Technical Considerations for Seismic Isolation of Nuclear Facility Structures

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Disclaimer

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Abstract

Seismic isolation reduces the response of a structure to horizontal ground motion through the installation of a horizontally flexible and vertically stiff layer of seismic isolation hardware between the superstructure and its substructure. The dynamics of the structure are thus changed such that the fundamental vibration period of the isolated structural system is significantly longer than that of the original, non-isolated, structure. This leads to significant reductions in the accelerations and forces transmitted to the isolated superstructure during an earthquake. At the same time, the seismic isolation devices undergo large horizontal deformations: they are engineered to safely sustain such motions and thus protect the isolated superstructure.

Modern seismic isolation devices and components, and engineering methodology for their use to modify the seismic response of structures have been developed in 1970's and 1980's. Since then, a significant number of conventional buildings, industrial structures and bridges have been seismically isolated in the US and abroad. Seismic isolation has been used to design and construct nuclear facility structures in France and South Africa in late 1970' and early 1980's. In the US, two seismically isolated reactor designs, PRISM and SAFR, were developed through DoE support and examined by NRC mid 1980's and early 1990's.

The renaissance of nuclear energy is the catalyst behind the surge in exploring new civil/structural technologies to increase the seismic safety and reduce the cost and time to build new nuclear power plants. In nuclear facility structures seismic isolation can be used to, with high confidence, limit the inertial forces and vibrations transmitted from the ground to the isolated nuclear facility superstructure for a wide variety of potential nuclear power plant sites. This enables standardized design, fleet licensing, and modular construction of nuclear power plants.

This report presents the technical considerations for analysis and design of nuclear power plants and safety-related nuclear facility structures using seismic isolation. It is intended to serve as a reference to those engineers planning to use seismic isolation in their designs, as well as regulators reviewing applications that employ seismic isolation hardware and systems. Seismic isolation is treated as a civil/structural sub-system of a nuclear power plant whose risk-informed design is governed by the performance objectives defined in ASCE 43-05 "Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities". Behavior, mechanical properties, modeling, structural response analysis, design, qualification and in-service monitoring issues for seismic isolation design using the seismic isolation device most commonly used in the U.S. are presented.

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1 Introduction

The renaissance of nuclear energy is the catalyst behind the surge in exploring novel civil/structural technologies to increase the seismic safety and reduce the cost and time to build new nuclear power plants. Seismic isolation is one such civil/structural technology: it enables engineered modification of the seismic response of a structure and its systems and components.

Seismic isolation is not a brand new technology: modern seismic isolation devices and isolation design methodology have been developed in 1970’s and 1980’s. Since then, a significant number of conventional buildings, industrial structures and bridges have been seismically isolated in the US and abroad. During the 1980’s seismic isolation has also been used to design and construct nuclear facility structures in France and South Africa, and considered for nuclear facilities in Japan and the US. However, the downturn in nuclear power plant construction after the Three Mile Island and the Chernobyl accidents put a stop on further development of seismic isolation for nuclear facility structures.

Seismic hazard remains of the three major contributors to the total risk of nuclear reactor core damage and large radiation dose release. Seismic isolation is a technology that can significantly reduce the contribution of seismic hazard to the overall risk exposure of nuclear power plant structures. It can be used to, with high confidence, limit the inertial forces and the vibrations transmitted from the ground to the isolated nuclear facility superstructure. Furthermore, seismic isolation can be engineered to control the seismic excitation of the isolated superstructure at essentially the same low level for a wide variety of potential nuclear power plant sites. This enables modular design, fleet rather than individual plant licensing, and modular construction of nuclear power plants, thus significantly improving their economy.

1.1 Objective

Reduction of seismic risk and reduction of cost and time to build are the two principal incentives for license applicants to seriously consider seismic isolation of new nuclear power plant structures. The objective of this report is to summarize the technical considerations that will serve as the basis for design regulation and regulatory review of seismically isolated nuclear power plant structures.

1.2 Scope

This report presents the technical consideration for analysis and design nuclear power plants and safety-related nuclear facility structures using seismic isolation. It is intended to serve as a reference to those engineers planning to use seismic isolation in their designs, as well as regulators reviewing applications that employ seismic isolation hardware and systems.

The technical considerations presented in this document apply to the seismic isolation of nuclear power plants and safety-related nuclear facility structures and not their subsystems or individual components (such as equipment). It is assumed that any hardware used in the seismic isolation system will be installed in a low- or no-radiation environment. Therefore, the effect of radiation on the response of seismic isolation hardware is not discussed in this report.

To the extent possible, the technical considerations provided herein will be compatible with other resource documents on the seismic isolation of infrastructure, such as ASCE 7 *Minimum design loads for buildings and other structures* (ASCE 2010a), ASCE 4 *Seismic analysis of safety-related nuclear structures* (ASCE 2010b), and the AASHTO *Guide Specification for seismic isolation design* (AASHTO 2010).

1.3 Fundamentals of Seismic Isolation

Figure 1.1 shows an elevation view of a typical isolated structure and identifies its main parts. The terminology in Figure 1.1 will be used consistently throughout this report. This terminology is explicitly defined in the Definitions section of this report.

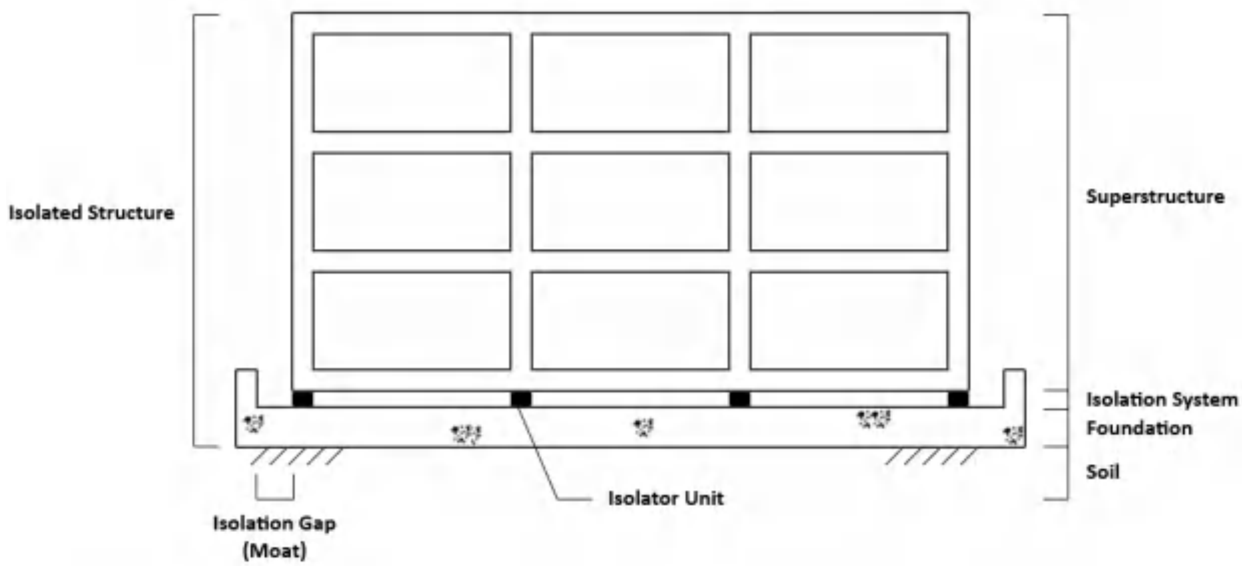


Figure 1.1 Elevation of a typical seismically isolated structure.

1.3.1 Traditional Approach to Seismic Design

Earthquake engineering design is intended to produce structures that perform in a certain manner when subjected to a given level of earthquake shaking. Single or multiple performance objectives can be targeted. For example, a structure might be designed for two performance objectives: immediate occupancy after a frequent earthquake and life safety after a rare earthquake. The design of seismically isolated structures is no different. Chapter 2 of this report discusses performance objectives forming a design basis, as well as deterministic and probabilistic acceptance criteria.

Before discussing the mechanics of seismic isolation, it is beneficial to explain the traditional approach to earthquake engineering design. Engineers typically employ one of two strategies to protect buildings supported on and tightly coupled to the ground from the damaging effects of earthquakes.

In the first strategy, engineers design the lateral force resisting system to be very stiff, producing a structure with a short natural period (or high natural frequency), small drifts and large accelerations in design earthquake shaking. Such framing systems will protect drift-sensitive components (e.g. partition walls in their plane) but could damage acceleration-sensitive components (including mechanical equipment, computers, and electrical systems). Historically, engineers of nuclear structures have used this design strategy to make sure that horizontal deformations in nuclear power plants are very small. Reinforced concrete nuclear power plant structures have been designed to remain essentially elastic in design earthquake shaking but detailed per ACI 349 (ACI 2006) to respond in a ductile manner in the event of shaking more intense than the design basis. This approach differs from the design of conventional structures wherein substantial damage is anticipated in design basis shaking because the structures are engineered to develop inelastic deformations in a controlled manner. Reinforced concrete shear walls and steel braced frames are typical stiff seismic framing systems.

In the second strategy, engineers design the lateral force resisting system to be flexible, producing a structure with a long natural period (or low natural frequency). As a result, earthquake-induced horizontal accelerations and forces are relatively small, but transient drifts are large and residual drifts might be significant. The design intent for such flexible systems is that deformations are distributed throughout the entire structure, rather than concentrated in one or just a few locations. Moment frames of structural steel and reinforced concrete are typical flexible seismic framing systems where stringent seismic detailing of the structural elements and connections is required to enable large inelastic deformations of the framing systems.

The design spectrum in Figure 1.2 illustrates the trade-off between horizontal displacements and horizontal accelerations of a structure subjected to typical earthquake excitation. A design spectrum plots spectral acceleration or spectral displacement versus natural period. Figure 1.2 clearly demonstrates that as the natural period of a structure increases (the structure becomes more flexible), horizontal accelerations and, therefore, inertial forces decrease while horizontal displacements increase. Conversely, as the natural period decreases (the structure becomes stiffer), horizontal displacements decrease while horizontal accelerations and inertial forces increase.

(a) inertial forces (b) displacements

Figure 1.2 Impact of period elongation on inertial forces and displacements in structures (courtesy of DIS Inc, reproduced with permission)

1.3.2 Basic Mechanics of Seismically Isolated Structures

Seismic isolation is a dynamic response modification technology accomplished through the installation of a horizontally flexible and vertically stiff layer of seismic isolation hardware between the superstructure and its substructure. Figure 1.1 shows a cross-section of a building equipped with seismic isolation hardware. Seismic isolation hardware typically comprises seismic isolation units, devices designed to enable horizontal motion while providing vertical support, and seismic isolation components, serving to connect the seismic isolation units to the substructure and the superstructure and to perform additional functions related to seismic isolation such as to provide additional damping. Because the seismic isolator units are usually (but not necessarily) installed at the base of a structure, seismic isolation is also referred to as base isolation.

Seismic isolation works by changing the dynamics of the structure such that the dynamic response of the isolated superstructures to seismic excitation is decoupled from that of its substructure. The decoupling is accomplished using one or a combination of the following two horizontal deformation mechanisms: shear deformation of a flexible device; and sliding on friction-controlled surfaces. The flexible device mechanism works by introducing an additional natural vibration mode shape of the isolated structural system: this mode shape is orthogonal to the vibration mode shapes of the superstructure and thus works as a filter that minimizes the vibration of the isolated superstructure. The sliding mechanism limits the force to the level of friction generated between the sliding surfaces: it allows the isolated structure to move when inertia of the isolated superstructure exceeds the friction. For a simple numerical model that demonstrates these concepts, please refer to Appendix A.

While the initiation of motion is different, both horizontal deformation mechanisms perform in a similar manner once the isolated superstructure is moving. They modify the dynamic response of the structure by making the fundamental horizontal-direction vibration period of the isolated structural system significantly longer than that of the original fixed-base (non-isolated) structure. This leads to significant reductions in the horizontal accelerations and inertial forces transmitted to the isolated superstructure during an earthquake compared to the traditional fixed-base structure. Making use of the design spectrum shown in Figure 1.2 and the fact that horizontal acceleration are inversely proportional to the fundamental vibration period of the structure in the range of interest, it is evident that elongation of the fundamental vibration period achieved by seismic isolation is a very effective method of reducing inertial forces experienced by the isolated superstructure. Reduction of seismic loads in the superstructure offers another benefit: the superstructure can be engineered to be sufficiently strong and stiff to control its deformation to acceptably small levels and thus protect its integrity and the integrity of the systems and components it contains. At the same time, the seismic isolation devices undergo large horizontal deformations: they are engineered to safely sustain such motions and thus protect the isolated superstructure.

1.3.3 Types of Seismic Isolation Units

Figure 1.3 shows an idealized horizontal force-displacement relationship for a typical seismic isolator unit. The isolator has high initial stiffness (K_u) to limit horizontal displacements induced by low-level lateral loads, such as wind loads. If the displacement exceeds the yield capacity of the isolator (u_y), its horizontal stiffness decreases substantially (K_d). This reduced stiffness provides the flexibility necessary to effect seismic isolation during an earthquake.

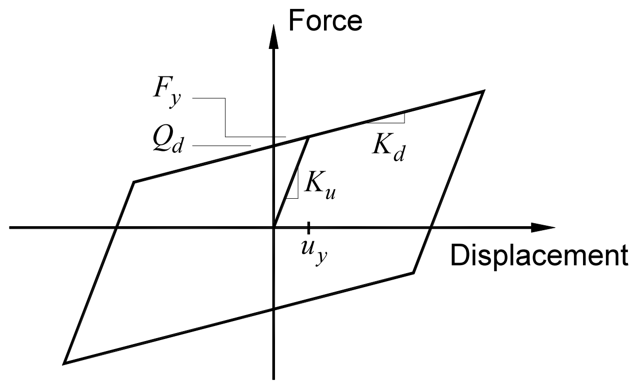


Figure 1.3 Force-displacement relationship for an isolator unit or system (Huang et al. 2009)

There are two main types of passive seismic isolation units. Each is briefly summarized in the paragraphs below. Chapter 3 of this report discusses each in more detail. Passive seismic isolation units do not have any internal sources of energy or controllers and control systems to modify their behavior using feedback control.

