which such a rupture on the fault location initiates is called focus (hypocenter) and its projection on the ground surface is called the epicenter. The distance below the ground surface between the epicenter and the focus is called the focal depth.

In order to protect the built environment, i.e., the civil infrastructure from the wrath of earthquakes, discerning the underlying physics involved in the genesis of earthquakes is crucial. The quantification of the energy released at the origin of the earthquake (i.e., at the focus) is made in terms of the magnitude of the earthquake, which is typically measured using the Richter scale. From Fig. 1.6, it is known that the distance from the epicenter to the location of the structure of interest is called the epicentral distance, and the distance from the hypocenter to the structure of interest is known as hypocentral distance. The intensity of ground shaking experienced at the structure of interest depends mainly on the epicentral distance. Hereby, the readers should note that the magnitude and intensity of the earthquake are two distinct features, and are required to be clearly distinguished and appropriately used in structural analysis and design. The magnitude of the earthquake is obtained from the energy released at the source of the earthquake, irrespective of the location of the structure of interest. On the contrary, the intensity of the earthquake depends on the location of the considered structure relative to the epicenter of the earthquake; intensity is indirectly related to the amount of energy released at the source. During the process of transmission of the seismic waves (body waves and surface waves) from the focus to the structure of interest, if the propagation is monitored then it can be used for giving an early warning. This is possible because the different seismic waves have varied speeds of propagation in the ground medium. An early warning to the residents is thereby given by calculating the difference in the time of arrival of the different types of seismic waves (Fig. 1.6). In the Indian context, sensors and instruments are installed by researchers from the Indian Institute of Technology (IIT) Roorkee, the sensors are placed ranging from the Himalayan region to the most populated northern cities such as the National Capital Territory (NCT) Region of Delhi. To understand the concept of prediction of major earthquakes, the different types of seismic waves are needed to be studied, as illustrated in Figs. 1.6 and 1.13.

There are broadly two types of seismic waves: body waves and surface waves. Further, the body waves are of two types depending on how the soil particles move. If the particles move in the form of compression and rarefaction, i.e., particles are moving in the direction of wave propagation, then such types of body waves are called pressure or primary (P) waves. However, in the shear or secondary (S) waves the particles move up and down relative to each other, i.e., the particles move in the transverse direction with respect to the direction of transmission of the seismic waves. In both these waves, the movement of particles is traceable in the cross-section of the medium through which they propagate.

Generally, the surface waves are most destructive to the structures; however, their propagation velocity is relatively much lower than that of the body waves, particularly that of the primary (P) waves. The primary waves propagate faster as compared to the secondary waves and the other surface waves. The surface waves are of two types: the Love waves, where the particles move sideways (when viewed from the top, it

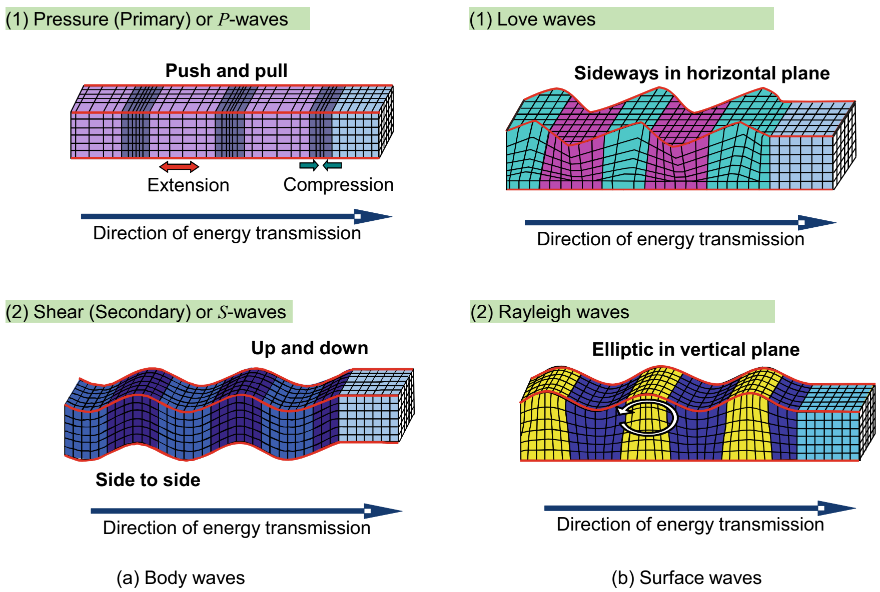


Fig. 1.13 Types of seismic waves [16]

looks like a serpentine motion), and the Rayleigh waves, where the particles move in an elliptical orbit (in the cross-sectional view). These characteristic information about the P and S waves are exploited for predicting or giving early warnings to the users or the citizens, even maybe on their cellphones or smartphones.

For example, in the earthquake time history shown in Fig. 1.6, the arrival of the primary and secondary waves at a location can be deciphered. The duration between the arrival of the P and S waves is calculated, and this information from at least three locations is used in locating the epicenter using the method of triangulation. Furthermore, the hypocentral distance can also be calculated by a similar assessment. Further, using this knowledge, a prior warning could be given to the residents of a heavily populated region, that a significant earthquake might be approaching, and eventually some invaluable lives could be saved.

Now, shifting our focus toward the structural design aspects. It is very important to consider what are the characteristics of the ground motions felt at a particular location (site) such as what should be peak ground acceleration (PGA) taken in the structural design. The ground acceleration experienced at a particular location decreases with the increase in the distance of the site from the epicenter (and even the focus); such a decrease is referred to as attenuation of the earthquake ground motion. Several ground attenuation relationships have been proposed by researchers for various regions. Certain ground motion attenuation relationships have been proposed for the Indian peninsular region as well. In Fig. 1.14, such attenuation relationships are shown which depict how the peak ground acceleration (PGA) in the horizontal direction

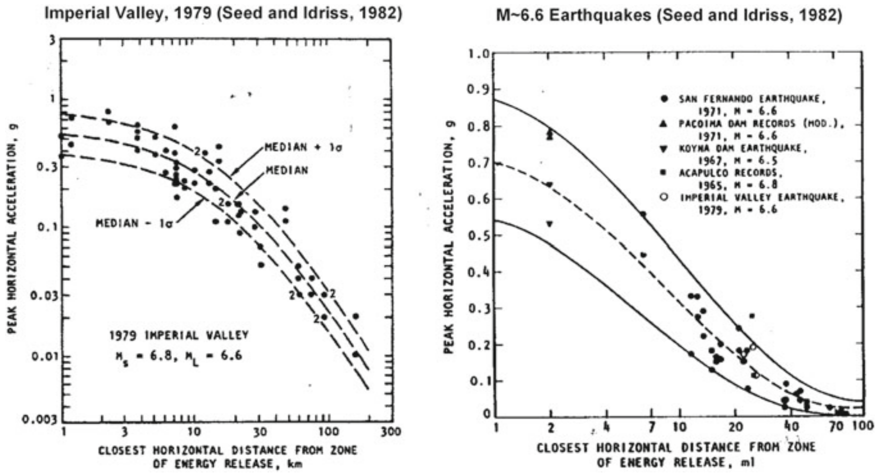


Fig. 1.14 Ground motion attenuation relationships [19], Seed and Idriss [23]

reduces away from the focus or the fault rupture location. Naturally, in the areas in closer vicinity of the epicenter, higher PGA is experienced as compared to that at farther distances.

For peninsular India, the attenuation relation for the PGA and the spectral acceleration (S_a) for the rock site is given by Raghukanth and Iyenger [19] as

$$\ln y = c_1 + c_2(M - 6) + c_3(M - 6)^2 - \ln R - c_4 R + \ln(\varepsilon)$$

where y is the ratio of spectral acceleration at bedrock level to acceleration due to gravity (S_a/g), M refers to the moment magnitude, ε and R denote the errors in the regression and hypocentral distance, respectively. The coefficients c_1, c_2, c_3, and c_4 are specified by Raghukanth and Iyenger [19].

1.4 Seismic Soil-Structure Interaction (SSI)

Now, switching over from the domain of geologists and seismologists to geotechnical engineers, the information given by the former is used by the latter to analyze site-specific earthquake ground motions. Based on the subsoil condition at a site of interest, the geotechnical engineers examine the effect of soil characteristics on the incoming earthquake waves. While obtaining the final input ground motion to the structure, the time period of the soil underneath and the stratifications or layer properties are important parameters of interest.

Usually, dynamic soil-structure interaction (SSI) needs to be considered during an earthquake, as well as the liquefaction potential of the soil should also be duly assessed. Different analytical or numerical approaches are followed for modeling the dynamic SSI, ranging from simplistic (approximate) to detailed (rigorous) modeling of the dynamic soil properties (Fig. 1.15a–d). In simplistic two-dimensional (2D) modeling, as depicted in Fig. 1.15a, spring-dashpot models can be employed for the deep foundations, by assigning suitable mass, spring coefficients, and damping coefficients, depending on the type of soil or rock underneath [25]. The modulus of subgrade reaction governs the dynamic parameters assigned in the spring-dashpot-type discrete modeling of the soil.

The various arrangements of the mass, springs, and dashpots in the discrete models (Fig. 1.15b) are rather simple to model in the commonly used commercial software packages such as SAP2000®, ETABS®, STAAD-Pro, etc. However, a detailed and accurate analysis for important structures, such as nuclear power plants, dams, or other strategically important structures, is conducted by either three-dimensional (3D) elastic half-space or continuum type of modeling approaches based on rigorous

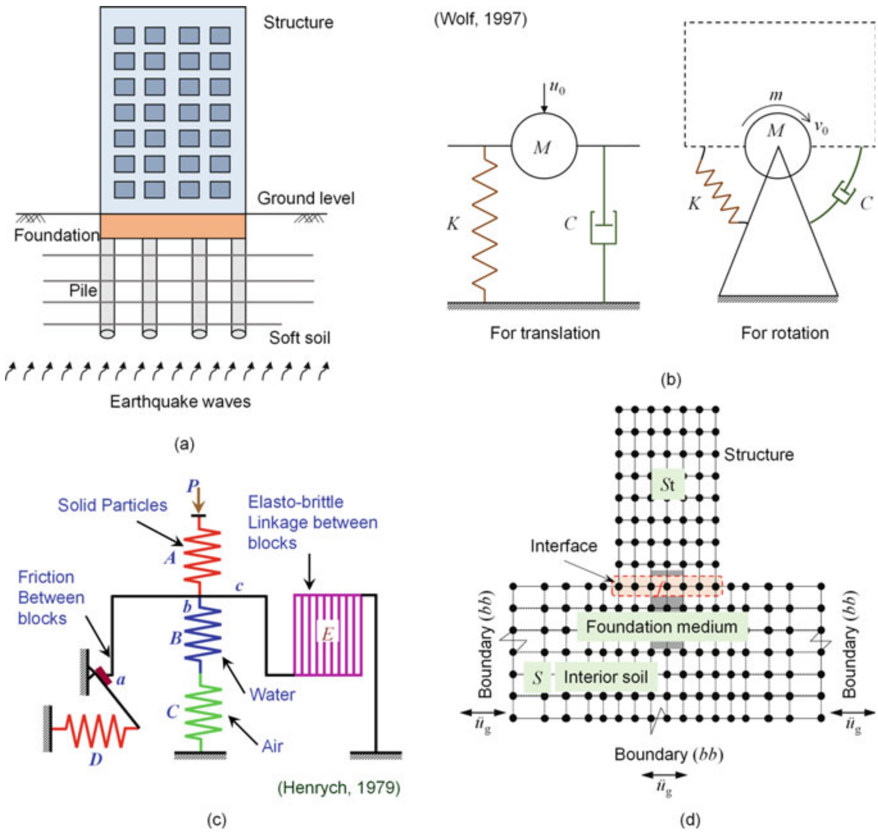


Fig. 1.15 **a** Seismic SSI in a multistory building with pile foundation, **b** spring-dashpot-mass models for translation and rotation, **c** rheological model for SSI, and **d** finite element (FE) model of soil-structure system for direct analysis methods

3D FE analysis. One of these models is shown in Fig. 1.15d, wherein the SSI problem is modeled using the FE approach. Among the simplified and detailed models, some other soil modeling approaches address the shortcomings of these two models, in terms of accuracy and computational time. These mathematical models proposed by geotechnical engineers consider various phenomena taking place within the soil mass, such as inter-particle friction, elastoplastic interactions, etc. Using this modeling approach, the behavior of the foundation in case of any earthquake event can also be studied. Moreover, individual solid particles of the soil are also modeled and the interaction between them is considered depending on whether it is a friction interaction or an elastoplastic interaction (Fig. 1.15c) [5]. Saturated soil exhibits a different response than unsaturated soil; therefore, some of these models take moisture content and air content into consideration appropriately. Nevertheless, spring-dashpot models are still more popularly used in day-to-day structural designs when SSI is to be accounted for.

1.5 Earthquake Disaster Management

Before discussing the role of structural engineers on how they deal with earthquake ground motions and the forces that are induced on the structures thereby, the concepts of earthquake disaster management should be known. While delving into risk-based earthquake disaster management, some terms like hazard, vulnerability (fragility), and risk are introduced. Hazard may be defined as the probability of occurrence of the earthquake at a particular location, whereas the return period of an earthquake is the duration in which the same intensity of the earthquake may occur again. Figure 1.16 shows seismic hazard curves, which are simply the plots of the annual probability of exceedance versus the earthquake intensity felt, defined in terms of PGA. Based on this relation, a distinction is established between low seismic hazard and high seismic hazard.

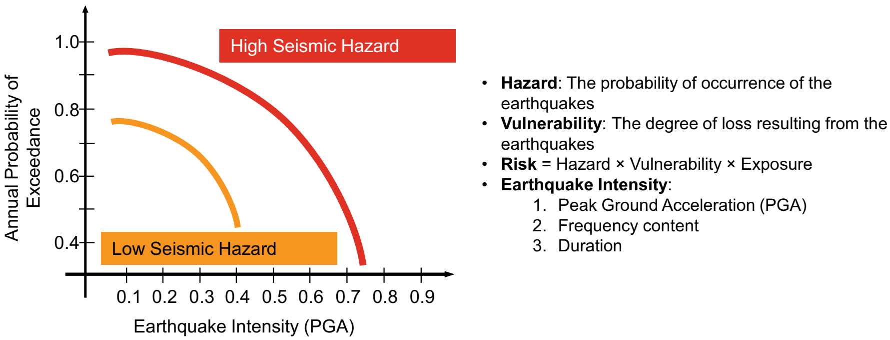


Fig. 1.16 Basic terminologies used in earthquake disaster management

The intensity of an earthquake is basically defined in terms of peak ground acceleration (PGA), frequency content, and the duration of the earthquake. While generating the seismic hazard curves, the PGA has been used as an intensity measure here; however, the other intensity parameters could also be used to obtain similar plots. Generally, for important structures, the earthquake intensity measure quantified in terms of frequency content is more suitable. The intensity of an earthquake based on duration accounts for the number of cycles of repetitive loading applied to a structure; therefore, for long-duration earthquakes, it is a necessary intensity measure. However, currently, the Indian codes mostly deal with the PGA as an intensity measure of the earthquake, and quite appropriately so.