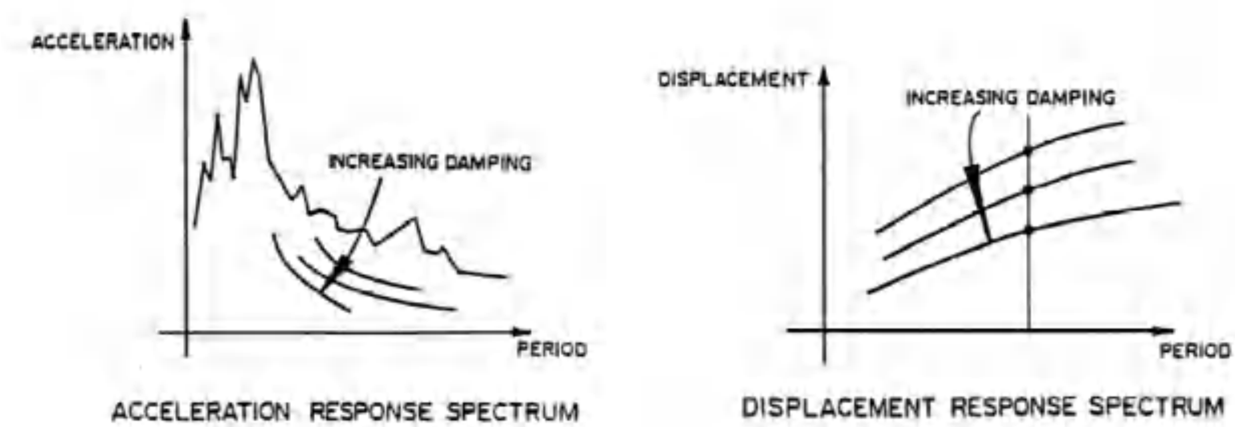
The first type of isolator unit is the laminated elastomeric bearing. The elastomeric material in the bearing is typically natural rubber, which has low horizontal stiffness and low vertical stiffness. The low horizontal stiffness and large deformation capacity of rubber provides the horizontal flexibility necessary for structural seismic isolation. Because of the restoring properties of natural rubber, elastomeric bearings re-center after earthquake shaking. The horizontal stiffness depends primarily on the shear modulus of the rubber, the bonded area of elastomer, the total thickness of rubber, and the lateral displacement of the bearing. To provide sufficient vertical stiffness, the rubber is bonded to thin horizontal steel plates (shims) resulting in a horizontally layered bearing. These steel shims do not affect the horizontal stiffness of the bearing, but prevent the elastomer from bulging when subjected to compressive forces. The shape of the bearing, its aspect ratio, and the amount of vertical load determine the stability properties of the bearing under vertical loads. There are three main types of elastomeric bearings: low damping bearings (LDB), lead rubber bearings (LRB), and high damping bearings (HDB).

The second type of isolator unit is the sliding and/or rolling bearing. Frictional sliding on contact surfaces or rolling provides the horizontal flexibility of such isolation systems, while direct contact of the bearing elements provides the vertical stiffness. Frictional sliding also dissipates energy. Control of motion in sliding and/or rolling bearings is achieved by designing the shape of the contact surfaces (flat or curved), by designing the friction coefficients between the contact surfaces, and by adding supplemental damping. The restoring force needed to re-center a sliding bearing is provided gravity acting through the curved shape of the sliding surface or by a separate device, such as an elastic spring. There are two main types of sliding bearings: flat surface bearings with sliding or rolling surfaces, such as FIP isolator and R.J.

Watson Eradiquake system, and curved surface bearings, such as the Friction Pendulum™ bearings (FPB).

1.3.4 Role of Damping in Seismic Isolation

Energy dissipated by the structural systems through means other than structural deformation or inertial motion is accounted for using the notion of damping. Commonly, a viscous damping model is used to represent such energy dissipation as directly proportional to the velocity of the structure's mass. Figure 1.4 shows the effect of increased damping on both the acceleration response spectrum and displacement response spectrum. Higher damping results in a decrease of both inertial forces and displacements of the structure, fixed-base or isolated. While the velocities of the structure are also reduced, large amounts of viscous damping result in increased magnitude of damping forces transferred to the structure. This damping force is 90 degrees out of phase with respect to the inertia and structure resistance forces.



(a) inertial forces (b) displacements

Figure 1.4 Impact of damping on inertial forces and displacements in structures (AASHTO 2010)

Most seismic isolator units contain some inherent damping. For example, LDB typically have small amounts of damping (approximately 2–4% of critical) while HBD can have large amounts of damping (as high as 20%). This inherent damping is hysteretic (displacement dependent), not viscous (velocity dependent). Damping in an isolated structure can be increased by installing supplemental viscous or hysteretic dampers either at the level of isolation or throughout the superstructure, or both. The effect of supplemental damping on the response of the isolated structure depends on the type of dampers used and the locations of installation. While having the potential to reduce both forces and displacements, supplemental viscous damping may also have adverse effects, including increased forces transmitted to the isolated superstructure and amplified floor spectra (Kelly 1999 and Politopoulos 2007). As a result, the effect of supplemental viscous damping should be investigated carefully on a case-by-case basis.

1.4 Benefits and Challenges of Seismic Isolation for Nuclear Facility Structures

The potential benefits of seismic isolation of nuclear facility structures are numerous.

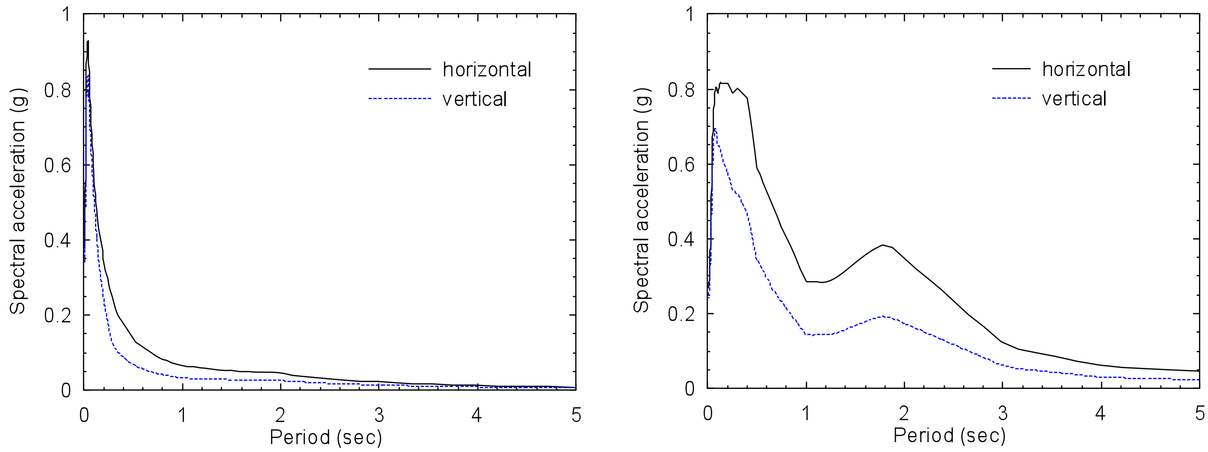
- **Safety**: A properly engineered seismically isolated superstructure will, with high confidence, experience significantly smaller inertial forces than if that same structure was fixed to its base in the same earthquake shaking event. The reduced forces make it easier to design the superstructure to remain essentially elastic: this, in turn, ensures that the deformations of the structure will remain small and in the acceptable range, improving its seismic performance and protecting the internal systems and components. Taken together, reduction in inertial forces and deformations experienced by the superstructure achieved by seismic isolation can substantially reduce the contribution of seismic hazard to the total probability of large radiation does release. This directly increases the safety margins of seismically isolated nuclear facilities. Huang and Whittaker (Huang, et al, 2008) have also found that seismic isolation is an effective strategy to protect the internal systems and components from the effects of air blast and ground shock caused by near-by explosions.
- **Reliability**: Seismic isolation can be custom-engineered to specific nuclear facility site characteristics, both with respect to site seismic hazard exposure and to local soil conditions and configurations. Engineering the substructure and the seismic isolation layer provides an opportunity to engineer the isolated superstructure for essentially the same level of seismic demand regardless of the site characteristics. This, in turn, facilitates standardization of nuclear facility design and makes the seismic behavior of the isolated superstructure more predictable, leading to increased reliability of isolated nuclear facilities. Equally important, standardization facilitates regulatory review and design certification of seismically isolated nuclear facilities.
- **Economy**: While seismic isolation adds an additional structural system between the substructure and superstructure, the added cost and complexity is very likely to be compensated for by the savings realized through reduction of strength, size and complexity of the superstructure. The savings come from the smaller inertial forces and displacements it will experience when seismically isolated. However, a significantly larger economy can be derived from simplified, standardized designs made to suite modular construction methodology. Today, modular construction is being implemented at the level of components and systems: use of seismic isolation enables expansion of the modular concept to the entire isolated superstructure.

1.4.1 Challenges Associated with Seismic Isolation

Successful engineering and implementation of seismic isolation requires careful consideration of a number of factors, several of which are identified below.

- **Non-redundancy**: The seismic isolation layer comprises a large number of essentially identical seismic isolation units (as well as additional seismic isolation components) that act in parallel. The performance of the seismic isolation system will not be significantly affected by failure of a one or a few seismic isolation units or components. However, the seismic isolation layer connects the isolated superstructure to the substructure forming a series system (Figure 1.1). As such, the seismic isolation layer is not redundant. Therefore, engineering measures must be taken to prevent simultaneous or cascading failure of many seismic isolation units due to exceeding their deformation or force capacities. Such engineering measures include deformation limiters achieved by physical (bumpers, stoppers, walls) or mechanical (increasing isolator unit stiffness, slider breaks) means, and additional passive or semi-active dampers. These measures, when engaged, may result in impact or increase in forces transmitted to the isolated superstructure. The engineering objective is to provide for graceful degradation of the seismic isolation layer instead of a cliff-edge sudden seizure of its isolation function.
- **Period Separation**: Seismic isolation is most beneficial when the fundamental vibration period of the superstructure and the fundamental period of the isolated structure are well separated. As a rule of thumb, the period of the isolated structure should be 3 to 4 times that of the superstructure (Kelly 1993, Skinner et al. 1993, Naeim and Kelly 1999). In general, period separation will not be an issue for nuclear power plants because the typical superstructures have very short fundamental horizontal vibration periods (typically less than 0.5 seconds) and the isolated structure would have relatively long fundamental horizontal vibration periods (typically longer than 2 seconds).
- **Soft Soils**: Design of seismic isolation for nuclear facility structures located on soft soils sites will be different from the design for hard soil or rock sites. Figure 1.5 plots response spectra for both a rock and soil site in the Central and Eastern United States: note the spectral amplification in the 2-second period range on the soft soil sites. For the rock site, isolation to a period of 2 seconds and greater will produce dramatic reductions in inertial forces, floor spectral ordinates, drifts and the frequency of core damage (Huang et al. 2008). For the soil site, seismic isolation to a period of 2 seconds would not be recommended because its benefits would be small (not

because it would not work). An isolation system design to a longer period (of 3 seconds or longer) would be required to realize the full benefits of seismic isolation.



(a) rock site (b) soil site

Figure 1.5 Design basis earthquake spectra for the Central and Eastern United States (Huang et al. 2009)

- **Long Period and Near-Field Excitation**: Seismically isolated structures can be sensitive to long period (low frequency) earthquake excitation because such excitations may contain substantial energy in the frequency range close to the fundamental vibration frequency of the isolated structures. In such case, potential for resonant response must be investigated by assessing the duration of such ground motions and the amount of damping provided by the seismic isolation system. Similarly, on sites close to faults, analysis of the isolated structure must account for near-field and directivity effects in order to mitigate the potential for large displacements at spectral periods close to the fundamental vibration period of the isolated structures and the ability of the isolators to freely displace in the predominant direction of excitation. Seismic hazard analysis of seismically isolated structures should be used to identify the potential for both long-period and near-field excitation.
- **Vertical Acceleration**: While the isolation system decouples the superstructure from horizontal ground motion, it does not significantly alter the response of the superstructure to vertical ground motion. Therefore, the superstructure itself and the internal systems and components attached to the superstructure will remain exposed to potentially large vertical accelerations. Furthermore, some isolation systems allow for uplift, which is necessarily followed by re-engagement or impact. Propagation of such impact excitation may exacerbate the vertical acceleration demand on equipment attached to the superstructure. Mosqueda et al. (2004),

Warn and Whittaker (2006a) and Huang et al. (2009) discuss the effects of vertical acceleration on the response of seismically isolated structures. Additional research on the impact of vertical acceleration on nuclear facility SSCs is needed.

- **Nonlinearity**: Both flexible and sliding seismic isolation units are inherently nonlinear at the deformation levels of interest (Figure 1.3). The flexible bearing, such as the LRB, HDB, and FPB seismic isolation devices, derive their nonlinearity from the material characteristics of the rubber used to make them. The sliding bearings, such at the FPS, derive their nonlinearity from the intermittent nature of friction at the sliding surfaces and the second order effect of gravity on curved sliding surfaces. As a result, linear elastic methods of analysis, including both time and frequency domain analysis methods, are not appropriate for the final design of such isolation systems. Chapters 3 and 4 of this report discuss the issues of modeling and analysis in more detail.
- **Detailing**: Because of potentially large relative horizontal displacements at the isolation interface, any component or system (pipes and conduits, for example) that spans the isolation interface must be designed to accommodate these large displacements. In addition, a seismic gap (or moat) must be provided along the perimeter of an isolated structure to accommodate these relative displacements (see Figure 1.1). It is crucial that the seismic gap be kept free of any objects that may obstruct the movement of the isolated structure during an earthquake.

1.5 History of Seismic Isolation

The ideas of decoupling a structure from the damaging effects of an earthquake by rocking or sliding date back to the Classical period of Greece and Persia (600 to 300 BC) and the 12th century Japan. Seismic isolation as an earthquake protection method reappeared in early 20th century (Naeim and Kelly, 1999). Only recently, however, have advances in manufacturing processes of seismic isolation units and dynamic analysis methods made seismic isolation practical. Development of bridge elastomeric and sliding bearings 1950's was a precursor to development of modern seismic isolation devices. In the 1970's, researchers in New Zealand developed and tested prototype models of simple elastomeric bearings. Since then, researchers and engineers in dozens of countries have refined the technology and successfully implemented it in hundreds of buildings and bridges.

Since the first rudimentary design was installed in a Macedonian school in 1969 (Kelly Class notes Ch 1), approximately 7,000 buildings worldwide have been constructed or retrofitted to include the new technology (Martelli and Forni, 2009). Most of these structures have been designed and built within the last 15 years, after the efficacy of the first modern isolators was observed in response to the large ground motions during the 1994 Los Angeles and 1995 Kobe earthquakes. Since that time, the growth of base isolation in Japan has been staggering as it has been widely accepted as an effective structural engineering tool to limit earthquake-induced damage. Although the United States has been one of the forerunners in creating and testing base isolation technology, implementation has lagged behind that of other countries like Japan, China, and Russia, as shown in Figure 1.6, due to prohibitive design code measures.

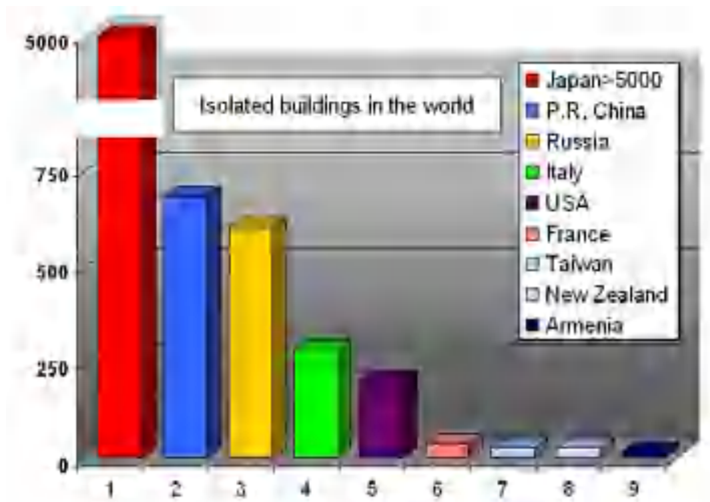


Figure 1.2 Use of base isolation by country (Martelli and Forni, 2009).

In the United States, engineers have used seismic isolation in many different types of structures, ranging from highway bridges to government buildings. Many of these structures support essential services (hospitals and emergency response centers, for example) and therefore require higher levels of performance during an earthquake. Modern design codes for conventional (non-nuclear) structures include provisions that define the design of seismic isolation for such structures. A summary of these design procedures is presented in Chapter 5. Because of a number of issues unrelated to the design code implementation of the concept of seismic isolation, the technology is not yet economical for most typical applications, resulting in only a handful of seismically isolated structures constructed each year in the United States.

Design and evaluation of seismically isolated structures in the US is regulated in conventional building codes, such as IBC 2010 , ASCE 7 and AASHTO.

1.5.1 Recorded Performance of Seismically Isolated Structures

The following subsections briefly summarize the response of isolated structures during several major earthquakes.

1.1.1.1. 1994 Northridge Earthquake

The 1994 Northridge Earthquake shook several seismically isolated structures in the Los Angeles area. One of these was the USC University Hospital, an 8-story structure with highly irregular plan. During the Northridge Earthquake, the peak ground acceleration (PGA) recorded at the site was 0.37g. The isolation system (a combination of LRB and LDB) reduced the peak horizontal acceleration at the roof to 0.21g, almost a 50% decrease from the PGA. As a result of its isolation system, the hospital remained elastic and is expected to perform well in future design basis earthquakes (Nagarajaiah and Xiaohong 2000).

1.1.1.2. 1995 Kobe Earthquake

The 1995 Kobe Earthquake shook several isolated structures in Kobe, Japan. At Tohoku University, two full-scale demonstration structures were built side by side. One was isolated and the other was not. The isolators served to reduce the accelerations in the isolated superstructure by a factor of 10 relative to the fixed base structure.

1.5.2 Seismically Isolated Nuclear Facility Structures

Application of base isolation spread to the nuclear power industry with the completed construction of plants in Cruas, France and Koeberg, South Africa in 1983 and 1984, respectively. Both plants utilized neoprene pads and sliders, a system later deemed inappropriate for seismic application in the United States and subsequently rejected in future plans in favor of newer isolation types (Malushte and Whittaker, 2005). To date, there are 6 NPP units utilizing base isolation, all in France and South Africa.

The US efforts toward seismically isolated nuclear facility structures during the 1980's culminated in NRC pre-application review of the GE PRISM (NUREG 1368). While this review found sufficient data existed at the time to warrant further development of a design application, the PRISM project did not continue for other reasons. Therefore, no regulatory guidance for design and evaluation of seismically isolated nuclear facility structures exists today. An ASCE committee working on a revision of ASCE 4 document will provide some guidance for design of seismically isolated nuclear facility structures in accordance with the ASCE 43-05 design code.

A special subcommittee of the Japan Electrical Association developed and Electrical Technical Guide JEAG 4614-2000 (JEAG, 2000) to provide technical guidance on seismic isolation systems for structural safety and design of nuclear power plants. To date, this is the only seismic isolation code document directly applicable for nuclear facility structures.

1.6 Report Content

The following report is divided into 5 chapters. Chapter 2 discusses performance objectives associated with seismic isolation. Chapter 3 provides detailed information regarding the two main types of isolators. Chapter 4 discusses current techniques for modeling and analyzing seismically isolated structures. Chapter 5 summarizes the procedures currently used to design and detail seismically isolated structures. A list of references follows Chapter 5. Appendix A contains a numerical example that demonstrates the benefits of seismically isolating a single-degree-of-freedom system.

2. PERFORMANCE OBJECTIVES AND ACCEPTANCE CRITERIA USED IN SEISMIC ISOLATION DESIGN FOR NUCLEAR FACILITY STRUCTURES

This chapter provides information about the intended design approach, performance objectives and acceptance criteria for the design of seismically isolated nuclear facility structures.

It is assumed that the seismic isolator units will be treated as structural components and thus be in the domain of structural design codes such as ASCE 4 (ASCE 2010b), ASCE 7 (ASCE 2010a) and ASCE 43 (ASCE 2006).

2.1. Design Basis

A seismically isolated nuclear facility structure has three components: the superstructure, the isolation layer, and the foundation embedded into the underlying soil. The design basis for all three components is drawn from ASCE 4 and ASCE 43.

ASCE 43 defines the Design Basis Earthquake (DBE) using a Probabilistic Seismic Hazard Assessment (PSHA) to derive a Uniform (or Equal) Hazard Response Spectrum (UHRS) for the site and modifies it further using a Design Factor. The Design Factor is calibrated to achieve an annual seismic core damage frequency of 10^{-6} considering the failure probabilities inherent to current design codes and the design goals proposed in ASCE 43.

The goal is to *reasonably* achieve both of the following design objectives:

1. Less than 1% probability of unacceptable performance for the DBE ground motion (defined in Section 2.0 of ASCE 43-05).
2. Less than 10% probability of unacceptable performance for 150% of the DBE ground motion (defined in Section 2.0 of ASCE 43-05).

A probabilistic design method proposed in ASCE 43 simultaneously achieves these two objectives. A performance goal has a quantitative and a qualitative part. Quantification of a performance goal is probability-based: a mean annual hazard exceedance frequency H_D (associated with the UHRS) and a Target Probability Goal P_F (in terms of annual frequency of exceeding acceptable behavior) are defined. The (H_D, P_F) pair defines the quantitative probabilistic performance goal, expressed conveniently as a probability ratio $R_P = H_D/P_F$. The qualitative performance goal is defined using the Limit State concept to describe qualitatively the acceptable structural behavior. The qualitative and quantitative portions of a performance goal are combined in the definition of the Target Probability Goal P_F. The limit state description is used to define the event of exceeding acceptable behavior.

ASCE 43 defines 5 Seismic Design Categories (SDC). Categories 3, 4 and 5 are associated with nuclear facility structures, systems and components (SSC) through ANSI/ANS 2.26. Target Probability Goals are listed in Table 2-1.

Table 2-1 Target Probability Goals (from ASCE 2006)

Seismic Design Category	SDC-3	SDC-4	SDC-5
P_F	10 x 10^{-5}	4 x 10^{-5}	1 x 10^{-5}
H_D	40 x 10^{-5}	40 x 10^{-5}	10 x 10^{-5}
$R_P = H_D/P_F$	4	10	10

ASCE 43 defines 4 Limit States based on structural deformations to describe levels of acceptable structural damage. These damage states range from significant damage with a structure close to collapse (state A) to no significant damage with a structure in operational condition (state D).

The target performance objective for nuclear power plant structures is SDC-5D. This performance objective is associated with an annual probability of unacceptable performance of $1x10^{-5}$. Acceptable performance is described as "essentially elastic behavior." This performance description is associated with conventional structural framing such as concrete shear walls and moment frames.

2.2. Acceptable Performance for Seismically Isolated Structures

Acceptable performance for an isolated structural system is described in this section. The definitions provided below are those of the seismic isolation provisions of ASCE 4 and described in Huang et al. (2009).

In seismically isolated nuclear structures, the accelerations and deformations in structures, systems and components (SSCs) are relatively small. The SSCs are expected to remain elastic for both DBE shaking and beyond design basis shaking. As such, unacceptable performance of an isolated nuclear structure will most likely involve either the failure of isolation bearings or impact of the isolated superstructure and surrounding building or geotechnical structures.

2.2.1. Isolation System

Acceptable performance of an isolator unit depends on its type (i.e. LDB, LRB, HDB, or FPB). The behavior of each type will be described in more detail in Chapter 3 of this report. For the purposes of discussion herein, unacceptable performance will be defined as:

1. Permanent damage to the isolation device, such as tearing or disassembly, such that it is unable to perform its gravity load carrying function.
2. Exceeding the displacement limit of the device, such that simultaneous failure of a large number of devices in the isolation layer does not occur.

The acceptance criteria for isolator units should be set such that there is a high confidence of low probability of failure and that the overall target annual seismic core damage frequency does not exceed the value required by NRC.

2.2.2. Superstructure and Foundation

The superstructure and foundation of the isolated structure will still comply with the SDC-5D performance objective set forth in ASCE 43.

2.3. Acceptance criteria

The acceptance criteria for the isolated structure will address the superstructure and foundation separately from the isolation system. The acceptance criteria for the superstructure and foundation may be adopted from ASCE 43 for non-isolated structures. The acceptance criteria for isolation system should be based on a comparison of the available deformation capacity of the isolator units and the expected deformation demand imposed on them.

For the purpose of the Huang et al. (2009) study and the seismic isolation provisions of ASCE 4, three performance statements are made for achieving the two performance objectives set forth in Section 1.3 of ASCE 43-05:

1. Individual isolators shall suffer no damage in DBE shaking.
2. The probability of the isolated nuclear structure impacting the surrounding structure (moat) for 100% (150%) DBE shaking is 1% (10%) or less.
3. Individual isolators shall sustain gravity and earthquake-induced axial loads at 90th percentile lateral displacements consistent with 150% DBE shaking.

Performance statement 1 can be realized by production testing of each isolator supplied to a project for median DBE displacements and co-existing gravity and earthquake-induced axial forces. Analysis can be used in support of performance statement 2 provided that the isolators are modeled correctly and the ground motion representations are reasonable. Performance statement 3 can be realized by prototype testing of a limited number of isolators for displacements and co-existing axial forces consistent with 150% DBE shaking, noting that an isolation system may contain on the order of a hundred isolator units and failure of the system would involve the simultaneous failure of a significant percentage of the isolators. Fulfilling of performance statement 3 may require engineering measures to prevent simultaneous or cascading failure of many seismic isolation units due to exceeding their deformation or force capacities. Such engineering measures include fail-safe system such as deformation limiters achieved by physical (bumpers, stoppers, walls) or mechanical (increasing isolator unit stiffness, slider breaks) means, and additional passive or semi-active dampers. These measures, when engaged, may result in impact and/or gradual increase in forces transmitted to the isolated superstructure. Possible failure of isolator devices to carry gravity load may impose additional demands on the superstructure isolation mat and the foundation mat. Assuming that simultaneous failure of multiple isolators is prevented by the fail-safe systems described above, it may be sufficient to design the superstructure and the foundation mats to span the gravity loads over a single isolator.

The deformation demand for DBE and 150% DBE shaking should be evaluated based on a comprehensive evaluation of the seismic hazard and demand statistical distributions using appropriate mathematical models of the isolated structure. Simplified models, such as dynamic systems with few degrees of freedom, may be used. This is discussed in Huang et al. (2008, 2009) and summarized in Chapter 4 of this report.

Based on the statistical evaluation of the dispersion of the demand and capacity data, appropriate amplification of demand and reduction of capacity should be performed. This should be done such that the target performance objectives of ASCE 43 are satisfied and that the implied annual frequency of seismic core damage required by NRC is maintained. Huang et al. (2009) present such calculations for isolated nuclear structures for rock and soil sites in the Central and Eastern United States and a rock site in the Western United States.

3. MECHANICS OF SEISMIC ISOLATORS

This chapter provides detailed information about the properties and performance of modern seismic isolators.

3.1. Basic Mechanics of Seismic Isolators

This section presents the basic mechanics of the two main types of seismic isolators: elastomeric and sliding bearings.