Subsequently, the vulnerability of the structures for varying earthquake intensities is determined in terms of PGA, as well as the mode of failure that may occur is predicted. If the probability of failure of the structures is plotted against the intensity measure of the earthquakes (PGA, in this case), these are known as the fragility curves, indicating how fragile the concerned structures are to a given intensity of an earthquake. Based on the fragility curves, the seismic risk to which the civil infrastructure is exposed can be calculated. Such seismic risk calculations are conducted by initially developing the fragility curves of the considered structure (also called vulnerability curves sometimes), and subsequently calculating the hazard at the specific site and also exposure level. To sum up, the plots of the probability of failure of the structure versus the earthquake intensity (PGA or spectral acceleration) are the fragility curves. There is another school of thought in the research community which considers the vulnerability curves different from fragility curves; in their opinion, the former also accounts for the risk and cost implications.

Eventually, in achieving the ultimate objective of an earthquake resilient society, earthquake hazard mitigation strategies are adopted based on the seismic risk posed by the considered structure. In seismically developed countries such as Japan and the west coast of America, extensive studies have been conducted to quantify the seismic risk posed by the built environment and also on the measures to mitigate the earthquake hazard. One of the reasons for calculating fragility or vulnerability is to quantify the failure probability of a particular structure in a compact form. Though fragility and vulnerability are generally used interchangeably, to be specific the structural damage is considered in the former whereas monetary and human losses are considered in the latter.

For routine earthquake-resistant design of structures, Indian Standard (IS) 1893 (Part 1 to 5): 2016 [1] provides procedures and guidelines. Keeping the debate aside, whether this is a code or a standard, IS 1893 mainly recommends using a linear response spectrum, either for the seismic coefficient method or for the complete response spectrum analysis of structures. The response spectrum plot is obtained by conducting dynamic analysis on a linear single degree of freedom (SDOF) system. The peak displacement response of an SDOF system with a specific time period is determined when the SDOF system is subjected to a particular earthquake. By gradually increasing the time period of the SDOF system, ranging from relatively stiffer to more flexible structures, the peak structural displacement responses could be plotted against their corresponding time periods, this is known as the displacement spectrum curve for the specific excitation. Further, each ordinate of the displacement spectrum could be multiplied by their corresponding natural frequency to obtain the pseudo velocity spectrum. Similarly, the pseudo acceleration spectrum is derived by multiplying the square of the natural frequencies with the ordinates of the displacement spectrum curve. Avoiding the computational burden of evaluating the actual velocities and acceleration of all the considered structural systems, the pseudo velocity spectrum and the pseudo acceleration spectrum plots provide a good estimate of the energy dissipated by the system and the base shear attracted by the system, respectively. A valuable modification to the conventional response spectra curves is the tripartite plot, which combines all the three, acceleration (A), velocity (V), and displacement (D) response spectra into a single plot. One such example of a tripartite plot of the Imperial Valley, 1940 earthquake is shown in Fig. 1.17. Note that the term spectra is the plural form of spectrum. In the case of actual earthquake ground motions, the response spectrum has several peaks and valleys (i.e., it is not smooth). Hence, the response spectra are obtained for various site-specific earthquakes, and the plots are normalized and averaged to convert those into a single curve. Further, the graph is smoothened and scaled according to the different soil conditions, to obtain the design response spectrum as given in IS 1893 (Part 1) 2016 (refer to Fig. 1.18).

The Indian seismic code (Part 1 of IS 1893: 2016) has provided the design response spectra depending on the type of rock or soil available at the particular site. From Fig. 1.18, it can be observed that there are two types of design spectra available in the code, one is for the equivalent static approach and the other for the response spectrum approach for earthquake analysis of structures. The equivalent static approach is rather simple, which is an extension of the past practice followed in an earthquake engineering discipline. Structural engineers used to take about 10% of the total weight or seismic weight of the structure in the horizontal direction for designing

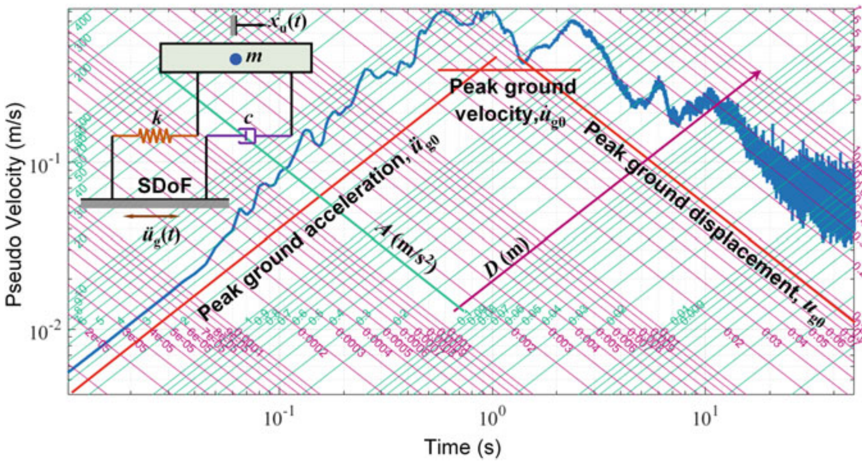


Fig. 1.17 A-V-D response spectra on a tripartite plot with 5% damping ratio (Imperial Valley, 1940)

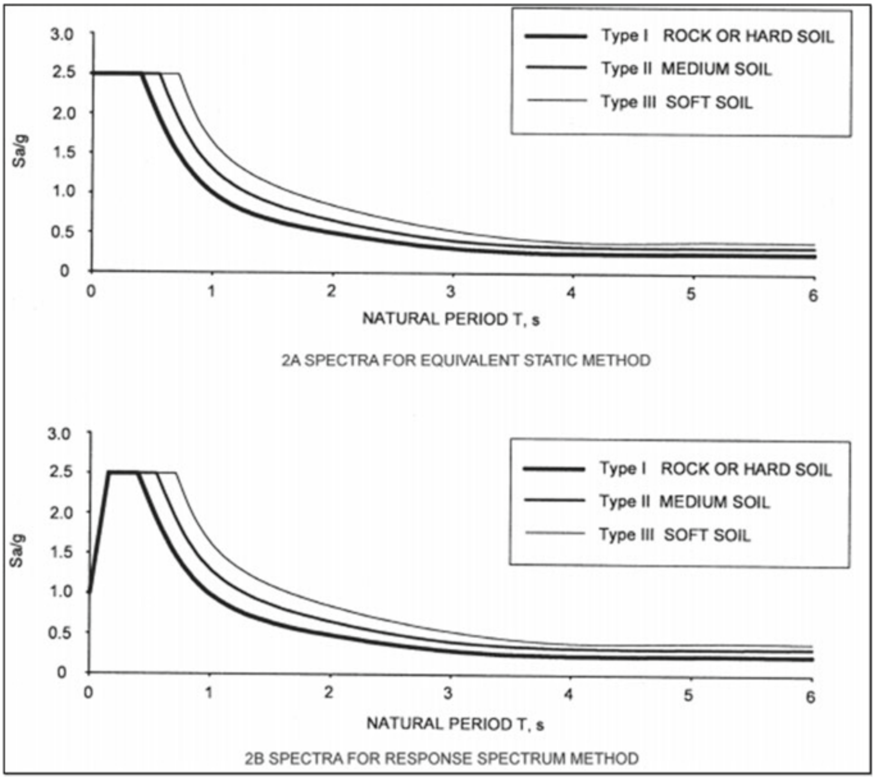


Fig. 1.18 Design response spectra as per IS 1893 (Part 1): 2016

lateral load resisting elements, such as columns. An improvement from such a rule of thumb approach, now the seismic base shear is calculated using the equivalent static method by considering only the first mode of vibration of the structure. The seismic coefficient method (SCM) is one such equivalent static approach for earthquake-resistant design of structures, as suggested in IS 1893: 2016 (Part 1). Nonetheless, some researchers even consider it to be a dynamic approach because the response spectra are obtained by conducting dynamic analysis.

For certain important structures such as the nuclear containment structures, simplified seismic analysis based on the generic spectrum or the design spectrum is not suitable. In that case, a site-specific design spectrum must be used, and a detailed structural model is prepared to account for all nonlinearities. From site-specific design spectra, spectrum-compatible earthquake time histories are generated to conduct nonlinear time history analysis of the important structures to evaluate their seismic response. For obtaining the site-specific design spectra, certain methods are available, for example, GovindaRaju et al. [4] have suggested one such method. Herein, important guidelines to calculate the site-specific design spectra are provided (Fig. 1.19a) by taking into account the shear wave velocity profile, the modulus reduction curve,

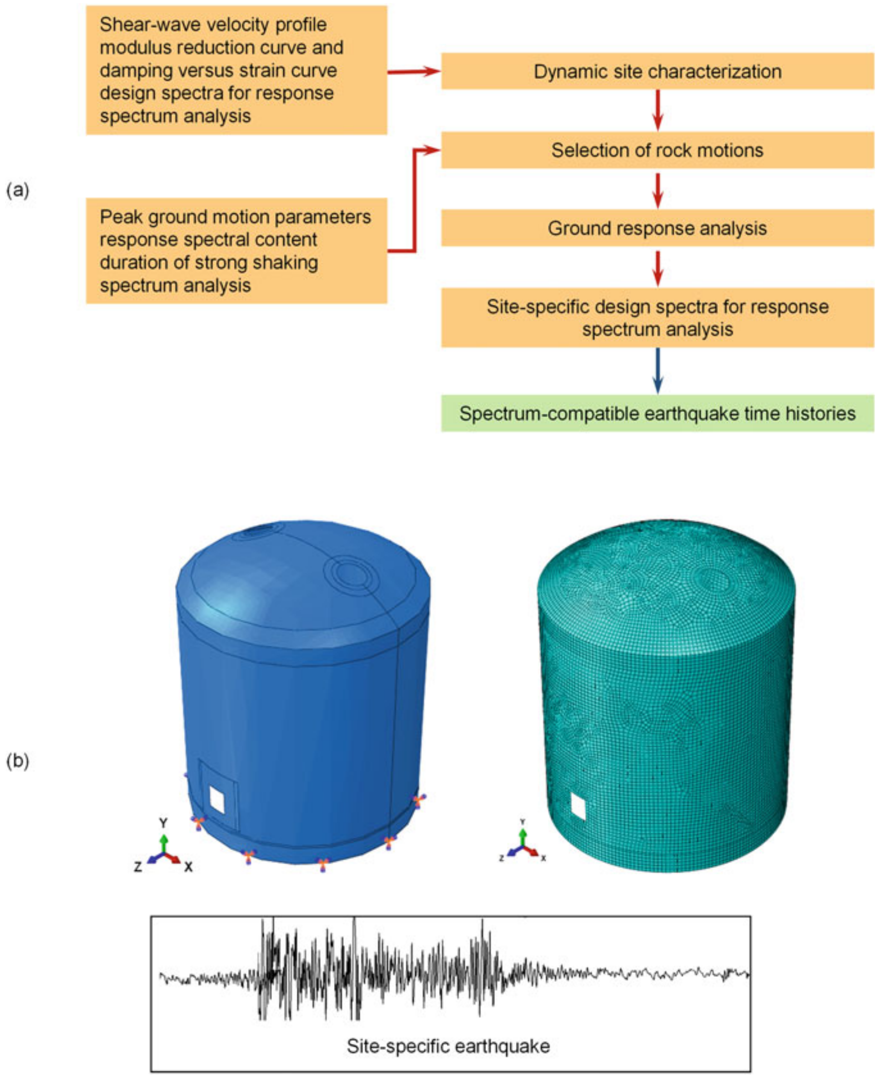


Fig. 1.19 Site-specific earthquake response analysis for important structures: **a** site-specific design spectrum and spectrum-compatible ground motions, inspired from Govinda Raju et al. [4], **b** inspired from [18] studies

and the damping ratio curve of the soil profile. Peak ground motion parameters, response spectra content, duration of strong shaking, and spectrum analysis of the considered ground motion are used to select the strong motions. Eventually, the site-specific design spectra for response spectrum analysis are derived. Further, this site-specific response spectrum could be utilized to carry out the spectrum-compatible earthquake nonlinear time history analysis. Synthetic or artificial earthquake ground motions can be generated easily by using some tools; for example, the WAVGEN code developed at the Indian Institute of Technology (IIT) Kanpur could be used. Furthermore, seismic fragility estimation of a containment shell has been carried out at the Indian Institute of Technology (IIT) Bombay based on the formation of through-wall cracks using a three-dimensional (3D) finite element (FE) model. Conducting stochastic studies on earthquake response of structures in the time domain demands the generation of a large number of synthetic ground motions with certain statistical distribution or attributes. Hence, such practical limitations prohibit using these state-of-the-art seismic analysis and design techniques in routine design offices. For routine seismic design of structures, simplified albeit conservative design methods are to be relied upon as advocated by codes and standards.

Although important structures require nonlinear time history analysis, currently the Indian code does not provide much details about it. In IS 1893: 2016, three approaches to earthquake-resistant design of structures are mentioned: (a) seismic coefficient method (SCM), (b) response spectrum method (RSM), and (c) time history (TH) analysis method. However, only the first two methods, SCM and RSM, are given in detail within IS 1893 (Part 1): 2016. The SCM is basically an approximate method which adopts a static approach to solve a dynamic problem. The SCM considers only the fundamental (first) mode of vibration of the structure to evaluate its seismic response. In the SCM, the time period of the structure is estimated based on some empirical formulas, which depend on the characteristics of the structure such as the type of infill used. Since the SCM is an approximate approach, naturally the method is made more conservative. The SCM is quite popularly used for the earthquake-resistant design of routine structures. A better, theoretically improved approach as compared to the SCM is the response spectrum analysis (RSA) or response spectrum method (RSM), which also accounts for the higher vibration modes. For classically damped (considering damping to be proportional to the mass and/or stiffness of the structure) linear structures, the RSM is quite suitable, especially when lumped mass modeling has been adopted. All the modes of vibration which contribute toward greater than 90% of the modal mass participation can be considered to be contributing to the total dynamic response of the structure. Nonetheless, even in the RSM, the superposition of the modal response is statistical and thereby approximation is adopted inherently.