3.1.1. Elastomeric Bearings

Elastomeric bearings typically use either natural or synthetic rubber as their elastomeric material. Rubber is an ideal material because it has high elastic deformation capacity and extremely high elongation at break. It is also virtually incompressible (Constantinou et al. 2007).

Vulcanization, or curing, is the process by which raw rubber is converted from a plastic state to an essentially elastic state. The conditions during vulcanization influence the rubber's strength, elasticity, resistance to solvents, and sensitivity to temperature changes. Often chemicals are added to the rubber during vulcanization. These additives can produce a variety of effects. Accelerators shorten the duration of heating or reduce the amount heat required for vulcanization. Fillers modify the mechanical properties of the rubber, including the hardness, stiffness, elongation at break, creep and relaxation characteristics, and fatigue life. Anti-ozonants protect the rubber from cracking due to ozone attack. Anti-oxidants delay degradation caused by exposure to oxygen and also reduce aging effects (Constantinou et al. 2007).

Elastomeric bearings are constructed by bonding sheets of rubber to thin steel plates called shims. These shims, which do not affect the horizontal stiffness of the bearing, prevent the rubber from bulging under compressive load and also increase the vertical stiffness of the bearing (by confining the rubber). Insufficient vertical stiffness of isolators may result in rocking of the superstructure during an earthquake. Figure 3.1 shows the cross-section of a typical elastomeric bearing.

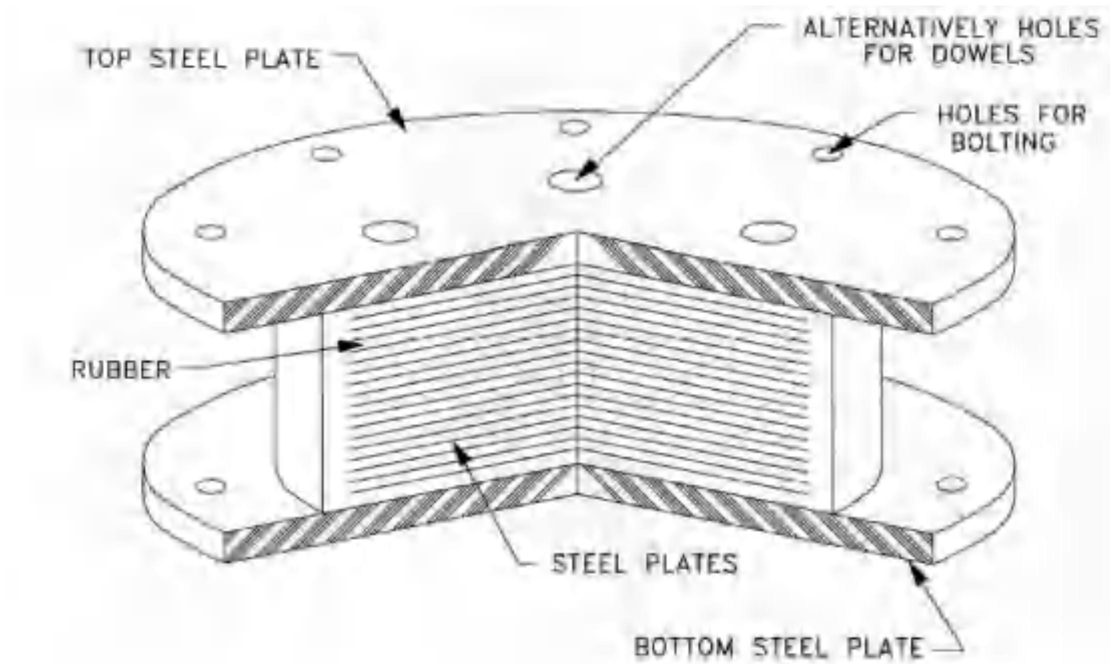


Figure 3.1 Cross-section of a typical elastomeric bearing (Constantinou et al. 2007)

The most important mechanical property of an elastomeric bearing is its horizontal stiffness, K_H, which can be expressed as (Naeim and Kelly 1999):

$$K_H = \frac{G \cdot A}{T_r} \tag{3.1}$$

where G is the strain-dependent shear modulus of the elastomer, A is the bonded area of the elastomer, and T_r is total thickness of the elastomer (equal to the product of the total number of layers of rubber, n_r, and the thickness of each layer, t_r). The shear modulus of unfilled natural rubber typically ranges from 0.45 – 0.70 MPa (65 – 100 psi) at 100% shear strain (Constantinou et al. 2007).

The period of an elastomeric bearing, T_b, can be expressed as:

$$T_b = 2\pi\sqrt{\frac{W}{g \cdot K_H}} = 2\pi\sqrt{\frac{W \cdot T_r}{g \cdot G \cdot A}} \tag{3.2}$$

where W is the vertical load carried by the bearing, g is the acceleration caused by gravity (9.81 m/s^2 or 32.2 ft/s^2), and all other terms defined previously. Because the shear modulus depends on the strain, the period of the bearing depends on the displacement of the bearing.

Another important property of an elastomeric bearing is its shape factor, S. The shape factor is the ratio of the loaded area to the load-free area of a single layer of rubber (thickness = t_r). For a cylindrical bearing with diameter D, the shape factor is:

$$S = \frac{\frac{\pi}{4} D^2}{\pi \cdot D \cdot t_r} = \frac{D}{4t_r} \tag{3.3}$$

Typically, the shape factors of elastomeric bearings used in seismic applications are greater than 10 and often substantially greater than 10. The shape factor has significant influence on the buckling and overturning capacities of a bearing at large lateral deformation. Large shape factors help minimize shear stress caused by compression and also increase the buckling load capacity of the bearing (Constantinou et al. 2007).

3.1.2. Sliding Bearings

Similar to an elastomeric bearing, the most important mechanical property of a sliding bearing is its horizontal stiffness, K_H. The horizontal stiffness of a Friction Pendulum™ bearing, the most common type of sliding bearing, is:

$$K_H = \frac{W}{R} \tag{3.4}$$

where W is the vertical load supported by the bearing and R is the radius of curvature of the bearing. The period of a Friction Pendulum™ bearing, T_b, is:

$$T_b = 2\pi \sqrt{\frac{W}{g \cdot K_H}} = 2\pi \sqrt{\frac{R}{g}} \tag{3.5}$$

Notice that the period depends only on the radius of curvature (R) and not on the size (A, T_r) or vertical load (W).

3.2. Types of Seismic Isolators

There are many different types of seismic isolators. This section provides detailed information about four of the most popular and frequently used isolators. Three are elastomeric bearings (low damping bearings, lead rubber bearings, and high damping bearings) while one is a sliding bearing (Friction Pendulum™ bearings).

The text of this section is excerpted from Huang et al. (2009), which in turn is based on multiple sources, including Zayas et al. (1987, 1990), Kelly (1993), Skinner et al. (1993), Naeim and Kelly (1999), Constantinou et al. (1999, 2007) and Fenz and Constantinou (2008a). Constantinou et al. (2007) and Naeim and Kelly (1999) provide substantial information on the construction, analysis and design of elastomeric and sliding isolation systems for the interested reader.

3.2.1. Low Damping Bearings

Low damping bearings (LDB) use (unfilled) natural rubber as their elastomeric material. LDB have damping ratios less than 10%, and often less than 5% (Constantinou et al. 2007). Typically, LDB have damping ratios between 2% and 4% of critical depending on the bearing displacement (Kasalanti 2009). Figure 3.1 shows the cross-section of a typical LDB.

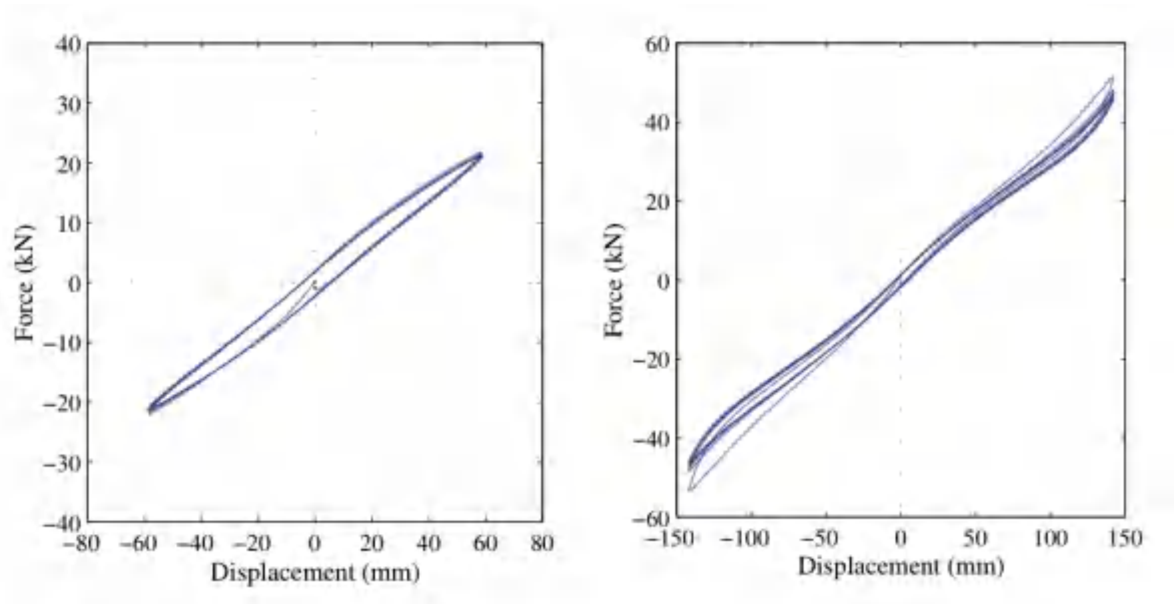
3.2.1.1. Mechanical Properties

The following three sections discuss the *general* behavior of LDB when subject to shear, compression, and tension loads. The mechanical properties of a *specific* LDB depend on many factors, including the composition of the rubber, the conditions during vulcanization, and the geometry of the bearing. Therefore, the exact behavior of the *specific* LDB used in design should be established and verified by testing.

3.2.1.1.1. Shear

Figure 3.2 shows the horizontal force-deformation relation for a typical LDB at two levels of shear strain. Figure 3.2(a) shows the results of a test at small shear strain (75%) while Figure 3.2(b) shows the results of a test at large shear strain (175%). For both tests, the behavior is essentially linear elastic. However, at large shear strain (approximately 175%) the horizontal stiffness of the bearing increases modestly. In general, low damping natural rubber stiffens at shear strains larger than 200% (Thompson et al. 2000, Morgan et al. 2001). Therefore, for shear strains less than 200%, LDB can be modeled as linear elastic elements. The shear modulus of LDB ranges from 0.45 – 0.70 MPa (65 – 100 psi) at 100% shear strain.

The area enclosed by the force-deformation relation (hysteresis loop) can be used to approximate the damping ratio of the bearing. The hysteresis loops in Figure 3.2 enclose a small amount of area, reflecting the fact that LDB have little inherent damping (typically between 2% and 4% of critical).



(a) 75% shear strain (b) 175% shear strain

Bearing properties: diameter = 250 mm; shape factor = 9.8; effective damping ratio < 5%; effective shear modulus ≈ 0.65 MPa
Testing properties: frequency = 1 Hz; ambient temperature = 20°C; axial force = 28kN

Figure 3.2 Force-deformation relation for a typical low damping bearing (Constantinou et al. 2007)

3.2.1.1.2. Compression

LDB (and elastomeric bearings in general) exhibit high vertical stiffness when subjected to compressive loading. This is due to the incompressibility of natural rubber and also the confined conditions of stress created by the steel shims (Constantinou et al. 2007).

3.2.1.1.3. Tension

For small tensile loads, LDB (and elastomeric bearings in general) exhibit stiffness comparable to the stiffness in compression. But as the tensile force increases, the bearing begins to develop small cracks in the volume of the rubber (a process called cavitation). This typically occurs at tensile stress ranging from 1.5 – 2.5 MPa (0.20 – 0.35 psi) but depends on the exact composition of the rubber. Once cavitation occurs, confinement is lost and the stiffness of the bearing decreases substantially (Constantinou et al. 2007). For this reason, elastomeric bearings are typically designed to avoid developing tensile forces.

3.2.1.2. Variation in Mechanical Properties

The mechanical properties of LDB are affected by many parameters. The following sections describe the *general* effect each parameter has on the mechanical properties of LDB. The *exact* effect of each parameter should be established for the *specific* LDB used in design.

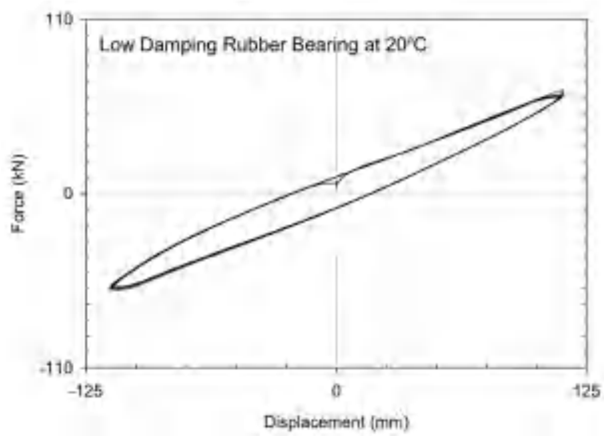
3.2.1.2.1. Temperature

Low temperatures substantially increase the stiffness, strength, and damping of LDB. Figure 3.3 demonstrates this effect. Figure 3.3(a) shows the force-deformation relation for an LDB at 20°C (68°F), a typical operating temperature for bearings with interior exposure (i.e. inside a building or structure). Figure 3.3(b) shows the force-deformation relation for an LDB at –26°C (–15°F). Both stiffness and damping increase by approximately 50–60% as a result of the drop in temperature. When the temperature falls below the glass transition temperature of natural rubber (–55°C or –67°F), the material becomes brittle and many of its mechanical and physical properties undergo significant and rapid changes.

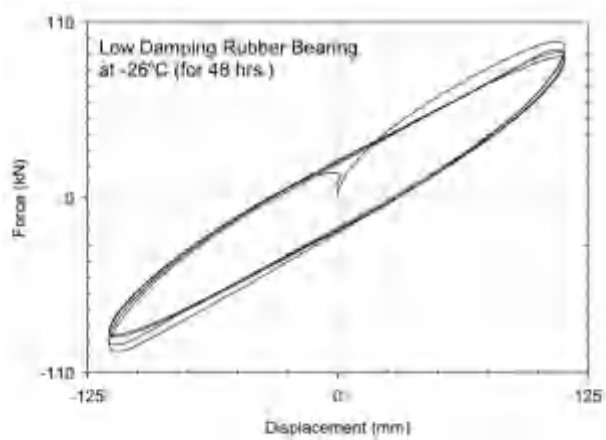
The duration of exposure to low temperature also influences the stiffness and damping of LDB. As the duration of exposure increases, both stiffness and damping increase. This time-dependent stiffening is more pronounced for LDB with larger shear moduli (Constantinou et al. 2007).

Tests performed by Constantinou et al. (2007) demonstrate that high temperatures have insignificant effect on the stiffness and damping of LDB, producing only minor reductions in both quantities.

It is important to note that for most nuclear applications, the isolation system will reside within a controlled environment. Therefore, large variation in ambient temperature is not expected.



(a) 20°C



(b) –26°C

Bearing properties: shape factor = 10.7; width = 411 mm; height = 411 mm

Testing properties: frequency = 0.35 Hz; amplitude = 113 mm; bearing pressure = 6.9 MPa

Figure 3.3 Effect of temperature on mechanical properties of LDB (Constantinou et al. 2007)

3.2.1.2.2. Axial Load

The mechanical properties of LDB are sensitive to the axial load carried by the bearing. In general, as the axial force in the bearing increases, its horizontal stiffness decreases. This stiffness reduction is most pronounced in bearings with low shape factors (Ryan et al. 2005).

3.2.1.2.3. Load History

In general, elastomeric bearings exhibit higher strength and stiffness when tested for the first time. These properties are called virgin or unscragged. After several cycles of continuous testing, the strength and stiffness of the bearing decrease to a stable but lower value. These properties are termed scragged. Experiments performed by Thompson et al. (2000) show that the extent of scragging increases as the shear modulus of the rubber decreases and as the effective damping increases. As a result, LDB are generally less susceptible to scragging effects than HDB because they have lower values of effective damping.

For many years, engineers believed that testing damaged the molecular structure of rubber and, consequently, that scragged properties of bearings were permanent. Recent experience has shown, however, that bearings recover their unscragged properties over time. The required length of time depends on the rubber compound, the extent of its curing, and the ambient temperature. As a result, unscragged properties can be used in the analysis and design of isolation systems with elastomeric bearings (Constantinou et al. 2007).

Tests performed by Constantinou et al. (2007) demonstrate that the number of cycles of loading does not significantly influence the mechanical properties of LDB. Also, scragging effects are typically negligible.

3.2.1.2.4. Loading Rate

Tests performed by Constantinou et al. (2007) show that the rate of loading does not affect the mechanical properties of LDB significantly.

3.2.1.2.5. Aging and Environmental Effects

Natural rubber can stiffen over time as it continues to vulcanize. As a result, the effective shear modulus of the bearing may increase. The magnitude of this increase depends on several factors, including the completeness of initial vulcanization at time of manufacturing and the ambient temperature while in service (Thompson et al. 2000; Morgan et al. 2001). If fully cured natural (low damping) rubber is used, the age-related stiffening of the bearing will be minor.

In addition, the mechanical properties of natural rubber can degrade with prolonged exposure to oxygen and ozone. In bulk rubber (such as the rubber in LDB), only a shallow surface area is affected. To prevent this, elastomeric bearings are normally fabricated with a protective cover layer of rubber containing various anti-ozonants and anti-oxidants (Constantinou et al. 2007).

3.2.2. Lead Rubber Bearings

A lead rubber bearing (LRB) is essentially a low damping bearing with vertical holes into which lead plugs are inserted. Commonly, a single lead plug is inserted into the center of the bearing (see Figure 3.4). This lead core, which is typically 15–33% of the total diameter, increases the damping (energy dissipation) of the bearing. The amount of energy dissipated depends on the size of the lead core and the degree of its confinement.

Lead is an ideal material because it has high horizontal stiffness before yielding and then behaves perfectly plastic after yielding. Also, it is the only common metal for which the processes of recovery, re-crystallization, and grain growth occur simultaneously at room temperature, so it can forever recover its original mechanical properties following inelastic action (Skinner et al. 1993).

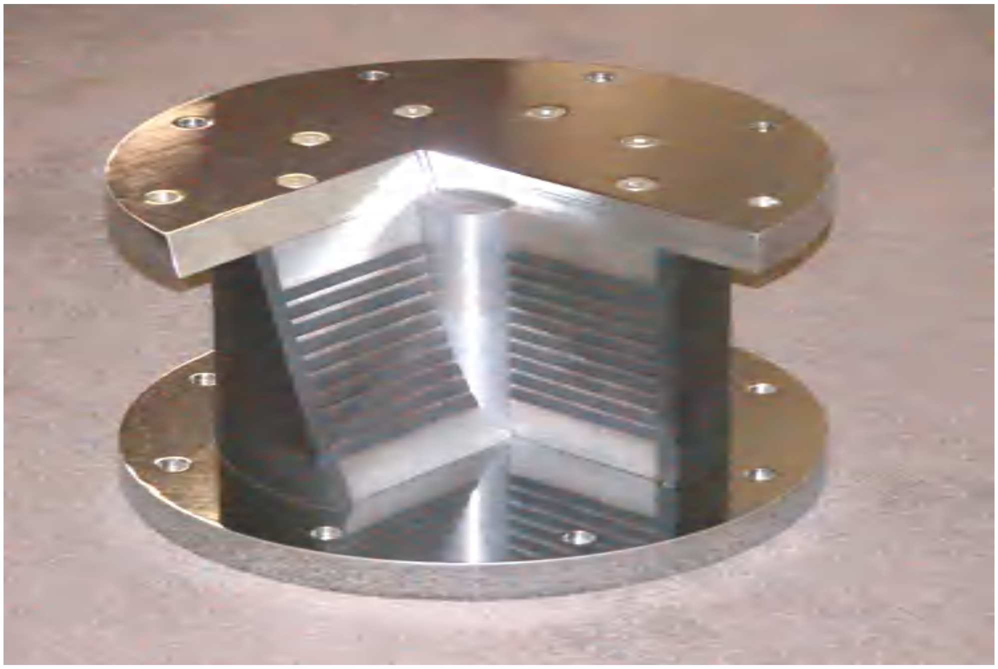


Figure 3.4 A cut-away view of a lead rubber bearing (courtesy of Dynamic Isolation Systems, Inc.)

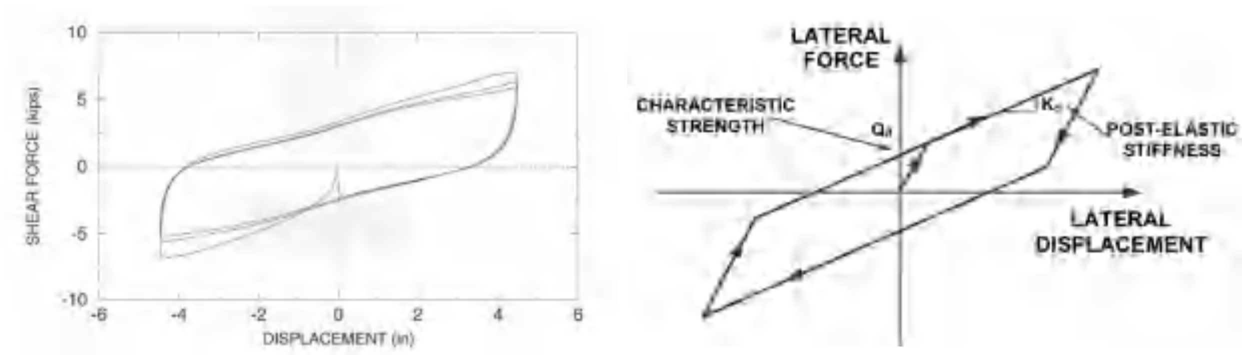
3.2.2.1. Mechanical Properties

The following three sections discuss the *general* behavior of LRB when subject to shear, compression, and tension loads. The mechanical properties of a *specific* LRB depend on many factors, including the composition of the rubber, the conditions during vulcanization, and the geometry of the bearing and lead plug. Therefore, the exact behavior of the *specific* LRB used in design should be established and verified by testing.

3.2.2.1.1. Shear

Figure 3.5 shows the horizontal force-deformation relation for a typical LRB. Figure 3.5(a) shows the actual force-deformation relation of an LRB tested in the lab, whereas Figure 3.5(b) shows the idealized force-deformation relation typically used in design. In Figure 3.5(b), the characteristic strength Q_d is governed by the dynamic yield strength, size, and degree of confinement of the lead core. The post-elastic stiffness K_d is a function of the shear modulus of the rubber (G), the bonded area (A_r), and the total thickness (T_r) of the rubber (see Equation 3.1). The maximum design shear strain of LRB varies as a function of manufacturer but generally ranges between 125% and 200% (Constantinou et al. 2007).

The force-deformation relation (hysteresis loop) for LRB (Figure 3.5) encloses more area than LDB (Figure 3.2), reflecting the fact the LRB have more inherent damping than LDB.



(a) actual (b) idealized

Bearing properties: bearing diameter = 184 mm; diameter of lead plug = 38 mm

Testing properties: frequency = 0.5 Hz; amplitude = 250% shear strain; bearing pressure = 6.9 MPa

Figure 3.5 Force-deformation relation for a typical LRB (Constantinou et al. 2007)

3.2.2.1.2. Compression

The lead core does not significantly affect the behavior of LRB when subjected to compression. Therefore the behavior of LRB in compression is similar to LDB. Please refer to Section 3.2.1.1.2 of this report for more information.

3.2.2.1.3. Tension

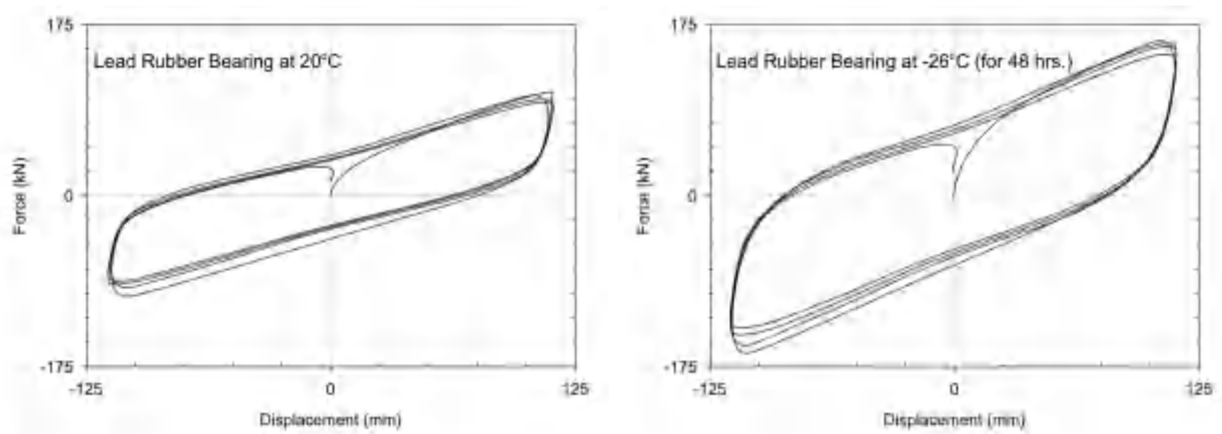
The lead core does not significantly affect the behavior of LRB when subjected to tension. Therefore the behavior of LRB in tension is similar to LDB. Please refer to Section 3.2.1.1.3 of this report for more information.

3.2.2.2. Variation in Mechanical Properties

The mechanical properties of LRB are affected by many parameters. The following sections describe the *general* effect each parameter has on the mechanical properties of LRB. The *exact* effect of each parameter should be established for the *specific* LRB used in design.

3.2.2.2.1. Temperature

As previously discussed, changes in temperature significantly affect the mechanical properties of LDB. Because an LRB is essentially an LDB with a lead core, the observations made in Section 3.2.1.2.1 of this report also pertain to the rubber portion of LRB. Figure 3.6 shows the force-deformation relation for an LRB tested at two different ambient temperatures. Again, low temperatures substantially increase the stiffness, strength, and damping of LRB.