The RSM approach is based on the linear response spectrum, therefore it is not suitable for the nonlinear structures. For non-classically damped structures, where external damping may have been added, the RSM is unsuitable, unless the desired nonlinear response spectrum is made available. In such cases, the nonlinear time history analysis (NLTHA) is the most suitable and advantageous method to evaluate the seismic response of the structure. The different kinds of advanced engineering materials used in structures introduce nonlinearity to the structure, and hence suitable nonlinear time history analysis must be carried out. In the case of the nonlinear time history analysis, diverse material models or behaviors can effectively be modeled while taking into account non-classical damping, i.e., including the externally added damping. Moreover, the geometric nonlinearity and large displacement effects can be suitably accounted for while conducting the NLTHA. When higher degrees of nonlinearity exist, such as in the case of the pounding of structures, the NLTHA is adopted suitably. In seismic pounding of adjacent (closely spaced) structures, the structures collide because of oscillation in opposite directions, i.e., out-of-phase, thus the structures hit each other at different time instances and at different locations. Thereby, the contact behavior of the two structures after pounding can be appropriately modeled with smaller analysis step sizes. The NLTHA can also account for possible spatial and temporal distributions of inelasticity within the structure.

Figure 1.20 depicts a simplified dynamical model of a nonlinear structure where the stiffness or damping of the structure could be nonlinear and this nonlinearity can be accounted for by conducting a nonlinear time history analysis. For important structures, a finite element-based (FE) modeling approach is adopted along with spectrum-compatible time history of the earthquake ground motion, and the problem is solved using either numerical integration methods or state-space analysis method. Here, as the equations are nonlinear, a sound numerical approach is required for conducting the time history analysis. Some of the commercially available software such as SAP2000®, ETABS®, etc., offer nonlinear response history analysis facilities. Future versions of the Indian standards are expected to give more details

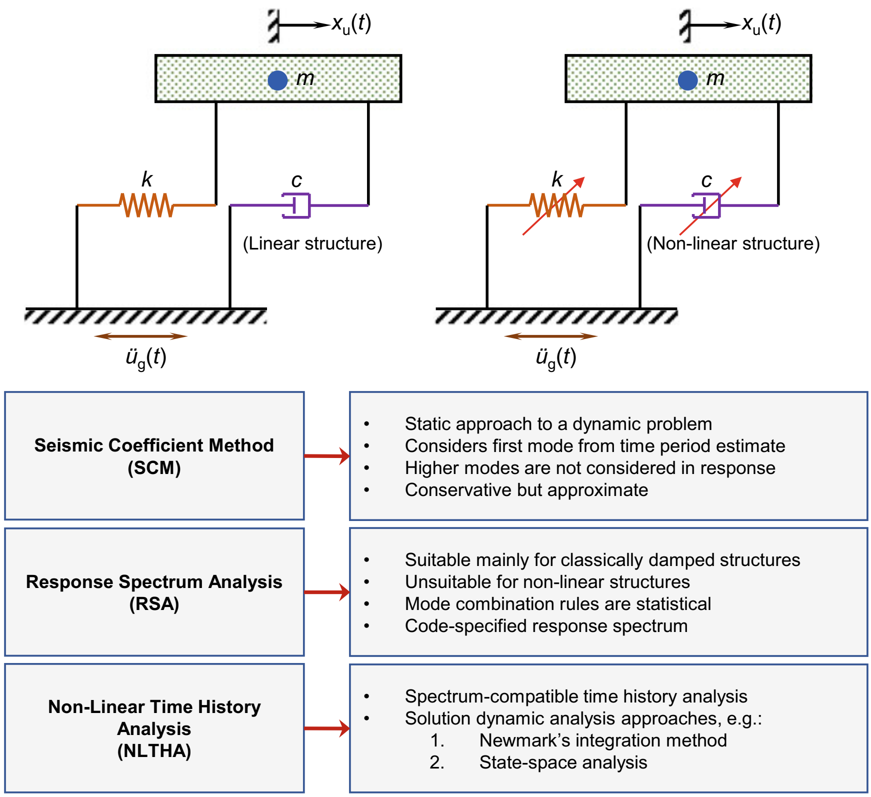


Fig. 1.20 Different analysis techniques for linear and nonlinear time history analysis

and recommendations on how to conduct the nonlinear response history analysis of structures.

1.6 Seismic Design Philosophy: Performance-Based Seismic Analysis

Performance-based earthquake engineering (PBEE) is an emerging discipline in earthquake analysis and design of structures. Performance-based seismic design (PBSD) of structures is a new philosophy which is being followed in some of the seismically advanced countries. The upcoming version of the IS code on seismic design of structures will also have provisions/guidelines concerning performance-based seismic design (PBSD). However, it is also believed that the current Indian Standard (IS) code accounts for some of the performance levels which are a part of PBSD indirectly. To understand the PBSD, it is important to know about the seismic performance levels, which in turn depend on the damage states of the structure. Figure 1.21 explains the different damage states of a structure pictorially. If the considered building remains operational with no damages after being hit by a major earthquake, and all the systems are fully functional even during the earthquake, then it is called an operational level performance. No (or negligible) damage is experienced by the structure in the seismic performance of operational (O) level, i.e., continued functionality. In the next case, the structure is subjected to a more severe earthquake event than that at the operational level, and some minor damage is seen in the building which is serviceable and does not jeopardize the immediate use of the structure. Then,

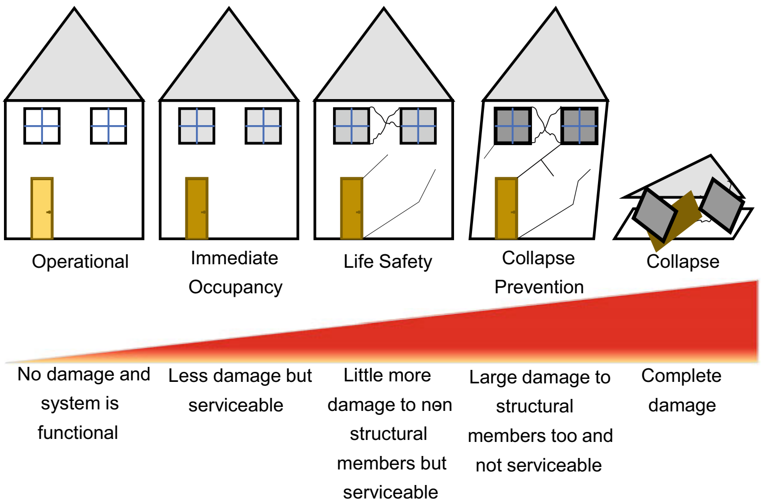


Fig. 1.21 Damage states and performance level thresholds

the structure can be immediately occupied after the earthquake event and it is known as the immediate occupancy (IO) category in the performance-based seismic design. With further increased intensity of an earthquake, more damage is experienced by the structure, i.e., some non-structural members might have failed, which calls for repair work. Nevertheless, life safety has been ensured; hence, this seismic performance level of the structure is called the life safety (LS) level. In the fourth damage state of the structure, severe or large damages are expected, even in the lateral load resisting elements, such as the columns and shear walls, due to the increased intensity of the earthquake. In this case, it may not be advisable to strengthen or rehabilitate these structural elements after the earthquake; however, since sudden collapse of the structure is prevented, it is known as the collapse prevention (CP) performance level. Eventually, for an earthquake of intensity significantly higher, complete damage and collapse of the entire structure might occur; however structural design needs to preclude it (beyond considered design level event).

In the PBSD, the major three categories of damage states are considered: light damage, moderate damage, and severe damage; and, correspondingly, immediate occupancy (IO), life safety (LS), and collapse prevention (CP) seismic performance levels of the structures are considered. Depending upon the type of structure being designed and its intended use under the design basis of the earthquake, the IO level, LS level, and CP level are decided as seismic performance. For example, a hospital building should at least satisfy the IO level of seismic performance, if not the O level. In order to ensure that a structure meets these seismic performance levels, depending on the anticipated earthquake ground motion it experiences, nonlinear analysis is conducted by subjecting the structure to the different levels, i.e., different intensities of an earthquake. The outcome of this analysis is seismic performance evaluation of the structure, which is possible by conducting a pushover analysis.

1.7 Static Pushover Analysis

There are two different ways of conducting static pushover analysis: (a) pushing the structure laterally by applying some forces and monitoring the displacement experienced, or (b) applying some displacement in a specific manner and monitoring the force generated, as shown in Fig. 1.22. The first approach is called the force-based approach and the second one is called the displacement-based approach. Conventionally, the force-based approaches have been followed, however nowadays displacement-based approaches are becoming more popular due to their advantages over the former. Displacement-based approaches are suitable for capturing the post-peak response, particularly for the highly inelastic behavior of the structure. In static pushover analysis, horizontal forces are applied (quasi-statically) in a predefined pattern on the structure, and load–deflection curve is obtained for the entire structure. For example, the distribution of the horizontal load applied to the structure, to conduct the pushover analysis, could be similar to the first mode shape. For the

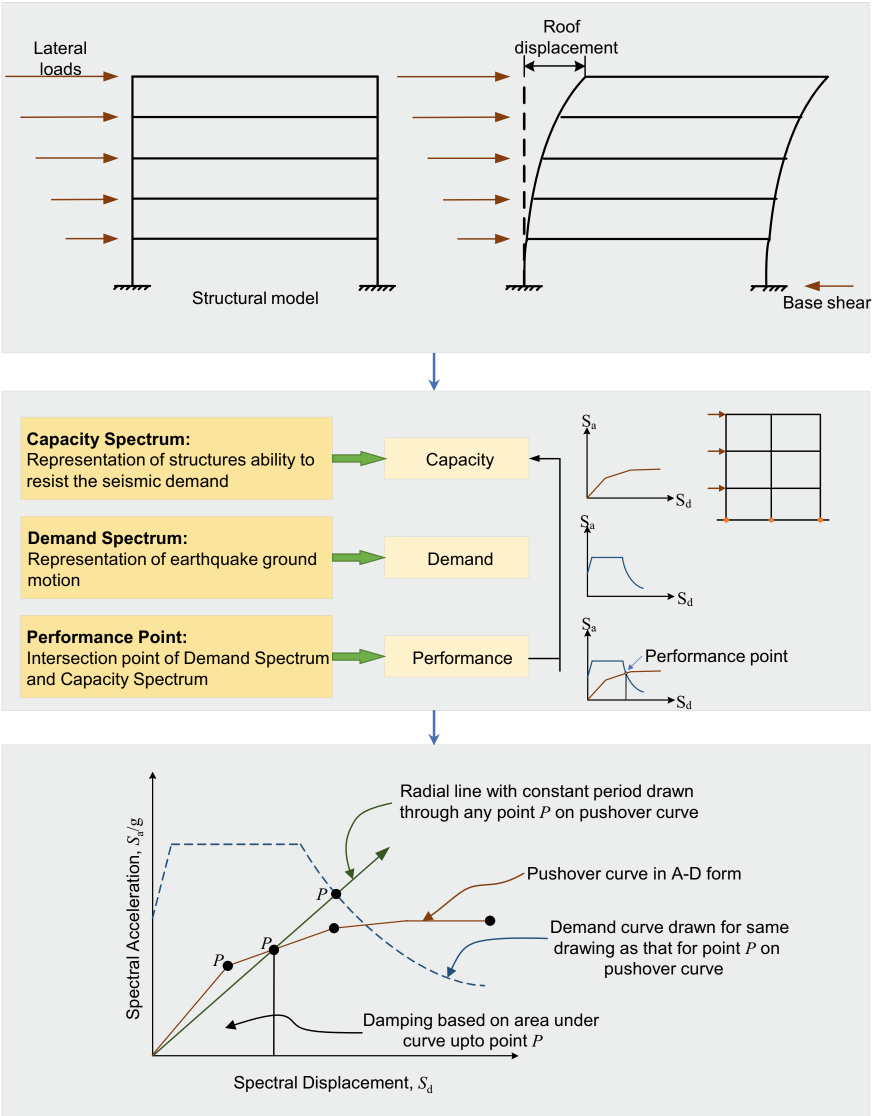


Fig. 1.22 Static pushover analysis [7]

applied horizontal load, the peak deflection is calculated and the load–deflection curve is plotted (Fig. 1.22).

In Fig. 1.22, a multistory building is applied with a certain pattern of horizontal forces and the corresponding displacement of the building is noted. Then, gradually the applied horizontal load is increased and with each increment the deflection of the building is noted. At a particular level of horizontal loading first hinge forms in the structure. With a further increment in the horizontal load, subsequent hinges keep forming until the collapse mechanism is formed, i.e., the structure becomes unstable. However, for large structures, only a few initial hinge formations are adequate to conclude the analysis. In this process of lateral pushing, the structure experiences high inelastic deformations, and a nonlinear load–deflection curve is obtained, either through a force-based approach or a displacement-based approach. This nonlinear load–deflection curve gives an idea about the seismic load-carrying capacity of the entire structure, hence it is called a capacity spectrum. It is a representation of how well a structure can resist the seismic forces which are induced in it. The load–deflection curve is converted into a spectral acceleration (S_a) and spectral displacement (S_d) curve, by dividing the applied horizontal force by the seismic or inertial mass, as the inertial force is a product of mass and acceleration.

The response spectrum of an earthquake is generated by plotting peak responses of SDOF systems with varying time periods and constant damping. The acceleration response spectrum plotted for an earthquake in terms of S_a and S_d (refer to Fig. 1.22) is an indication of the seismic demand on a structure. Hence, this curve is also called the demand spectrum, signifying the requirement imposed from the earthquake-induced loading on the structure. The two curves, capacity spectrum and demand spectrum, are then overlapped, and their intersection is called the performance point. Such overlapping of the two curves, i.e., capacity and demand spectrum curves to examine the seismic performance is shown in the spectral acceleration versus spectral displacement curve, shown in Fig. 1.22. Based on this performance point obtained, the seismic performance of a structure under consideration can be assessed, as to whether it meets the immediate occupancy (IO) criterion, life safety (LS) criterion, or the collapse prevention (CP) criterion (refer to Fig. 1.21).