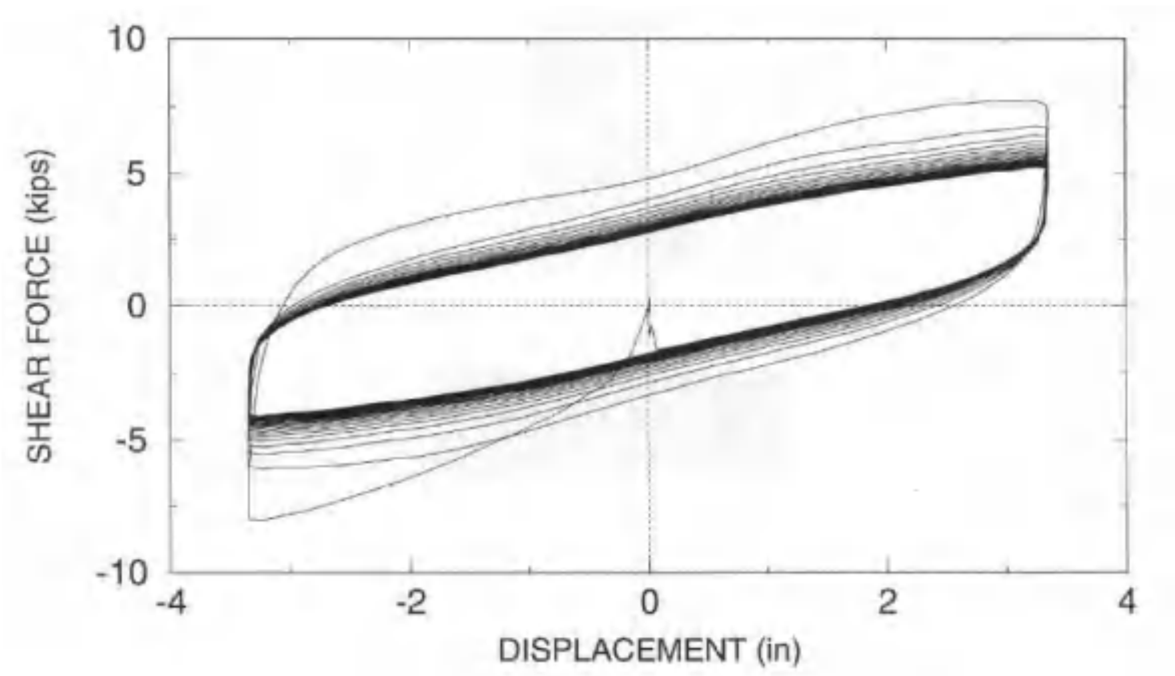
(a) 20°C (b) –26°C

Bearing properties: shape factor = 10.7; width = 411 mm; height = 411 mm; diameter of lead plug = 70 mm
Testing properties: average bearing pressure = 6.7 MPa; amplitude = 113 mm; frequency = 0.35 Hz

Figure 3.6 Effect of temperature on mechanical properties of LRB (Constantinou et al. 2007)

Additionally, the mechanical properties of lead are affected by changes in temperature. As lead heats, its effective yield stress decreases, which reduces the amount of energy dissipated per cycle (Constantinou et al. 2007). The lead core can heat as a result of prolonged exposure to elevated ambient temperature (see Figure 3.6). The lead core can also heat as a result of repeated cycles of motion. Energy dissipated by the lead core takes the form of heat. Because steel has much higher thermal conductivity than natural rubber, this heat primarily escapes from the bearing through the steel shims and the steel end plates (see Figures 3.4 and 3.1). If the bearing undergoes repeated cycles of motion, heat builds up faster than it can escape, which raises the temperature of the lead core. The magnitude of rise depends on the size of the lead core. Eventually the temperature of the lead core stabilizes, at which point the mechanical properties of the bearing also stabilize.

Figure 3.7 demonstrates this effect. The bearing in Figure 3.7 was subjected to 25 cycles of motion at ±150% shear strain. Both the yield strength of the lead core and energy dissipated per cycle decrease as the number of cycles increases and the lead core heats. After many cycles, the temperature of the lead core stabilizes, at which point both the yield strength of and energy dissipated by the bearing stabilize.



Bearing properties: bearing diameter = 184 mm; diameter of lead plug = 38 mm
Testing properties: frequency = 0.5 Hz; amplitude = 150% shear strain; bearing pressure = 6.9 MPa

Figure 3.7 Force-displacement relation for LRB subject to 25 cycles of motion at ±150% shear strain

3.2.2.2.2. Axial Load

Tests performed by HITEC (1999) indicate that the mechanical properties of LRB are sensitive to changes in axial load. In general, as axial force increases, horizontal stiffness decreases while damping increases, though the effect is minor. Constantinou et al. (2007) report that the compressive axial load carried by the bearing does not affect the mechanical properties of LRB significantly.

3.2.2.2.3. Load History

The mechanical properties of LRB depend on the loading history. As the number of cycles increase, the yield stiffness of the lead core and energy dissipated per cycle both decrease. This is due to rising temperatures in the lead core (which is described fully in Section 3.2.2.2.1 of this report).

3.2.2.2.4. Loading Rate

As previously discussed, the mechanical properties of (unfilled) natural rubber are largely insensitive to loading rate. The same cannot be said of lead: its yield strength increases as the loading rate increases. If however, the bearing is subjected to many cycles of motion, the yield strength of the lead core will eventually decrease as its temperature increases.

3.2.2.2.5. Aging and Environmental Effects

The mechanical properties of lead are not expected to change significantly during the lifetime of a typical structure (Constantinou et al. 2007). The variation in the mechanical properties of natural rubber was discussed in Section 3.2.1.2.5 of this report.

3.2.3. High Damping Bearings

High damping bearings (HDB) use natural rubber with additives (or fillers) as their elastomeric material. These additives modify the mechanical properties of the rubber, including its hardness, stiffness, elongation at break, creep and relaxation properties. The most important consequence of adding fillers is that it increases the inherent damping of the rubber. HDB have damping ratios greater than 10% of critical, with some bearings having as much as 20% damping. Figure 3.1 shows the cross-section of a typical HDB.

3.2.3.1. Mechanical Properties

The following three sections discuss the *general* behavior of HDB when subject to shear, compression, and tension loads. The mechanical properties of a *specific* HDB depend on many factors, including the composition of the rubber, the conditions during vulcanization, and the geometry of the bearing. Therefore, the exact behavior of the *specific* HDB used in design should be established and verified by testing.

3.2.3.1.1. Shear

Figure 3.8 shows the force-deformation relation for a typical HDB. At small shear strain, the horizontal stiffness of the bearing is relatively high, which helps control wind-induced displacements. At moderate shear strain, the horizontal stiffness decreases, which provides the structure with the horizontal flexibility required during the design basis earthquake. At large strain (approximately 180%), the horizontal stiffness of the bearing increases, which helps control ultimate horizontal displacement in beyond design basis earthquakes. The maximum shear strain depends on the compounding of the rubber, but generally ranges from 200 – 350% (Constantinou et al. 2007). The shear modulus of HDB typically ranges from 0.35 – 1.40 MPa (50 – 200 psi).

Notice the large area enclosed by the force-deformation relation (hysteresis loop) in Figure 3.8, reflecting the inherent damping in HDB. Also notice the large amount of scragging. Tests by Thompson et al. (2000) and Morgan et al. (2001) suggest that the amount of scragging increases as the volume of fillers increase. Therefore, scragging effects are typically more pronounced in HDB than in LDB or LRB.

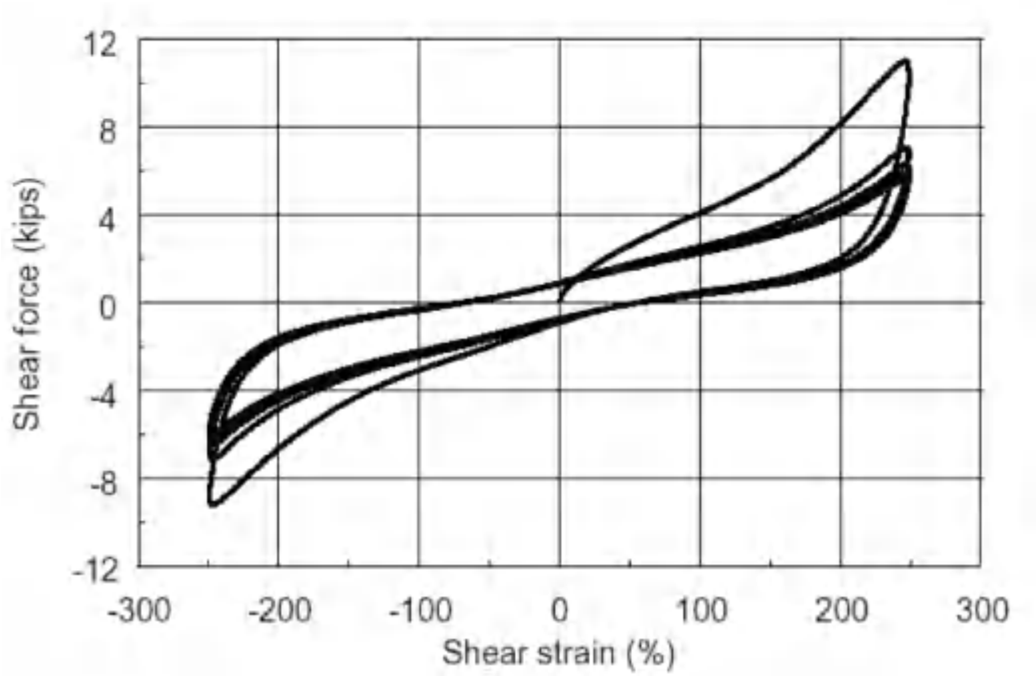


Figure 3.8 Force-displacement relation for typical HDB (Thompson et al. 2000)

3.2.3.1.2. Compression

The addition of fillers does not significantly affect the behavior of HDB when subjected to compression. Therefore the behavior of HDB in compression is similar to LDB. Please refer to Section 3.2.1.1.2 of this report for more information.

3.2.3.1.3. Tension

The addition of fillers does not significantly affect the behavior of HDB when subjected to tension. Therefore the behavior of HDB in tension is similar to LDB. Please refer to Section 3.2.1.1.3 of this report for more information.

3.2.3.2. Variation in Mechanical Properties

The mechanical properties of HDB are affected by many parameters. The following sections describe the *general* effect each parameter has on the mechanical properties of HDB. The *exact* effect of each parameter should be established for the *specific* HDB used in design.

3.2.3.2.1. Temperature

Tests performed by HITEC (1999) indicate that the mechanical properties of HDB are sensitive to changes in temperature. At elevated temperatures, both horizontal stiffness and damping decrease, though the effect is minor (less than 10% change from the mechanical properties at normal operating temperature).

3.2.3.2.2. Axial Load

Tests performed by Constantinou et al. (2007) and HITEC (1999) indicate that the mechanical properties of HDB are sensitive to changes in axial load. In general, as axial force increases, horizontal stiffness decreases and damping increases. However, some HDB showed increased horizontal stiffness at large shear strains that was magnified by large axial load (Aiken et al. 1989).

3.2.3.2.3. Load History

The mechanical properties of an HDB depend on its load history. During the first of several cycles at high shear strain, the bearing has higher effective horizontal stiffness and damping than during subsequent cycles (scragging). By the third cycle, however, the response of the bearing has stabilized (see Figure 3.8). The bearing will likely recover its unscragged properties over time (Constantinou et al. 2007)

3.2.3.2.4. Loading Rate

Tests performed by Constantinou et al. (2007) and HITEC (1999) show that the mechanical properties of HDB are sensitive to changes in loading rate. In general, as the loading rate increases (frequency of excitation increases), horizontal stiffness and damping both increase modestly.

3.2.3.2.5. Aging and Environmental Effects

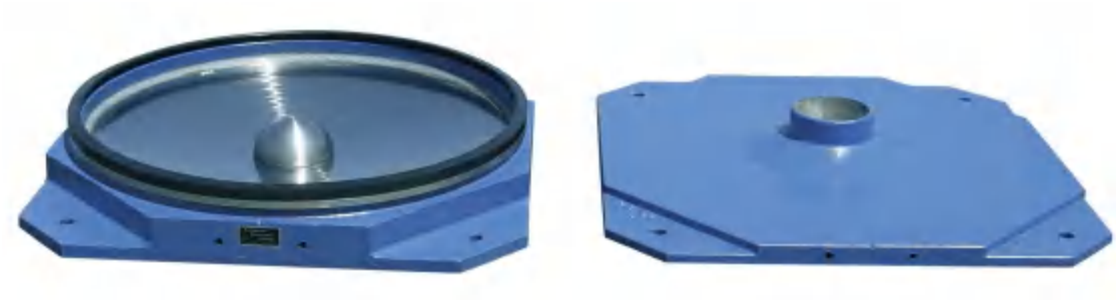
Often the high levels of damping in HDB are achieved through incomplete initial curing. Over time, however, the rubber continues to cure. This ongoing vulcanization of the rubber matrix occurs more rapidly in the first few years after the rubber is compounded, but slows over time as the free sulfur is consumed. For this reason, the effective shear modulus will likely reach a limiting value, which can only be evaluated for a given compound and vulcanization profile by long-term studies (Constantinou et al. 2007).

In addition, the mechanical properties of natural rubber can degrade with prolonged exposure to oxygen and ozone. In bulk rubber (such as the rubber in HDB), only a shallow surface area is affected. To prevent this, elastomeric bearings are normally fabricated with a protective cover layer of rubber containing various anti-ozonants and anti-oxidants (Constantinou et al. 2007).

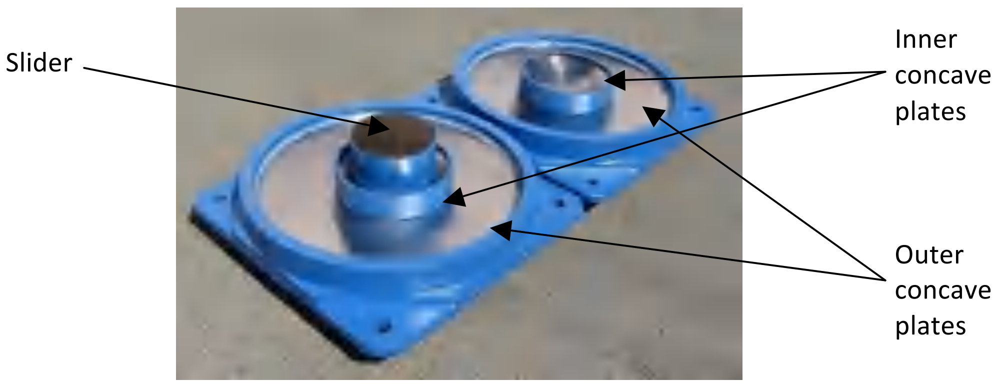
3.2.4. Friction Pendulum™ Bearings

Figure 3.9 presents components of two Friction Pendulum™ bearings (FPB). Figure 3.9(a) shows a single concave FPB. Its components include a housing plate, a concave dish with a spherical inlay of stainless steel, and an articulated slider coated with a low-friction composite material, which typically contains some polytetraflouroethylene (PTFE). The housing plate, shown in the right hand panel of Figure 3.9(a), is inverted and installed on top of the articulated slider, shown in the left hand panel. The slider moves across the spherical surface during earthquake shaking. The housing plate allows the slider to rotate as it moves across the spherical surface. Friction between the slider and the stainless steel inlay dissipates earthquake-induced energy, while the weight supported by the bearing provides a restoring force. The radius of curvature of the sliding surface determines the period of bearing (see equation 3.5).