Nowadays, performance-based seismic design (PBSD) is carried out popularly through pushover analysis, especially for designing some of the important structures. Nevertheless, probabilistic seismic vulnerability techniques or design approaches have been introduced and practiced based on an enhanced understanding of the behavior of structures under seismic actions, and also taking into consideration the probability of occurrence of structural failures due to the earthquake-induced loading. However, codes and standards are yet to completely adopt such latest probabilistic seismic assessment approaches. The seismic analysis of structures seen thus far in the codes are majorly deterministic (possibilistic); however, earthquake events are random in nature as well as the structural properties are inheretntly uncertain. For instance, the characteristics of an earthquake and its probability of occurrence are unpredictable in nature; every earthquake event is different from the other, and during each earthquake every structure experiences it differently. On the other hand, the material and geometric properties of the structural members are also uncertain. For example, the grades of concrete and steel considered in the design of the structures may not be precisely the same as the grades of concrete and steel achieved/used in the field; such variation in the properties needs to be considered through modeling the uncertainties stochastically. Seismic analysis and design approaches are being proposed for accounting some of these randomness in the ground motion and the

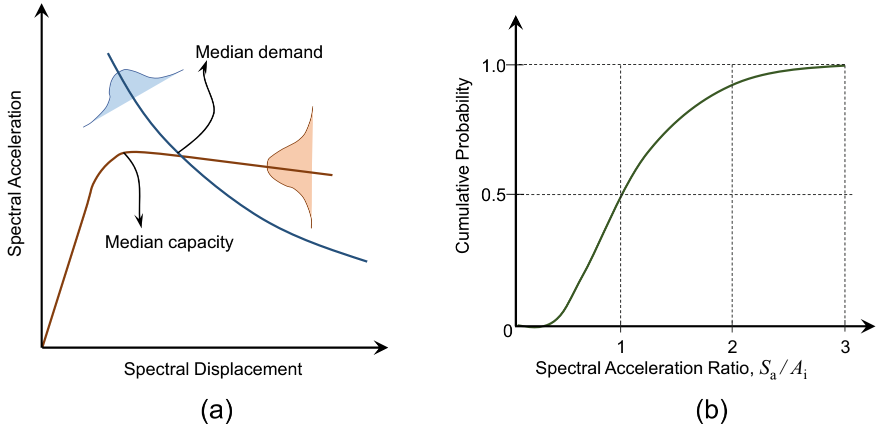


Fig. 1.23 **a** Capacity-demand acceleration-displacement spectra with randomness and/or uncertainty in structural behavior and ground motion response, **b** normalized fragility curve

uncertainty in the structural response. By considering such probabilities, the spectral acceleration and spectral displacement curves can be plotted with some probabilistic distribution, which means that there is a particular range in which demand and capacity vary, as seen from the two curves in Fig. 1.23a. Thus, seismic performance can be evaluated with a certain confidence level depending upon the structural and excitation parameters. The probability of failure of a structure, i.e., the chances of the structure reaching a particular limit state of failure, is calculated with the increasing peak ground acceleration (PGA) or spectral acceleration (S_a); the curve obtained as such is called fragility curve as shown in Fig. 1.23b. The probability of failure increases as the intensity of earthquake excitation increases, which is expressed via some intensity measure (IM) such as PGA or S_a.

Further, in Fig. 1.24, cumulative probability or probability of failure of the structure is obtained when the intensity measure (IM), i.e., either PGA or S_a is increased. Figure 1.24 also shows the relation of the fragility curves with the earthquake return period and confidence interval. Further, it is observed from the fragility curves (Fig. 1.24) that as the intensity of the earthquake increases, the probability of failure also increases. In the construction of a fragility curve the cumulative probability density function is expressed by

$$F(S_a) = \Phi\left[\frac{1}{\beta_c}\ln\left(\frac{S_a}{A_i}\right)\right]$$

where Φ = standard log-normal cumulative distribution function; S_a = spectral acceleration amplitude; A_i = median spectral acceleration necessary to cause i^{th} damage to occur; β_c = normalized composite log-normal standard deviation, incorporating aspects of uncertainty and randomness for both capacity and demand.

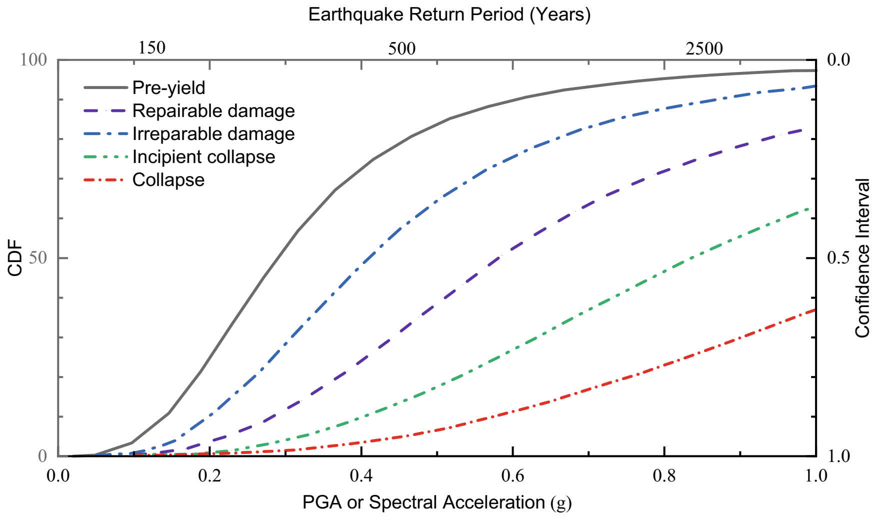


Fig. 1.24 Fragility curves for ductile design [12]

Naturally, the probability of some minor repairable damages to occur is higher as compared to the occurrence of irreparable damages or incipient collapse, especially at lower IM. In performance-based engineering, such fragility curves are extremely important to decide the damage states corresponding to the performance levels. Furthermore, some of the advanced techniques or latest innovations in structural earthquake engineering are employed herein to reduce the probability of failure even at high-intensity levels of earthquake and account for the uncertainties. Such seismic fragility analysis, conducted on the probabilistic scale by duly considering the randomness in excitations and uncertainties in parameters of structure, forms the backbone of the performance-based seismic design (PBSD) of structures.

One example of such an approach used in conducting seismic fragility analysis with uncertainty quantification is shown in Fig. 1.25. In the proposed framework, dynamic analyses of SDOF or MDOF system are conducted for either a unidirectional earthquake excitation or multi-directional earthquake excitations. In this probabilistic approach, in addition to the input structural parameters being uncertain, the input excitations are also probabilistic. Several random and probable samples of ground motion are generated to conduct the dynamic analysis of the SDOF or MDOF systems to determine their probability of failure in different modes, and the fragility curves are plotted by taking the uncertainties into consideration. The analysis steps shown in Fig. 1.25 in the form of a flowchart help in conducting this kind of seismic fragility analysis with uncertainty quantification. Upon calculating the capacity of the structure, the demand from the earthquake excitation is determined, and whether the structure has experienced failure in any mode is verified. If some failure is experienced, under the action of a particular intensity of earthquake, a point on the fragility curve is obtained corresponding to that IM. If no failure is

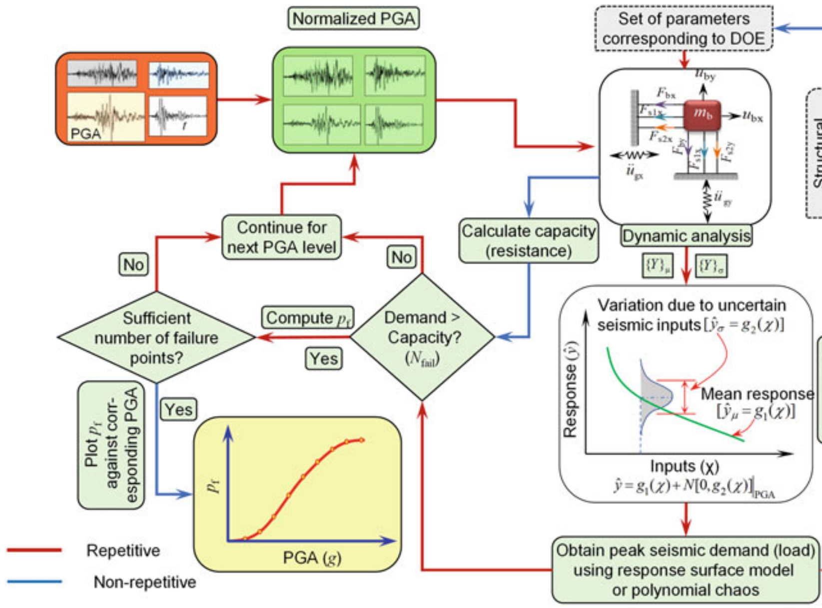


Fig. 1.25 Seismic fragility analysis with uncertainty quantification [21]

observed, the next increased intensity level of the earthquake is considered in the analysis, and this procedure is repeated. Subsequently, the probability of failure curve is plotted by joining the failure points and later vulnerability analysis can be carried out. To quantify the uncertainty, the dynamic parameters of the structure, dependant on material and geometric properties observed in the field, are varied. For example, possible variation (probability distribution) in the grade of concrete and that of the steel reinforcement grade is considered for calculating the seismic response of the structure.

1.8 Advanced Dynamic Response Modification Devices

There are other earthquake-resistant design approaches where the dynamic response of the structure is modified or controlled by installing mechanical devices or dampers. Such relatively sophisticated devices are primarily designed to modify the dynamic response of the structures, i.e., to reduce the amplitude of the vibration response which may be in terms of displacement or acceleration of the structure. A limited number of such devices have been employed in India; however in the rest of the world, especially in seismically developed countries, the use of dampers is quite prevalent. Japan being a seismic-prone country, such vibration response control devices are widely used in built infrastructure in Japan. Approximately, there are more than 2000 base-isolated buildings, more than 300 structures equipped with passive response dampers, and more than 40 active-controlled buildings in Japan. Comparatively, in India, there are about 10 base-isolated buildings, about 5 buildings installed with passive response control dampers, and the number of active control devices installed in structures in India is unknown.

The structural response control devices are typically categorized as (a) passive control devices, (b) active control devices, (c) semi-active control devices, and (d) hybrid control devices. Some of these devices are listed below; although it is not an exhaustive list, it is an indicative or representative list.

i. Tuned mass dampers (TMDs) and active tuned mass dampers (AMDs)
ii. Viscous, visco-elastic, and friction dampers
iii. Buckling-restrained bracings
iv. Base-isolation devices
v. Smart structures or systems or devices or dampers
vi. Semi-active and controllable dampers

Figure 1.26 depicts one simple example of a classical damper, known as the tuned mass damper (TMD). The dynamic properties of the main structure in Fig. 1.26a are governed by its mass (m_s), stiffness (k_s), and damping (c_s). If the main mass is dynamically excited, it will experience some vibration. The amplitude of this vibration will increase if the frequency of the excitation is near its natural frequency, i.e., in the resonance condition. Now, let's consider a small auxiliary mass (m_d) is installed on or attached to the main mass via a spring with stiffness, k_d, and a damper with a damping coefficient, c_d (Fig. 1.26b). Also, the stiffness of the auxiliary mass is adjusted in such a manner that the frequencies of the two masses match with each other, and then it is observed that the auxiliary mass vibrates quite vigorously under the same dynamic excitation, thereby actually reducing the dynamic response of the primary structure. Therefore, the displacement response of the main structure is controlled. This auxiliary mass is known as the tuned mass damper (TMD). The

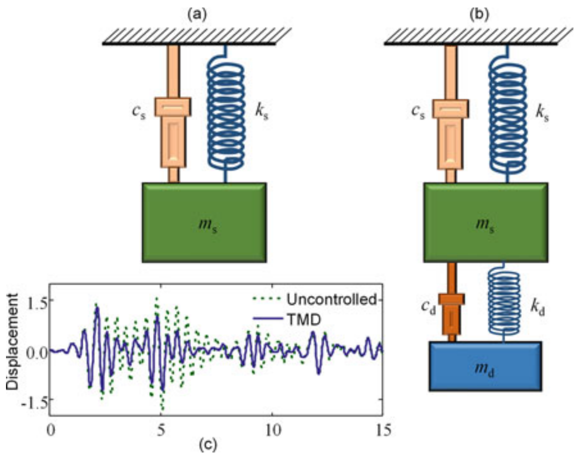


Fig. 1.26 **a** Uncontrolled structure, **b** tuned mass damper-controlled structure, **c** performance of the TMD

reduction in the dynamic response of the primary structure is achieved due to the tuning of the natural frequencies of the two connected masses, which forced them to oscillate in opposite phases to each other. The efficacy of a TMD in reducing the dynamic displacement response of the main structure is shown in (Fig. 1.26c) on time scale.

Essentially, these control devices provide a means to dissipate or absorb the energy imparted on the structure during an earthquake ground motion. When such energy management devices are not provided in the structure, the structural members like the lateral load resisting elements are required to dissipate or absorb this input energy via vibration of the members or via damage (formation of plastic hinges) in the members. In the conventional approach, the lateral load resisting members are designed so as to dissipate this input energy to the structures to make them earthquake-resistant. The nonlinearity (ductility) introduced in the structures improves its seismic performance considerably. Further, in capacity-based design, some structural members are identified to be designed with a higher level of nonlinearities, facilitating the formation of plastic hinges at specific predetermined locations. Thereby, the input energy is primarily absorbed by those members through large damages, thus protecting the other members from being damaged. Thus, in these tactfully engineered structures, the seismic performance of the structure is dictated by the designer such that all these nonlinearities are concentrated at certain locations or predefined members, whereas the rest of the structural members remain linear. Typically, the force–displacement (H-d) behavior of the ductile and non-ductile members is seen to be similar to what is shown in Fig. 1.27. However, if the entire input energy is required to be dissipated or absorbed by the structural members, such as columns and shear walls, they are liable to get damaged and they may even fail. Therefore, the mechanical devices become a viable alternative to either dissipate or absorb such input energy. In that case, the main structural members are not mandatorily required to be designed with such high nonlinearities (ductility) from an earthquake-resistant design viewpoint. Such design approach potentially avoids damage induced in the structural members, while dissipating or absorbing a significant amount of energy in the devices designed

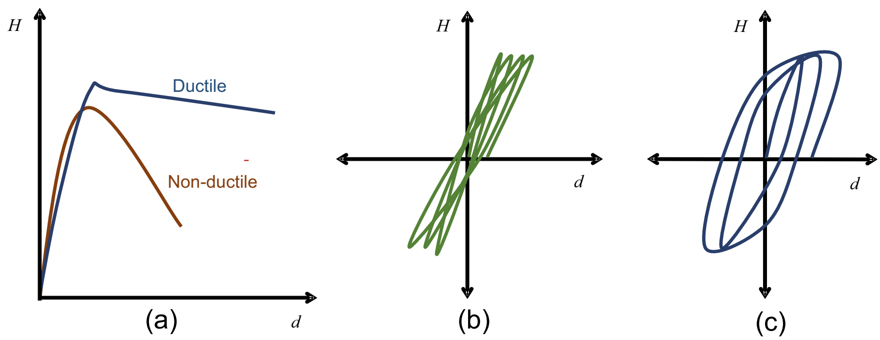


Fig. 1.27 Earthquake-resistant properties: **a** Ductile and non-ductile behaviors, **b** poor energy dissipation capacity, **c** good energy dissipation capacity

for the purpose. Thus, some specific devices are installed in the structures to cater to the seismic input energy. Also, after the end of an earthquake event, those external devices which get damaged due to the earthquake could conveniently be replaced with new ones. Repair or replacement of these control devices is much easier than repairing the structural members which have experienced seismic damage. These control devices could be replaced without even affecting the functionality of the structure.

In Fig. 1.27a, the area under the horizontal force (H) to displacement (d) plot in a non-ductile member is lesser than that in the ductile members, which indicates poor energy absorption. In an earthquake-resistant design structure, it is always better to achieve ductile behavior in the members so that adequate warning is received before the complete failure of the member. In the capacity design method, some of the earmarked members are designed such that their seismic capacity curve exhibits ductile behavior, while other structural members remain in their linear elastic zone, which automatically makes the area under the force–displacement curve larger, i.e., showing higher energy dissipation, as shown in Fig. 1.27c. Similarly, for the response control devices, a good energy dissipation or absorption capacity is intended as compared to the conventional materials and structures. A stable force–displacement curve having a larger area enclosed is an indication of higher damping, and is thus preferred in earthquake engineering. The supplemental damping devices which are used in seismic response control of structures are typically designed to exhibit bulkier hysteretic behavior when subjected to cyclic loading (refer to Fig. 1.28). Among several possible types of hysteresis loops, Fig. 1.28 depicts the elastic perfectly plastic (EPP) model which is seen in some types of response control devices, such as a friction pendulum system. Some other devices are characterized by elasto-plastic hardening (EPH) type of models, whereas some other devices are based on friction models. These are only a few of the different models of energy dissipation exhibited by the damping devices which are commonly used in practice.

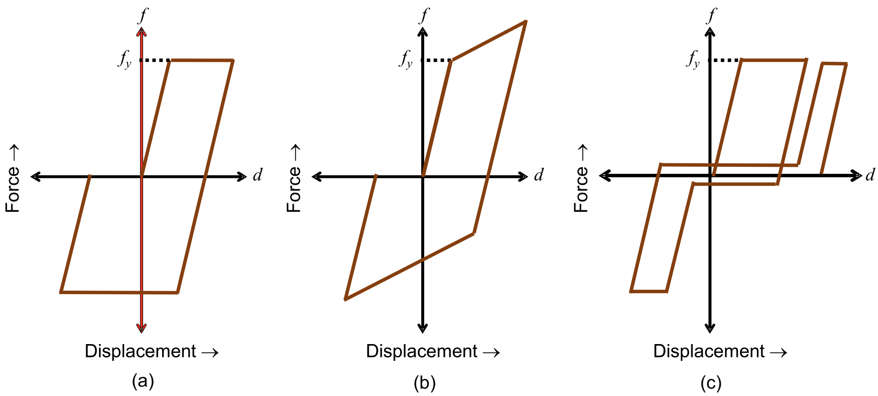


Fig. 1.28 Hysteresis models: **a** Elastic perfectly plastic (EPP) model, **b** elasto-plastic hardening (EPH) model, and **c** slip (Coulomb dry friction) model

Some of the most popular structural response control devices are shown in Fig. 1.29. Figure 1.29a,b depicts a fluid viscous damper (FVD) in which viscous fluid is contained in the cylinder and the piston moves when there is a relative displacement between the two ends of this damper. This can be compared to the shock absorbers used in automotive vehicles! In India, FVDs have been installed in Apollo Hospital Building in Delhi. Also, Airport Authority Building at Indira Gandhi International (IGI) Airport is equipped with the FVDs. However, the use of the FVDs is still relatively less popular in India as compared to the other seismically developed countries. In the visco-elastic damper, in addition to the damping through viscosity, an elastic force is also present in the restoring force behavior of the damper (Fig. 1.29c). The addition of damper(s) in a structure directly influences its dynamic properties such as damping and/or stiffness, i.e., the damped natural frequencies (also, time periods) would be different from the original undamped natural frequencies of the structure.

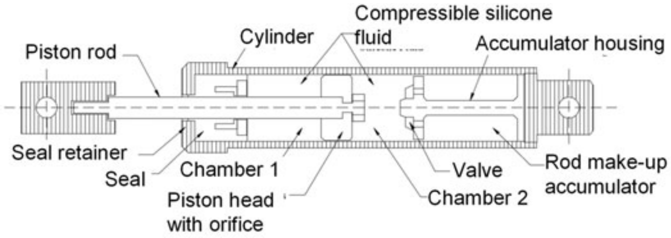
Some of the friction-type dampers (Fig. 1.29d, e) are also quite popular in the seismically advanced countries such as Canada and USA. Pall friction dampers are very commonly used in North America in the form of cross-bracings. In India, there are a few structures where some of these friction-type damping devices have been used; however, the number of such structure is very small. Sumitomo damper is another commonly adopted friction damper, which is similar to the fluid viscous damper in geometry though working on the principle of friction. Apart from that, at the Indian Institute of Technology (IIT) Delhi a few innovative types of dampers are developed at a laboratory scale. One such novel friction-type damping device is shown in Fig. 1.30. This friction-based damping device has an additional spring whose stiffness is designed to be nonlinear in nature. These dampers are either provided in the structural frame of the building or may even be used beneath the base of the building to reduce the large displacements in the base floor of base-isolated structures.

1.9 Seismic Base Isolation

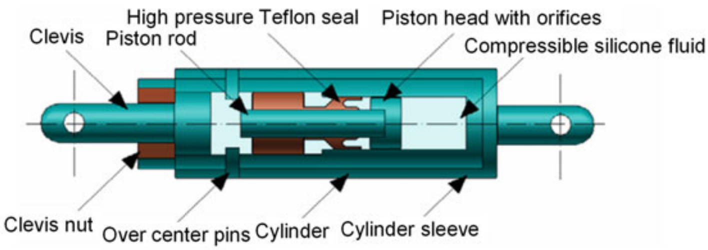
At the Indian Institute of Technology (IIT) Delhi, a number of novel seismic base-isolation devices have been studied and innovative/novel base-isolation systems are also developed. For example, a new base-isolation system is constructed with the help of bidirectional elliptical rolling rods (ERRs), as shown in Fig. 1.31. This device has been introduced to overcome some of the shortcomings of the existing base-isolation systems. Figure 1.32 shows the design and arrangement of the ERRs and the developed mathematical model thereof (Fig. 1.33). Essentially, these rods detach the building structure from the vibrating ground and thereby protect it from the fury of earthquakes. Thus, the ERRs actually isolate the structure from the ground (Figs. 1.31 through 1.33).

In the seismic base isolation of structures (Fig. 1.34), the control or response modification is achieved by providing flexibility and damping to the structure. It is relatively more popular than the use of dampers in improving the seismic performance of

(a)



(b)



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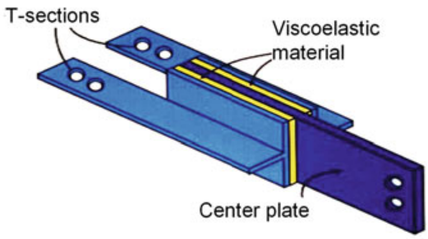

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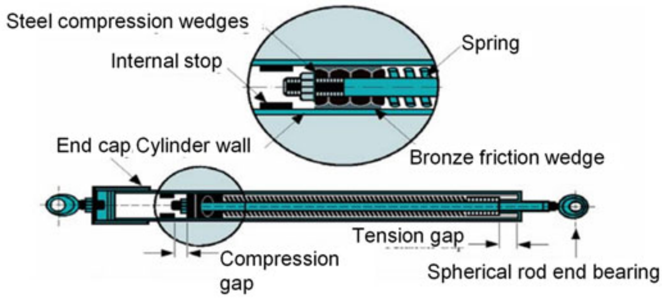


(e)

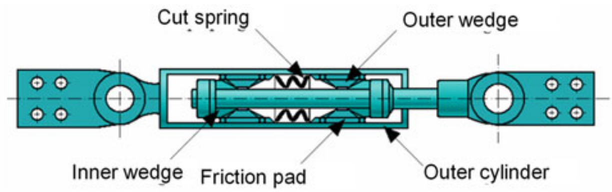


Fig. 1.29 Structural response control devices: **a** Fluid viscous damper [10], **b** pressurized fluid damper manufactured in Taylor Devices, Inc. [13], **c** visco-elastic damper [13], **d** energy-dissipating restraint manufactured in Fluor Daniel, Inc. [13], and **e** uniaxial friction dampers manufactured in Sumitomo Metal Industries, Ltd [13]

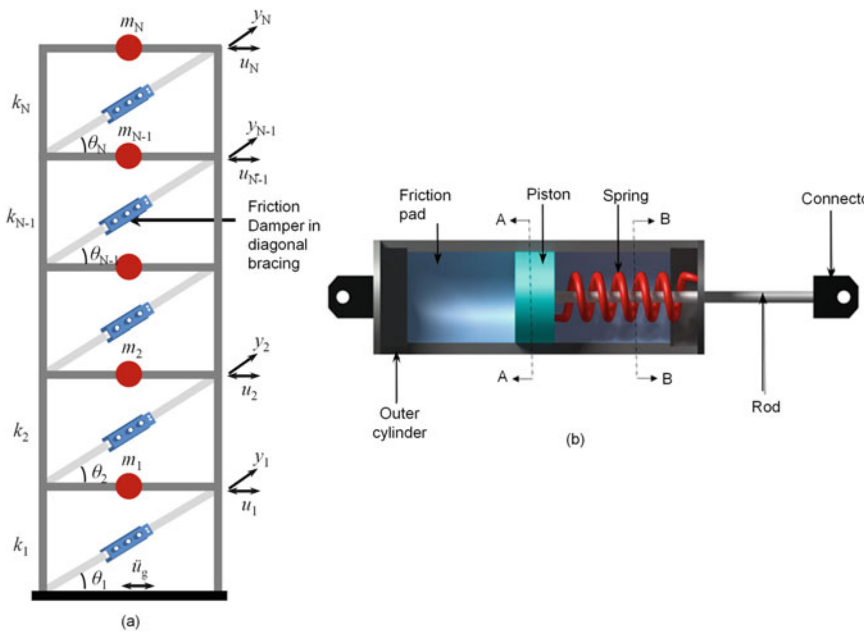
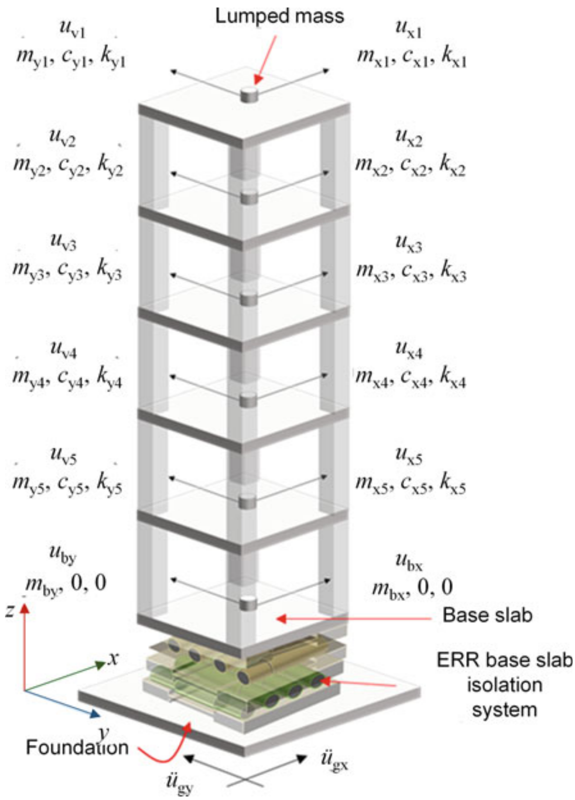


Fig. 1.30 **a** Mathematical models of *N*-story friction damper frame (FDF), and **b** interior view of the new friction-based damping system

a structure. Base isolation works on the principle of elongating (lengthening) the time period of the structure, i.e., reducing the structure's natural frequency. It is comparable with trees, in terms of dynamic property, which typically do not fall or fail during earthquakes because they are flexible, and sway largely during the earthquakes. In seismic base isolation, structures are similarly safeguarded by making them flexible, by lengthening their time periods so that they sway considerably during earthquakes but do not fail. The desirable flexibility is introduced through the use of various types of bearing or isolation systems available, some of which are quite widely used, such as laminated rubber bearings (LRBs) or lead core rubber bearings (also called New Zealand, NZ systems). The effectiveness of the base-isolation systems is due to changing the fundamental cantilever mode of vibration of the structure to the isolation mode, where the deformation is concentrated at the isolation level and the rest of the structure remains almost rigid. The factory-made isolation systems under the building experience most of the strain and therefore they take the wrath of the earthquakes, protecting the rest of the structure. Some of these bearings shown in Figs. 1.35, 1.36, 1.37, 1.38, 1.39 and 1.40 show various types of base-isolation systems which facilitate isolating the superstructure from the shaking ground and dissipating the seismic energy at the isolator level. These base-isolation systems help in shifting the period of the structures toward a more flexible side, thereby the spectral acceleration ordinates are reduced (refer to Fig. 1.34). During the earthquake event, the isolators may experience some damage; however, it is very easy to replace

Fig. 1.31 Mathematical model of flexible shear-type multistory building mounted on elliptical rolling rods (ERRs) [20]