Figure 3.9(b) shows a triple concave FPB. Its components include an articulated slider and inner and outer concave plates. A triple concave bearing has four separate sliding surfaces. Each sliding surface can be manufactured to have different radii and coefficients of friction. As a result, the isolated period of the bearing depends on its displacement and is determined through a combination of the radii of the sliding surfaces of the inner and outer plates (Fenz and Constantinou 2008a).



(a) Single concave



(b) Triple concave

Figure 3.9 Friction Pendulum™ bearings (courtesy Earthquake Protection Systems, Inc.)

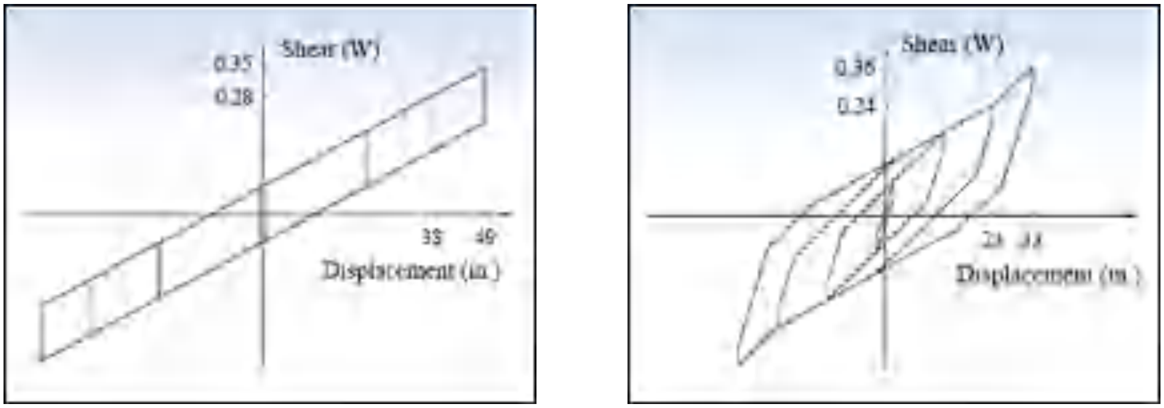
3.2.4.1. Mechanical Properties

The following three sections discuss the *general* behavior of FPB when subject to shear, compression, and tension loads. The mechanical properties of a *specific* FPB depend on many factors, including the type of sliding surface and the geometry of the bearing. Therefore, the exact behavior of the *specific* LDB used in design should be established and verified by testing.

3.2.4.1.1. Shear

Figure 3.10(a) shows the force-deformation relation for a typical single concave FPB. Notice that the response is approximately bilinear and very similar to that of an LRB (see Figure 3.5). The initial high horizontal stiffness of the single concave FPB is due to static friction between the plate and articulated slider. The post-sliding stiffness is a function of the radius of curvature of the concave plate (see Equation 3.4). The area enclosed by the loop depends on the coefficient of friction and contact pressure.

Figure 3.10(b) shows the force-deformation relation for a typical triple concave FPB. The response of a triple concave bearing is similar to that of an HDB. At small shear strain, the horizontal stiffness of the bearing is relatively high, which helps control wind-induced displacements. At moderate shear strain, the horizontal stiffness decreases, which provides the structure with the horizontal flexibility required during the design basis earthquake. At large strain, the horizontal stiffness of the bearing increases, which helps control ultimate horizontal displacement in beyond design basis earthquakes.



(a) Single concave (b) Triple Concave

Figure 3.10 Force-deformation relation for typical FPB (courtesy Earthquake Protection Systems, Inc.)

1.6.1.1

3.2.4.1.2. Compression

Because steel is very stiff, FPB typically have high compressive strength. Local crushing or bearing failure of the sliding surface might occur if the compressive force in the bearing becomes extremely large. This type of failure can be mitigated through proper sizing of the articulated slider. As a result, compressive failure of FPB is typically not a concern.

3.2.4.1.3. Tension

As a result of their design and construction, FPB have no inherent tensile strength. Therefore, FPB should be designed to avoid tension and uplift forces during an earthquake. Triple concave FPB typically feature a flexible rubber sleeve around the perimeter of the inner concave plates that keeps the assembly together if uplift should occur. This sleeve also prevents contamination of the inner sliding surfaces.

3.2.4.2. Variation In Mechanical Properties

The mechanical properties of FPB are affected by many parameters. The following sections describe the *general* effect each parameter has on the mechanical properties of FPB. The *exact* effect of each parameter should be established for the *specific* FPB used in design.

3.2.4.2.1. Temperature

The mechanical properties of FPB are sensitive to changes in ambient temperature. In general, as the ambient temperature increases, the coefficient of sliding friction decreases. Figure 3.10 demonstrates this phenomenon. At small velocities, ambient temperature has substantial effect on the sliding coefficient of friction. But as velocity increases, ambient temperature has less effect. This is due to frictional heating, which increases the temperature of the sliding interface, thus moderating the effect of low ambient temperatures (Constantinou et al. 2007).

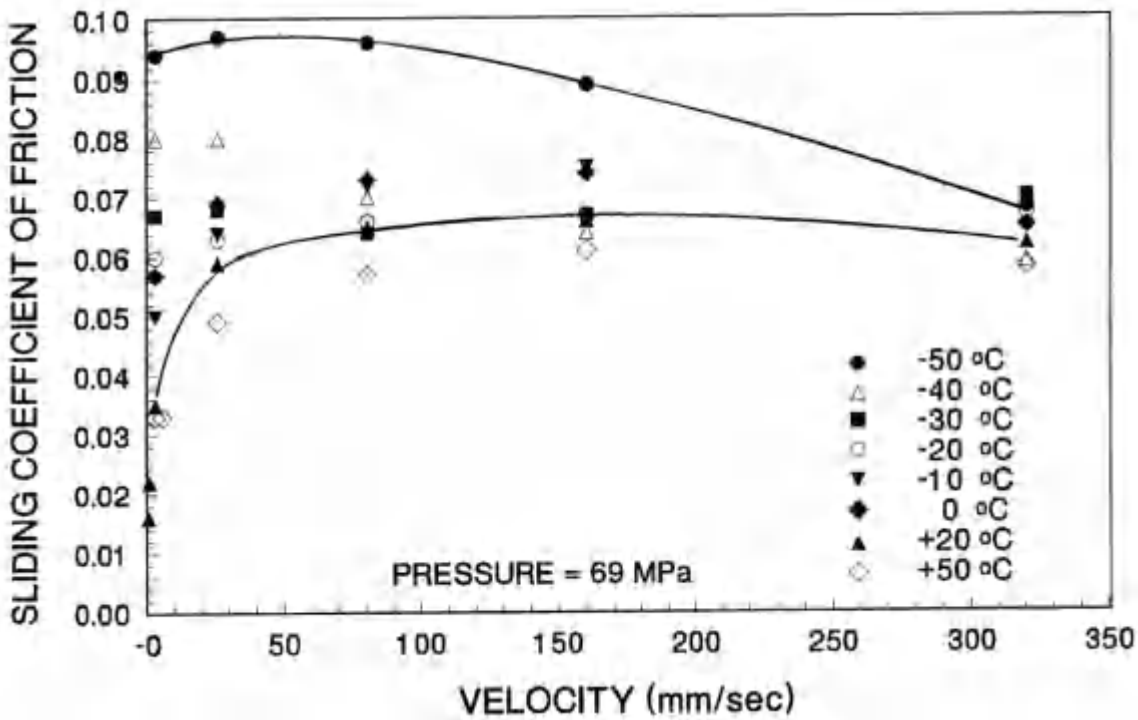


Figure 3.10 Effect of ambient temperature and velocity on the sliding coefficient of friction (Constantinou et al. 2007)

3.2.4.2.2. Axial Load

The mechanical properties of FPB are sensitive to changes in axial load. In general, as the axial load increases, the sliding coefficient of friction decreases (Constantinou et al. 2007). Figure 3.11 shows this relationship. The data in Figure 3.11 is for PTFE surfaces sliding on polished stainless steel surfaces, but the trend is similar for composite surfaces sliding on polished stainless steel surfaces (as is the case for FPB).

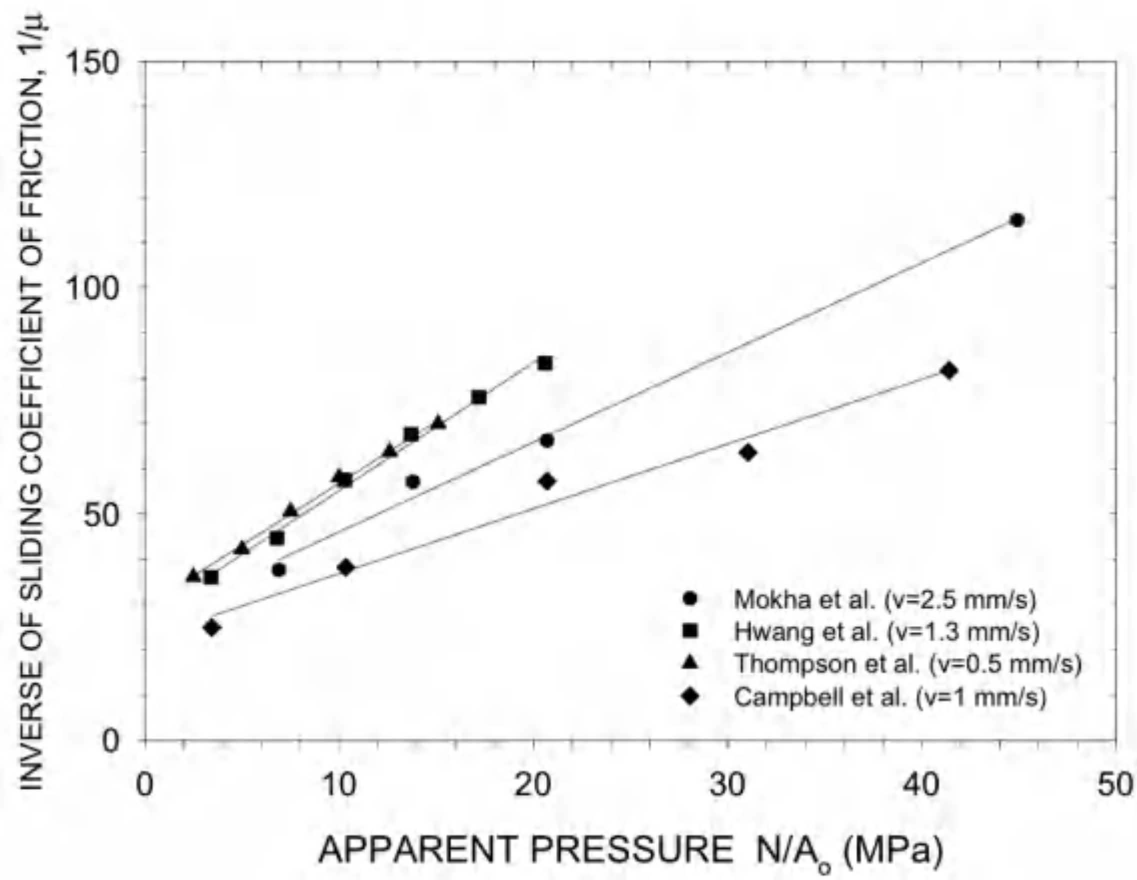


Figure 3.11 Effect of apparent pressure on the sliding coefficient of friction (Constantinou et al. 2007)

3.2.4.2.3. Load History

The mechanical properties of FPB depend on its load history. In the short term (i.e. during an earthquake), as the number of cycles of motion increases, the sliding coefficient of friction decreases due to frictional heating of the sliding interface. In the long term (i.e. over the lifetime of a structure), if an FPB undergoes many repeated cycles of motion, damage to the composite sliding surface may result. Excessive wear of the composite can substantially increase the coefficient of friction and even render the bearing useless.

Data on wear of FPB are limited to a small number of laboratory tests. Constantinou et al. (2007) report that for two different tests wear of FPB was not significant. In the first test, after 10,000 cycles and 1520 meters of travel, the FPB had lost only 2% of its composite sliding surface. In the second test, after 20,000 cycles and 3240 meters of travel, the FPB had lost 20% of its sliding surface.

3.2.4.2.4. Loading Rate

The mechanical properties of FPB are sensitive to changes in loading rate. In general, as the loading rate (velocity) increases, the coefficient of friction increases and approaches a limiting value. Figure 3.10 demonstrates this effect (specifically the curve corresponding to 20°C). At very high velocities (500 mm/sec or larger), there is substantial frictional heating, which may cause local melting of the PTFE-composite surface. When this happens, the sliding coefficient of friction will drop substantially (Constantinou et al. 2007). This decrease is not shown in Figure 3.10.

3.2.4.2.5. Aging and Environmental Effects

The mechanical properties of FPB are sensitive to environmental effects. Even the most corrosion-resistant stainless steel can suffer some atmospheric corrosion in aggressive environments (Constantinou et al. 2007). However, data relating corrosion to the coefficient of friction are still lacking.

In general, contamination of the sliding surface will cause the coefficient of friction to increase. Contamination can stem from many different sources, including cement dust, deicing salts, sand, and other debris. To the extent possible, FPB should be protected from these sources of contamination. Several options exist, including sealing the FPB with a flexible protective cover and installing the stainless steel surface of the FPB facing downward to prevent contamination from within the bearing. Nevertheless, it would still be appropriate to assume a small increase in the coefficient of friction due to in-service contamination (Constantinou et al. 2007).