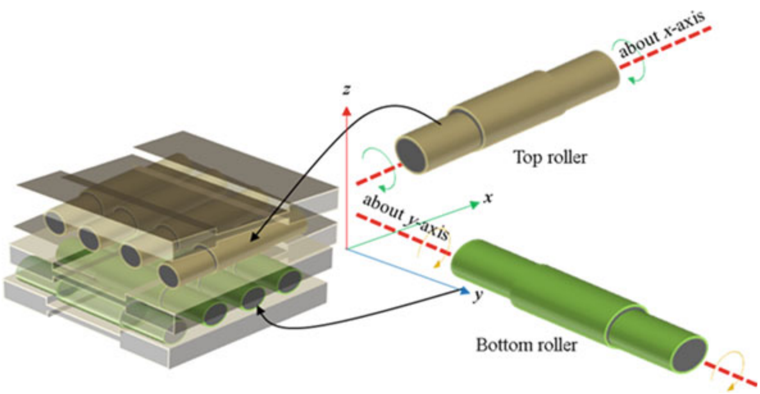


Fig. 1.32 Conceptual 3D views of the ERR with grooves [20]

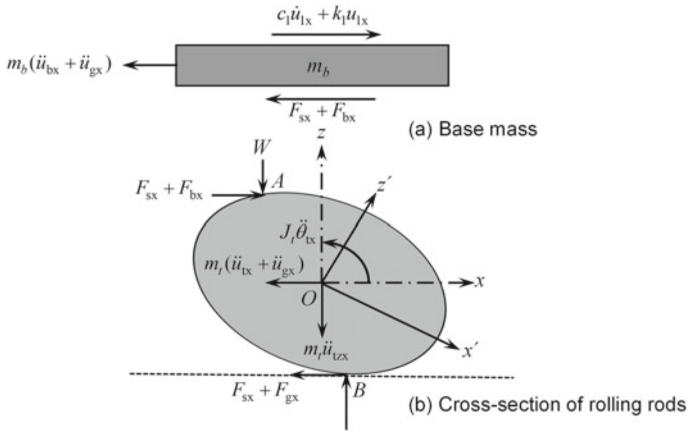


Fig. 1.33 Relationship between displacements of the base mass and ERR in x–z plane

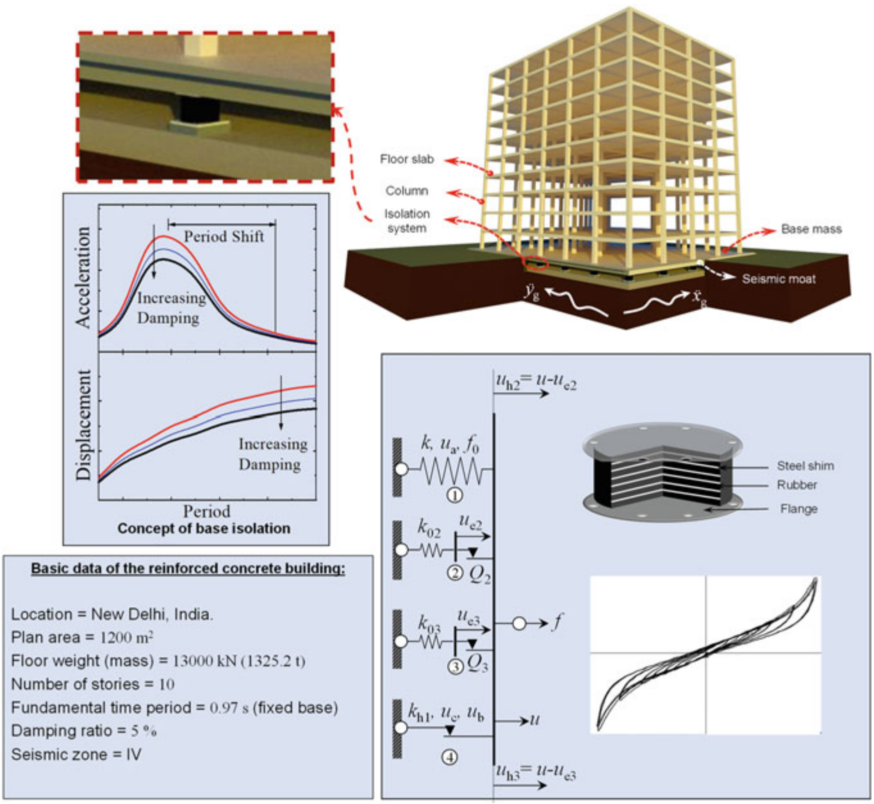


Fig. 1.34 Typical seismic base-isolated structure [28]

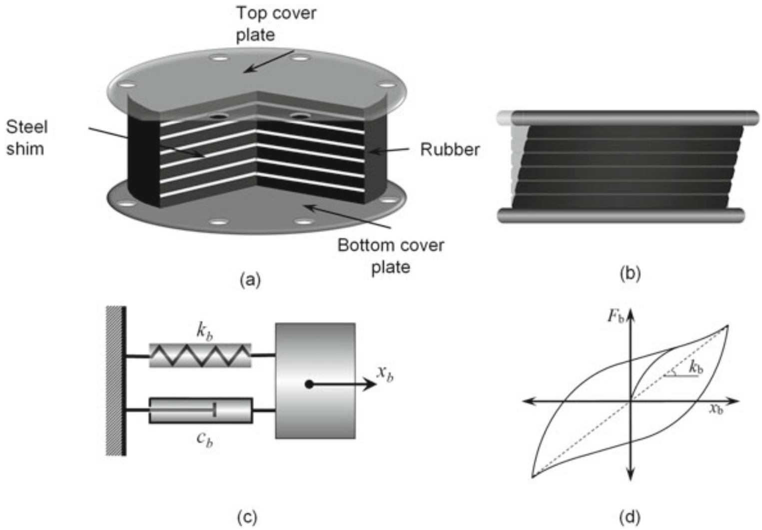


Fig. 1.35 **a** Laminated rubber bearing (LRB), **b** front view of LRB, **c** schematic diagram of LRB, **d** force–deformation behavior of LRB

them once the earthquake is over. Also, inspection and periodic maintenance of the isolation systems are convenient. Up to a certain level of earthquake excitation, for which the isolation system is designed, the superstructure is maintained to have a linear elastic behavior. It implies that no damage is experienced in the superstructure; and therefore the response reduction factor (R) could be kept inapplicable (i.e., equal to 1) to avoid any inelasticity and maintain continuous operational (O) level seismic performance. Thus, by employing the advanced base-isolation technology, the operational (O) level or immediate occupancy (IO) level of seismic performance is conveniently achievable, which will not cause any disruption in the functioning of the structure even after a major earthquake. Furthermore, seismic retrofitting by using the base-isolation technology is possible, which has a number of advantages, such as the superstructure intervention is almost nil, hence the original aesthetics of the structure is maintained intact; and also, the retrofitting operation can be carried out without affecting the functioning of the building [15].

A large number of base-isolation systems have been devised and implemented in real-life constructions. These isolation systems are broadly categorized as (a) elastomeric bearings, (b) sliding systems, and (c) rolling systems. Laminated rubber bearings (LRB), as shown in Fig. 1.35, are the most popular among all and are now widely available in India. Their mathematical modeling is carried out by using a simple spring and dashpot model and they provide a stable and bulky force–deformation hysteresis loop. A large amount of energy is dissipated by the laminated rubber bearing, which is quantified through the area enclosed within its force–deformation loop. Another popular bearing used across the world, including India, is a rubber bearing with a lead core inside, known as the lead core rubber bearing (New

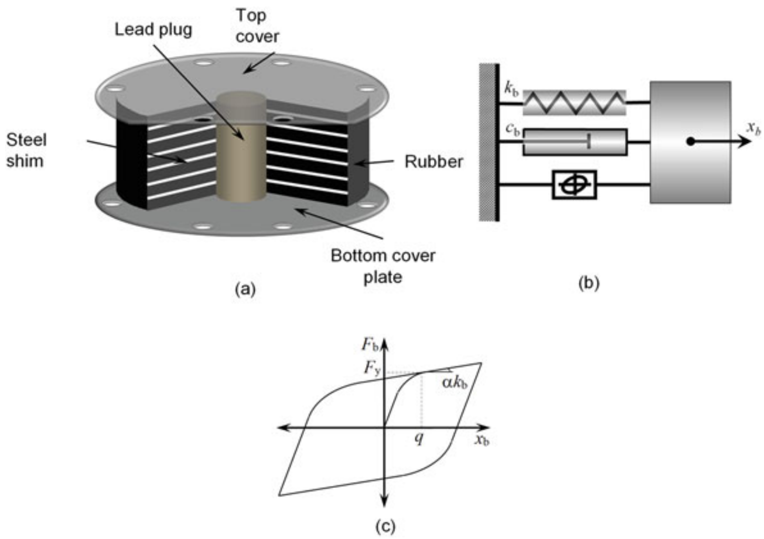


Fig. 1.36 **a** Lead core rubber bearing (New-Zealand, NZ system), **b** schematic diagram of NZ system, **c** force–deformation behavior of NZ system

Zealand, NZ system), as shown in Fig. 1.36. The central lead core provides the designed initial stiffness to the isolator so that it does not oscillate much during the ambient winds or small earthquake tremors. Additionally, the lead core dissipates energy, and even after rupture due to an earthquake event it recrystallizes afterward so that it is again available for sustaining the subsequent earthquakes, which means, the NZ system is not needed to be replaced after every earthquake.

The other isolation systems based on the sliding friction phenomenon are categorized as sliding isolation systems. Figure 1.37 depicts a pure-friction (PF) system, which is typically used in parallel or in addition to the other two isolation systems shown earlier (laminated rubber bearing and lead core rubber bearing) to reduce the cost of isolating the structure. Usually, the LRBs are costlier than pure-friction systems. The PF system is also called sliders, providing mostly velocity-independent low friction force, with no restoring capability. In bridges, such sliding isolation systems (without or with guides) are quite popular. Another very popular device used for structural response control of bridges, especially in the United States, is the friction pendulum system (FPS) as shown in Fig. 1.38. This ingenious sliding friction-based isolation system works on the principle of the swinging pendulum, where the structure mounted on top of the FPS oscillates with a designed time period with a tendency always to return to its original equilibrium position. During the oscillation, energy is dissipated in the sliding friction phenomenon. The design of the FPS includes curvature, which governs the time period of the isolator (more the curvature less is the time period and vice-versa), and the coefficient of friction at the articulated surface. Some other isolation systems used in practice are the resilient friction base isolator (RFBI) and the electricite-de-France system (EDF), shown in

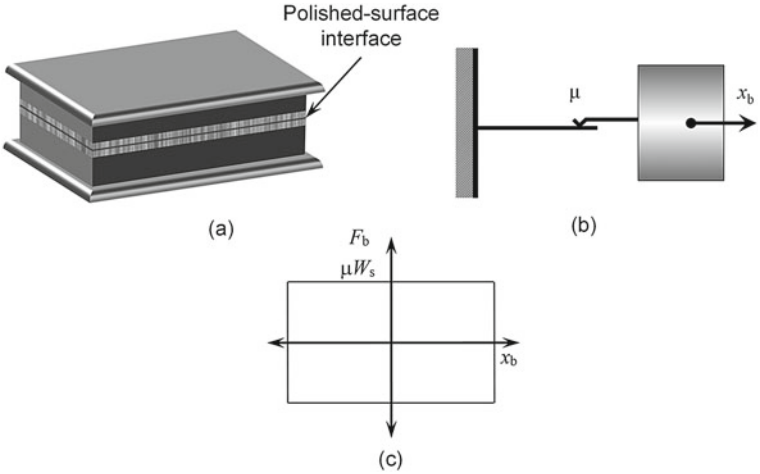


Fig. 1.37 **a** Pure friction isolation system (PF), **b** schematic diagram of PF, **c** force–deformation behavior of PF

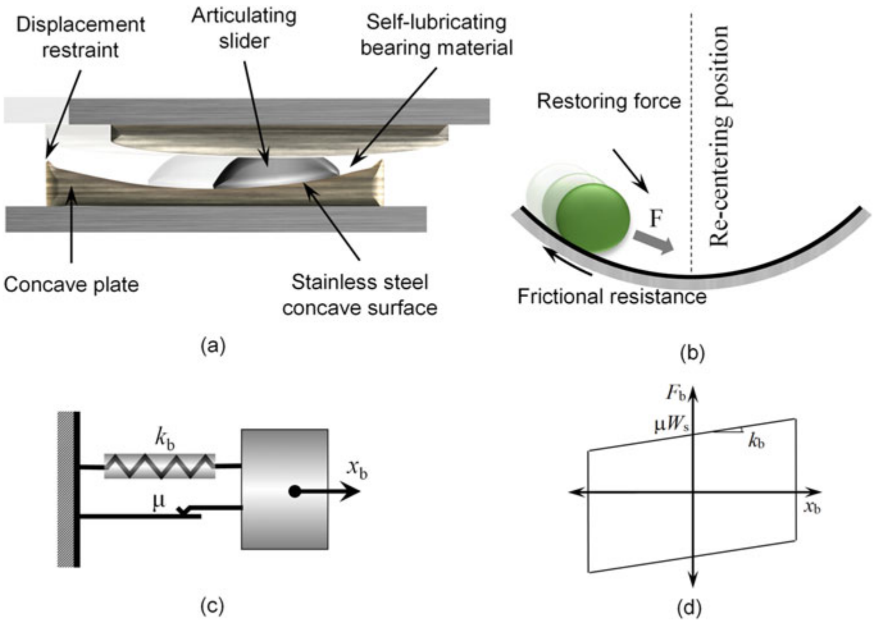


Fig. 1.38 **a** Friction pendulum system (FPS), **b** pendulum action, **c** schematic diagram of FPS, **d** force–deformation behavior of FPS

Figs. 1.39 and 1.40, respectively. The RFBI provides advantages of elastomeric and sliding isolation systems in one unit. The EDF is quite popular in France for seismic isolation of nuclear-containment structures.

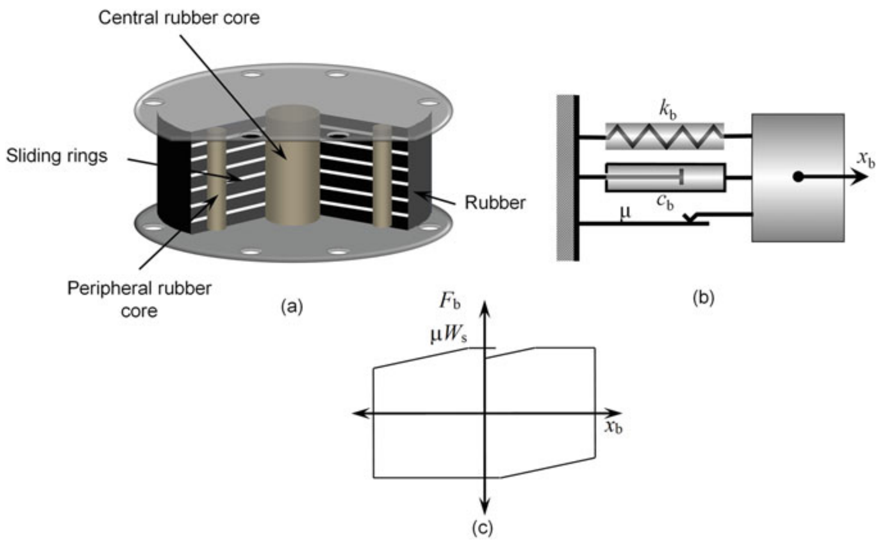


Fig. 1.39 **a** Resilient friction base-isolation system (RFBI), **b** schematic diagram of RFBI, **c** force–deformation behavior of RFBI

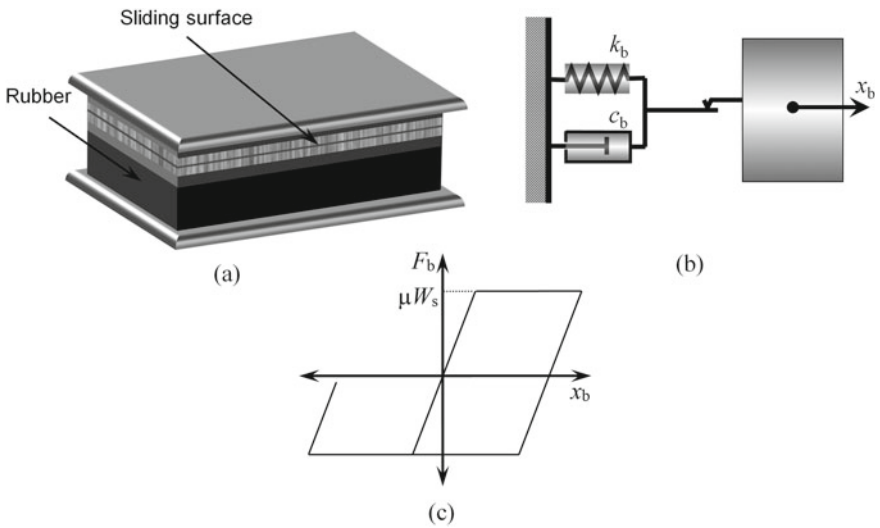
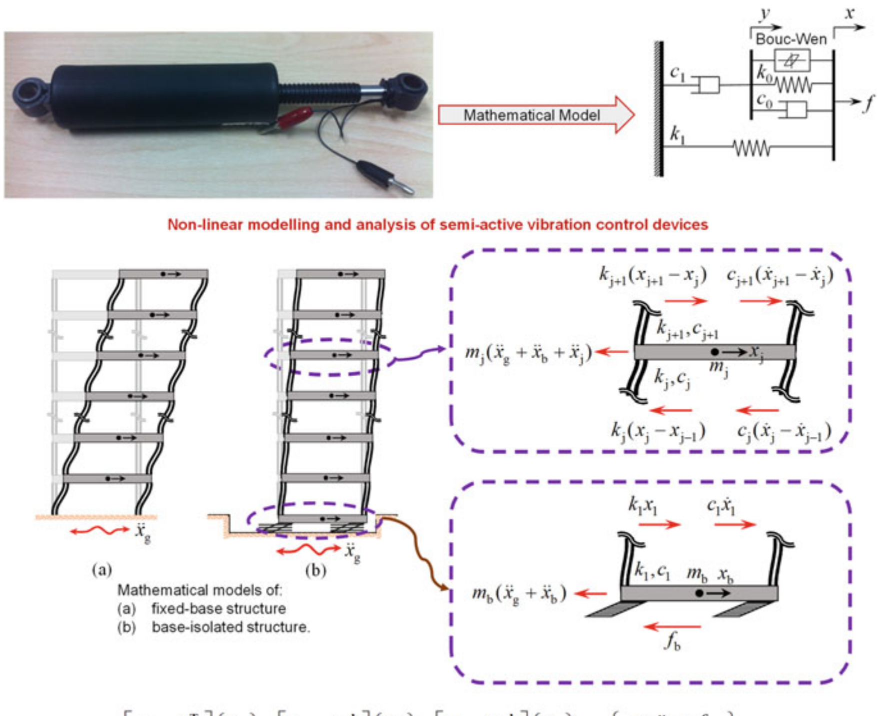


Fig. 1.40 **a** Electricite-de-France system (EDF), **b** schematic diagram of EDF, **c** force–deformation behavior of EDF

1.9.1 New Seismic Protection Devices—Isolation Systems

The response control devices discussed in the previous section were passive control devices, i.e., they function by virtue of their inherent dynamic properties which remain unaltered during the shaking. However, some seismic protection devices are designed to change their dynamic properties suitably during the shaking, which are called active or semi-active control devices. Especially, the semi-active type of control devices like magnetorheological (MR) dampers are employed to modify the structural response, as shown in Fig. 1.41. Here, a semi-active vibration control device in the form of an MR damper is tested at the Multi-Hazard Protective Structures (MHPS) Laboratory, Indian Institute of Technology (IIT) Delhi. The MR damper is mathematically modeled typically using the Bouc-Wen hysteresis model, for the purpose of simulating the dynamic response of the structures equipped with the MR dampers (Fig. 1.41). Experimentally obtained responses are used for validating the numerical simulation results so that the numerical models can be further used for modeling real-life buildings provided with the MR dampers and the seismic performance of the controlled structure could be estimated.



$$\begin{bmatrix} m_b & \mathbf{0}^T \\ \mathbf{0} & \mathbf{M}_s \end{bmatrix}\begin{Bmatrix} \ddot{x}_b \\ \ddot{\mathbf{X}}_s \end{Bmatrix}+\begin{bmatrix} 0 & -c_1\mathbf{r}^b \\ \mathbf{0} & \mathbf{C}_s \end{bmatrix}\begin{Bmatrix} \dot{x}_b \\ \dot{\mathbf{X}}_s \end{Bmatrix}+\begin{bmatrix} 0 & -k_1\mathbf{r}^b \\ \mathbf{0} & \mathbf{K}_s \end{bmatrix}\begin{Bmatrix} x_b \\ \mathbf{X}_s \end{Bmatrix}=-\begin{Bmatrix} m_b\ddot{x}_g+f_b \\ \mathbf{M}_s\mathbf{r}(\ddot{x}_g+\ddot{x}_b) \end{Bmatrix}$$

Fig. 1.41 Nonlinear modeling and analysis of semi-active vibration control devices

Fig. 1.42 Base-isolated showcase building (a collaborative project between Resistoflex and IIT Delhi)

It is crucial to take the investigated seismic response control technologies to the field for implementation in real-life structures. In a number of construction projects in India, the designers have shown keen interest in using some of these innovative technologies for the protection of structures from earthquakes. In Fig. 1.42, a base-isolated building constructed in Delhi is seen, where double curvature friction pendulum (DCFP) systems are used for isolation purposes. Here, the DCFP systems are installed at the first floor level of the structure, i.e., the isolators are placed on the column pedestals. The structure is designed for the operational (O) level of seismic performance.

1.9.2 Base-Isolated Structures at IIT Guwahati

Health monitoring of structures is carried out to understand how these novel vibration control devices or systems are performing, especially during an earthquake activity. The building shown in Fig. 1.43 is a demo base-isolated building constructed at the Indian Institute of Technology (IIT) Guwahati with the funding received from the Bhabha Atomic Research Centre (BARC). A health monitoring system is installed in this building to study the difference in the seismic performance of the non-isolated (fixed-base) and the base-isolated buildings during earthquakes. Moreover, a scientific study has been undertaken to study how the contents within the base-isolated building are protected as compared to the contents in the fixed-base building. For this purpose, Fig. 1.44 shows an algorithm developed and implemented for real-time

Fig. 1.43 Fixed-base and base-isolated structures at IIT Guwahati

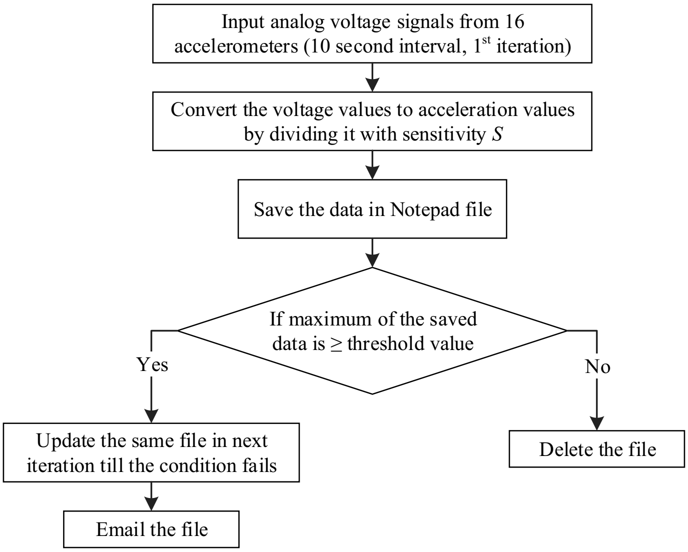


Fig. 1.44 Algorithm used for structural health monitoring

structural health monitoring of the two adjacent buildings (one base-isolated and another non-isolated) and secondary systems (SS) installed within them. The real-time data recorded in this process is also used for predicting the dynamic response of the structures during severe earthquakes. Further, data analytics tools are being used in this project to quantify the vulnerability of the base-isolated building under site-specific earthquake shaking of different intensities. Figure 1.45 shows the loca-

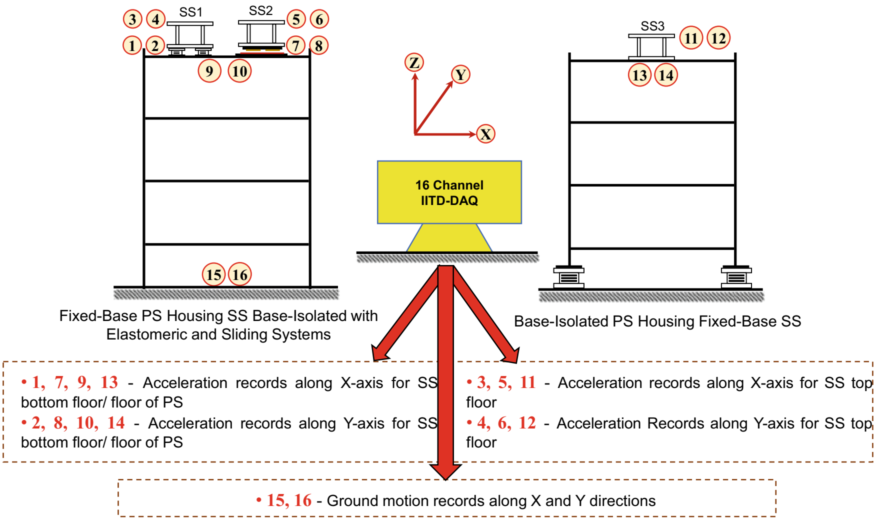


Fig. 1.45 Locations of sensor placement in the building

tions of the different sensors used for the real-time monitoring of the two buildings at the Indian Institute of Technology (IIT) Guwahati.

1.9.3 Tuned Mass Damper(s)

Apart from base isolation, another passive control device which is popularly used in real-life structures is known as the tuned mass damper (TMD), as introduced earlier. It is a classical dynamic response abatement device which uses an additional mass to dissipate the input seismic energy, thus reducing the response of the primary structure. Some recent modifications have been introduced to this device, by spatially distributing a number of TMD devices, which are called distributed multiple tuned mass dampers (d-MTMDs) as compared to the earlier single TMD (S-TMD) unit. Typically, such tuned mass damper(s) is/are used for controlling the vibration response in mid-rise to high-rise structures, as shown in Fig. 1.46. The TMDs are more popular for wind response control of structures, but they are quite beneficial for earthquake response control as well. The TMDs are used in bridges for both live load response control and earthquake response control (Fig. 1.46). In a classical problem of vehicle-bridge interaction (VBI), these devices are introduced to abate the dynamic response of the bridge under the vehicular loading. Moreover, the TMDs also find applications in tall structures such as reinforced concrete (RC) chimneys, provided in the form of annular rings in the cylindrical chimneys. Significant research contributions have been made in the application of the TMDs (passive, semi-active, and

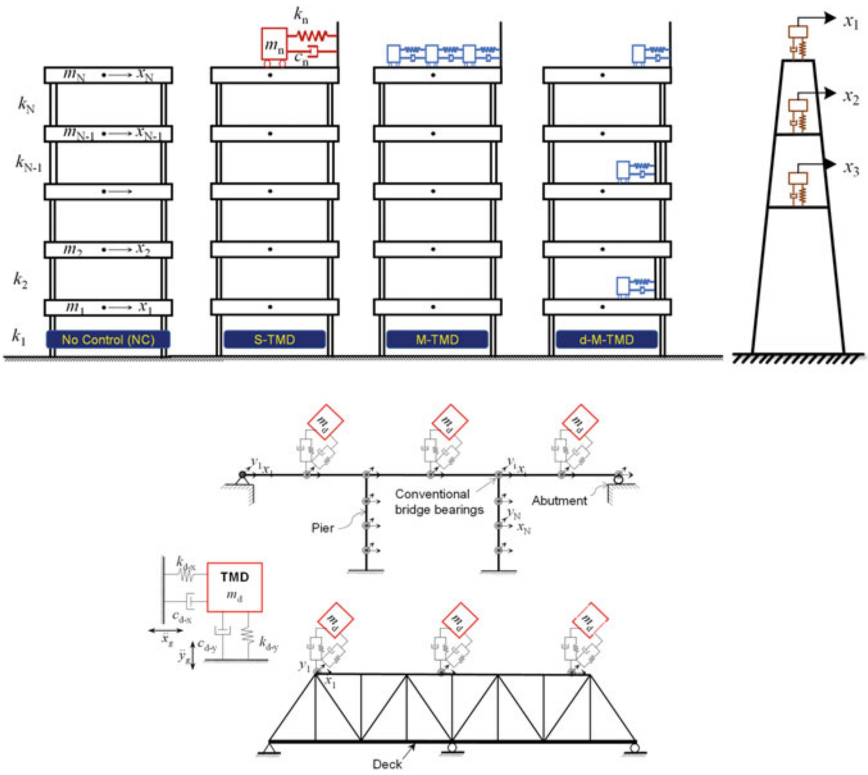


Fig. 1.46 Implementation of different types of tuned mass dampers in civil infrastructures [2, 14]

active) in dynamic response control of various structures under a variety of loading conditions, for which specialized literature can be referred.

1.9.4 Innovative Structural Control Algorithm

The latest technologies in the TMDs are meant to make them tuneable during real-time functioning, enhance semi-active response control by introducing some innovative control algorithms, and ensure their effectiveness under multi-hazard loading scenarios, by introducing artificial intelligence and machine learning (AI-ML) tools. Figure 1.47 shows the location of semi-active TMDs (SA-TMDs) provided in SDOF and MDOF structures, and an innovative response control algorithm for SA-TMD is shown in Fig. 1.48. In Figure 1.49, the acceleration and displacement response of the structure with the SA-TMD are found to be reduced significantly as compared to the uncontrolled structure and even to the structure with a single passive TMD. These are some of the ongoing investigations in dynamic response control devices.

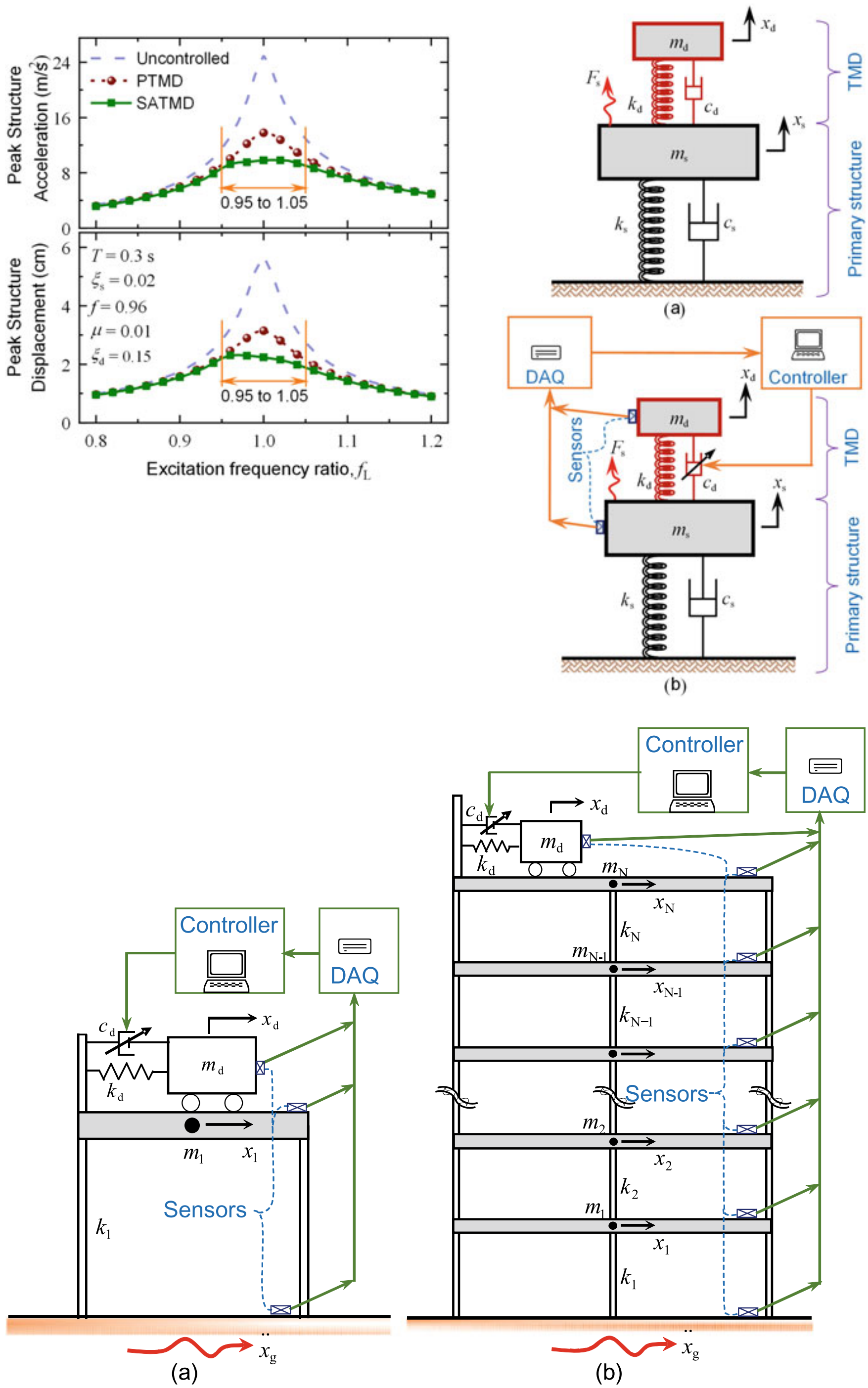


Fig. 1.47 Semi-active tuned mass damper (SA-TMD) installed in single and multistory buildings [26]

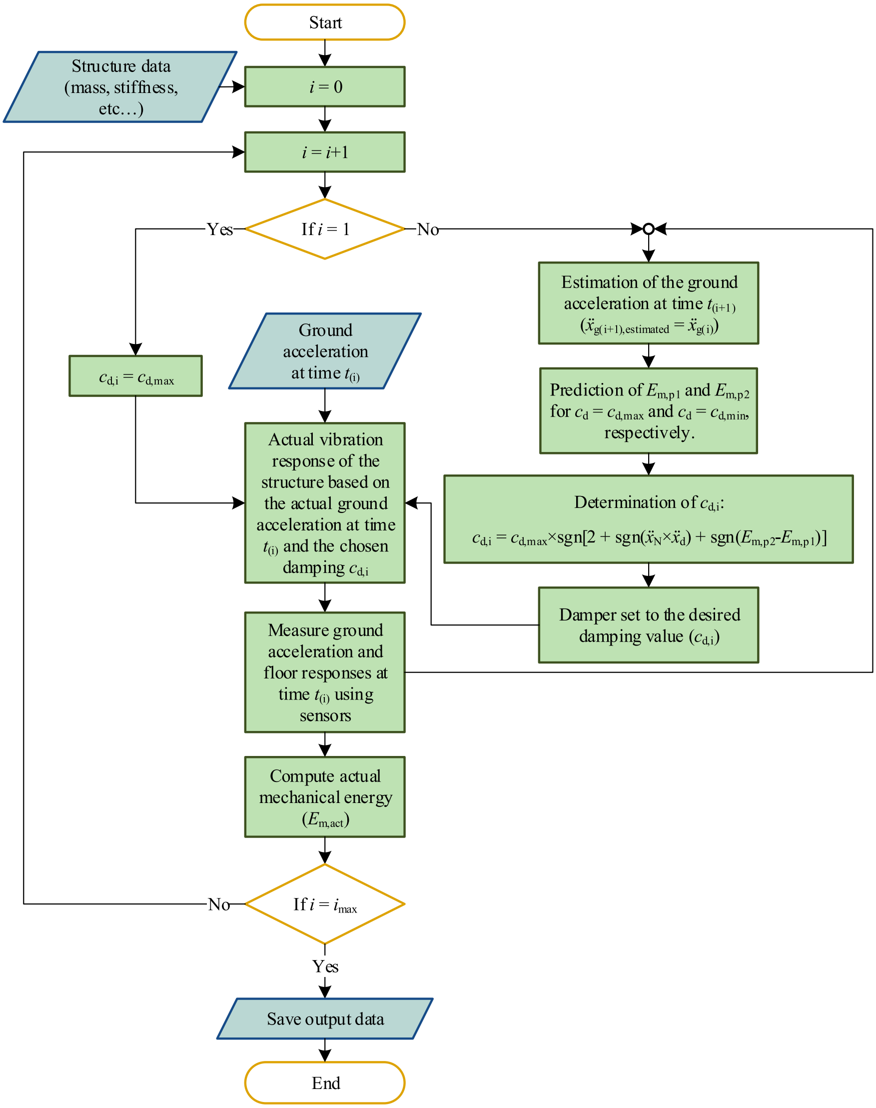


Fig. 1.48 A new energy-based predictive (EBP) control algorithm for the SA-TMD [26]

Subsequently, these devices will be further improved in terms of their mechanical performance by changing the way the energy is dissipating and the way the inertial forces are being added.

Upon conducting nonlinear time history analyses of bridges with distributed multiple tuned mass dampers (d-MTMDs), the response reduction (shown in Fig. 1.50) reveals the effectiveness of the passive d-MTMDs, as compared to the structure which is not controlled (NC) or structure controlled by a single TMD (STMD).

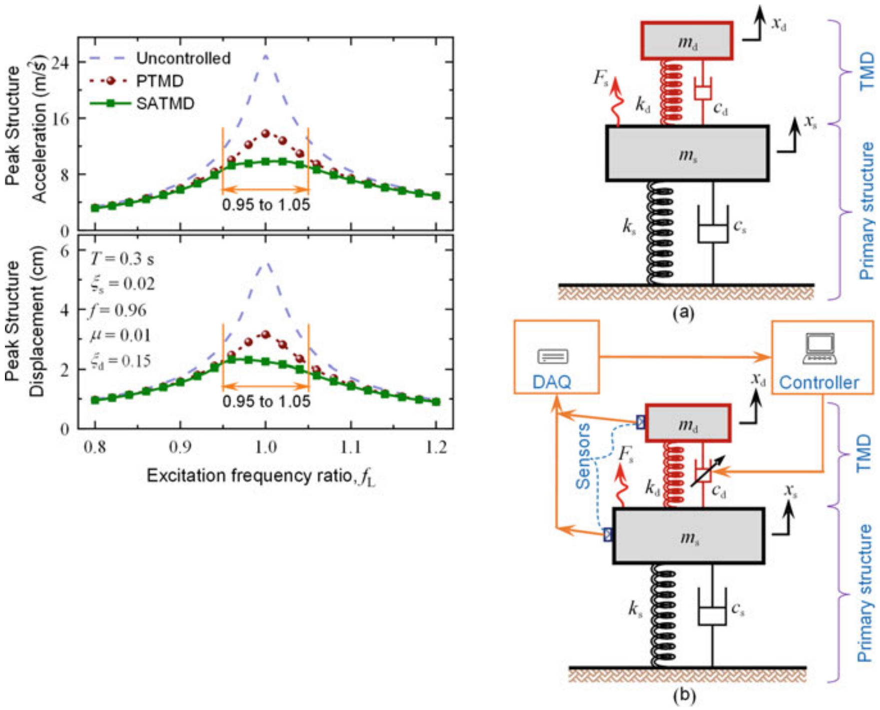


Fig. 1.49 Comparison of the effectiveness of the SA-TMD against the uncontrolled structure and the single passive TMD [27]

These have been some of the advanced technologies which are being used for controlling the seismic response of buildings and bridges. It gives merely a glimpse of the current state-of-the-art and the current practice, as well as opens new avenues for conducting research on emerging topics in earthquake engineering and technology.

1.9.5 Need for Future Research

In summary, various fundamental aspects of earthquake engineering and technology have been dealt with here at introductory level. The broad spectrum and various subdisciplines of earthquake engineering and technology have been presented systematically and succinctly. Upon putting the earthquake engineering and technology in a perspective, the future research needs are summarized below:

i. Improved strong ground motion prediction models are essential.
ii. Different materials are being innovated as compared to the conventional reinforced concrete (RC) or masonry structures. Such new materials are required to have high strength and still be lightweight as far as possible.

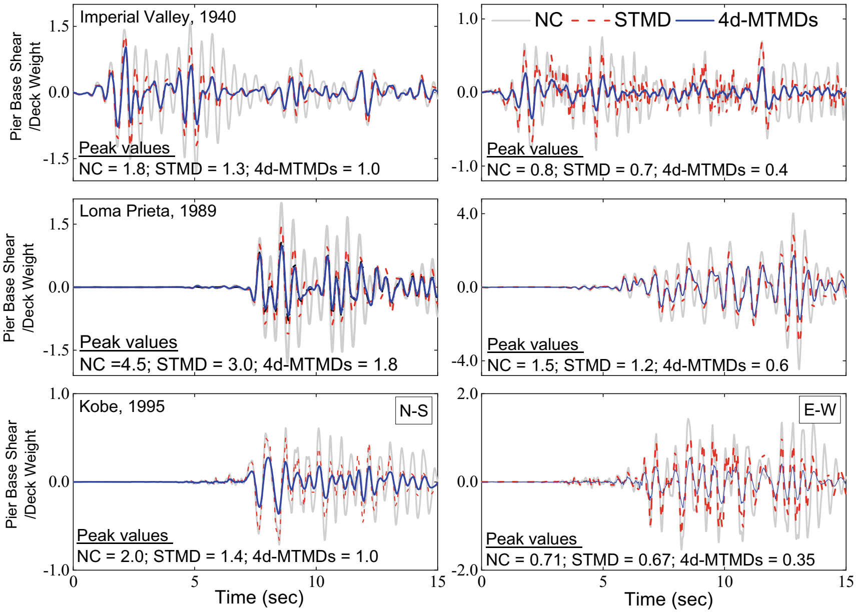


Fig. 1.50 Seismic response control of bridge with d-MTMDs [14]

iii. Nonlinear seismic analysis of structures should become a routine. In due course of time, it should be incorporated into the codal provisions as well.

iv. Computational structural dynamics will evolve much strongly with the advent of high-performance computing (HPC) facilities. Subsequently, not only for very important structures such as the nuclear power plant (NPPs) but also for other routinely designed important structures, conducting nonlinear time history analysis (NLTHA) should become a commonplace practice.

v. As compared to the conventional force-based seismic analysis, gradual transformation toward the displacement-based analysis is taking place, which is quite suitable for inelastic response evaluation as well as evaluating serviceability performance criteria.

vi. To quantify the seismic performance of structures even within the inelastic range, certain new approaches are being investigated as parts of development in performance-based earthquake engineering (PBEE). A need is felt to duly account for the probability of occurrence of earthquakes, randomness thereof, and uncertainties in the structural properties. Some of the current probabilistic approaches invented must be incorporated into the codes and standards in the future.

vii. Codes and standards which earlier were mainly following deterministic approaches should evolve to adopt more probabilistic approaches.

viii. Dynamic response control devices are being improved day by day. Drawing a parallel with the car industry, where years after years, the comfort level in riding the cars is improving with higher speeds and automation, there is a requirement to further improve the current structural response control devices to enhance their performance further.

ix. Structural health monitoring (SHM) will reveal how these new-generation engineered structures are performing during earthquakes as compared to the classical structures. The sensor-based data gathered from the continuous SHM should be used further to contribute to the development of new technologies in earthquake-resistant design structures with enhanced performance.

x. Some of the advanced techniques such as data analytics, machine learning, or deep learning should find their applications in structural analysis and response control.

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