It is important to note that for most nuclear applications, the isolation system will reside within a controlled environment. Therefore, effects caused by aggressive environments are expected to be minor.

3.3. Recommendations for Nuclear Facility Applications

Research by Huang et al. (2009) suggests that two types of elastomeric bearings and one type of sliding bearings are appropriate for the seismic isolation of nuclear structures: low damping bearings (LDB), lead rubber bearings (LRB), and Friction Pendulum™ bearings (FPB). High damping bearings (HDB) may not be appropriate for nuclear applications because of the potentially significant variation in mechanical properties caused by scragging. This variability makes it difficult to accurately model and predict their response.

Quality control and quality assurance procedures prescribed in ASCE 7 and AASHTO Specifications can be adopted. Nuclear facility structures should be designed to enable in-service inspection of isolator devices. Applicants should, in consultation with isolator device vendors, develop an appropriate inspection plan. This plan should include periodic verification of mechanical properties of isolator devices to assess the effect of ageing on their performance. To this effect, nuclear facility owners should maintain an adequate number of spare isolator units in the environment as essential identical as that for the in-service units. This may include maintaining the appropriate level of compression on the isolator units. The owner of the nuclear facility may be required to replace one or more of the in-service units with spares. Requirements for this operations, such as access to the isolation layer, strong points for jacking, and ability of the superstructure mat to span over one isolator unit, should be engineered.

4. REVIEW OF MODELING AND ANALYSIS TECHNIQUES FOR SEISMICALLY ISOLATED STRUCTURES

The following chapter discusses the current state of practice for the modeling and analysis of seismically isolated structures. In general, the techniques for modeling and analyzing isolated structures closely mirror those for conventional fixed-base structures of comparable configuration and function. The main difference in procedure arises from the additional requirements that account for the isolation system.

For this reason, the following chapter is divided into two main sections. The first section discusses techniques for modeling and analyzing the isolation system. The second section addresses the modeling and analysis procedures for the entire isolated structure.

4.1. Isolation System

This section summarizes the current state of practice for modeling and analyzing seismic isolators. Many of the examined design provisions are consistent with ASCE 4 (*Seismic Analysis of Safety-Related Nuclear Structures and Commentary*) Section 7.7 ("Seismically Isolated Structures").

4.1.1. Mechanical Properties

The isolation system governs the mechanical response of the entire isolated structure. Therefore, it is important to accurately model the mechanical properties of seismic isolators and to capture how they change when parameters that affect them change. For the types of bearings appropriate for use in nuclear facility structures, there is sufficient test data to develop reasonably good models of such bearings.

4.1.1.1. Linear-Elastic Models

As discussed in Chapter 3 of this report, most isolators have nonlinear force-deformation properties. The one exception is the low damping bearing (LDB). For shear strains less than 200%, LDB have approximately linear-elastic force-deformation properties. Therefore, only LDB can be modeled as linear-elastic elements.

However, for conventional (non-nuclear) structures, ASCE 7-10 permits nonlinear isolator units to be modeled as linear elastic elements if they meet a specific set of requirements. These requirements are outlined in Section 17.4.1 of ASCE (2010b). For such isolator units, the effective (linear) horizontal stiffness (K_{eff}) can be computed using the equation below:

$$K_{eff} = \frac{|F_{max}| + |F_{min}|}{|\Delta_{max}| + |\Delta_{min}|} \tag{4.1}$$

Where Δ_{max} is the maximum positive horizontal displacement of the isolator unit during prototype testing, Δ_{min} is the maximum negative horizontal displacement, and F_{max} and F_{min} are the horizontal forces corresponding to Δ_{max} and Δ_{min}, respectively. See Figure 4.1 for a graphical depiction of these quantities. The effective damping ratio (β_{eff}) for the isolator unit can be computed using the following equation:

$$\beta_{eff} = \frac{2}{\pi} \frac{EDC}{K_{eff}\left(\left|\Delta_{max}\right| + \left|\Delta_{min}\right|\right)^2} \tag{4.2}$$

Where EDC is the energy dissipated per cycle of loading.

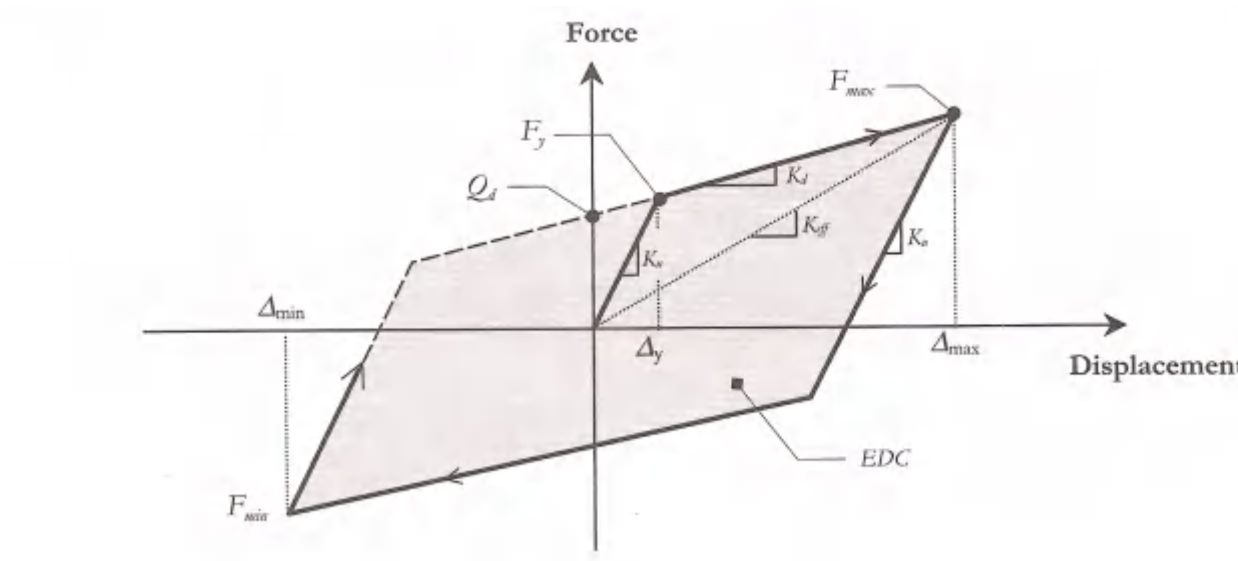


Figure 4.1 Linearization procedure for a typical nonlinear isolator unit (HITEC 1999)

For nuclear structures, linear-elastic models of the isolator units should be avoided except in the following situations. Linear-elastic models of the isolators are appropriate during the preliminary stages of the design process. If the isolation system comprises only low damping bearings, linear-elastic models of the isolator units can be used throughout all stages of the design process.

4.1.1.2. Nonlinear Models

Because most isolator units have highly nonlinear force-deformation properties, nonlinear models are required to effectively capture their behavior. These nonlinearities are important because they can cause significant response in higher modes of the superstructure. Typically, a bilinear model can adequately capture the behavior of a nonlinear isolator. More detailed models (tri-linear, for example) can be laborious and difficult to execute, but may be necessary depending on the selected isolator.

4.1.2. Variation of Mechanical Properties

As discussed in Chapter 3, the mechanical properties of seismic isolators are sensitive to a number of different parameters. Analytical models of the isolators should account for this variability. This can be achieved by assuming average properties or by conducting a bounding analysis.

The mechanical properties of isolators should include variation due to different rates or levels of applied loading, variation between different isolator units (as determined by prototype tests), and variation due to aging or other environmental effects (see Chapter 3 for additional information). Significant variation of isolator properties may need to be addressed through more than one model of the isolation system (bounding analysis).

4.1.3. Transmission of Vertical Motion

In regions of high seismicity, vertical earthquake motion can be significant. The vertical stiffness of the isolation system can amplify rocking and overturning of the structure during an earthquake. It can also affect the floor spectra of the superstructure. As a result, the model of the isolation system should include vertical force-deformation properties.

4.1.4. Coupling

In the design of three-dimensional seismically isolated structures, analytical models of the isolators typically neglect coupling effects in the horizontal plane (Abe et al. 2004). Unidirectional models of the isolators are used for each horizontal direction. The total displacement of the system is then taken as the vector sum of these two uncoupled components.

However, research has shown that coupling effects in the horizontal plane can have significant impact on the response of isolators. Research by Mokha et al. (1993) demonstrates that models of flat Teflon sliding bearings without horizontal coupling under-predict displacements and over-predict shear forces in the isolation system (relative to models with horizontal coupling). Furthermore, coupling effects between the horizontal plane and vertical axis exist and can have significant impact. Ryan et al. (2005) report that elastomeric bearings have been shown to soften in the vertical direction at large lateral deformation. For these reasons, fully coupled three-dimensional models of the isolators are ideal, but may be difficult and laborious to implement.

4.1.5. Verification of Mechanical Properties

The mechanical properties of each type of isolator used in the isolation system should be verified through laboratory testing. Ideally, full-scale specimens should be tested. If not practical, reduced-scale specimens are acceptable provided that scaling and size effects are considered. If data from previous verification tests are available for a specific isolator, this data may be used in design and laboratory testing is not required.

4.2. Isolated Structure

The following subsections summarize the current state of practice for the modeling and analysis of seismically isolated structures. Many of the examined design provisions are consistent with ASCE 4-10 (*Seismic Analysis of Safety-Related Nuclear Structures and Commentary*) Section 7.7 ("Seismically Isolated Structures").

4.2.1. General Modeling Requirements

A mathematical model of the entire isolated structure is required to demonstrate that the system has acceptable performance in the design basis event, as well as the beyond design basis event. In general, this model should involve enough complexity to accurately predict the response of the isolated structure in the design basis event (and beyond design basis event).

In particular, the model should include a three-dimensional characterization of the isolation system and should account for the spatial distribution of isolators. The model should be sufficiently refined to:

1. Calculate translation in both horizontal directions;
2. Capture potential effects of torsion in the horizontal plane;
3. Calculate translation in the vertical direction and rocking in the superstructure;
4. Assess overturning/uplift forces on the superstructure and isolation system (including individual isolators).

The mechanical properties of other materials used in the model (steel, concrete, soil, etc.) should be obtained from tests or the appropriate codes and standards.

4.2.2. Analysis Procedures

The following subsections describe the two main types of analysis procedures.

4.2.2.1. Response Spectrum Analysis

Response spectrum analysis is a simplified analysis procedure in which structural properties (often generated by the mathematical model of the structure) are used to determine key response parameters during the design basis event. In particular, the fundamental period of a structure is used to obtain the maximum spectral acceleration (or displacement). These spectral quantities can then be used to determine the maximum forces, accelerations, and displacements throughout the structure. The ground motion spectra used should reflect the seismic hazard from both long-period ground motions and near-field pulse motions.

The response-spectrum analysis procedure is not appropriate for nonlinear isolation systems and a family of spectra (see Huang et al. 2009) will be required for analysis to establish that performance goals are being met.

4.2.2.2. Response History Analysis

A response history analysis involves subjecting a mathematical model of a soil-structure system to an earthquake record. Typically, the model is subjected to multiple earthquake records. These records should contain motion in three orthogonal directions and should accurately reflect the type of shaking expected at a specific site.

A response history analysis can be performed using a mathematical model that combines nonlinear elements of the isolators with a linear-elastic model of the superstructure and equivalent linear model of the soil. A fully coupled, three-dimensional response history analysis is possible but numerically intensive. Simplified numerical models of the isolated superstructure will often suffice for the computation of displacement and force demands on isolator units, including the effects of rocking (if any).

4.2.3. Soil Structure Interaction

The model of the isolated structure should include the effects of interaction between the soil and isolated structure. Since linear models of isolators are inappropriate for LRB, HDB and FPB, frequency-domain approaches for considering soil-foundation-structure interaction should not be used.

4.2.4. Variation of Mechanical Properties

The analysis of the isolated structure should address variations in the mechanical properties of the different materials used in isolated structure. Huang et al. (2009) presents a procedure for such computations, considering the effect of variations in the mechanical properties of the isolators. The results of the Huang et al. (2009) study have been included in the analysis and design provisions of ASCE 4.

4.2.5. Floor Spectra

The model of the isolated structure can be used to generate acceleration and displacement spectra for each floor. These spectra are used to determine the forces and accelerations on equipment and machinery at various heights and locations throughout the superstructure.

The effect of nonlinear isolation systems on higher mode horizontal and vertical response is important to the development of floor spectra. Therefore, computation of the initial (elastic) stiffness of the isolator units must be made with care (Constantinou et al. 2007, Fenz and Constantinou 2008a, Huang et al. 2009).

5. REVIEW OF DESIGN PROCEDURES FOR SEISMICALLY ISOLATED STRUCTURES

The following chapter discusses the current state of practice for the design of seismically isolated structures. It is divided into four sections. The first section discusses the two main types of design procedures. The second section reviews current design standards for seismic isolation of conventional structures. The third section reviews design standards for safety-related nuclear structures.

5.1. Design Procedures

Two types of procedures can be used when designing a building: deterministic or probabilistic. Most current structural codes and standards are deterministic, though recently there has been increased interest in developing probabilistic design procedures. The next two subsections discuss each type of design procedure.

5.1.1. Deterministic

Deterministic design procedures implicitly assume both a probability of hazard and a probability of failure for the component or structure, but do not explicitly specify them. The randomness in the design variables associated with both demand and capacity is accounted for using deterministic factors, such as safety factors or load and resistance factors. Typically, this translates into checking the demand-capacity ratios of components in a structure for factored loads and reduced capacities and checking that demand-capacity ratios are less than 1.0.

5.1.2. Probabilistic

Probabilistic design procedures explicitly specify both the probability of hazard and the probability of failure, and explicitly account for randomness in design variables on both the demand and capacity side.

Instead of demand-capacity ratios, probabilistic procedures focus on risk reduction factors. The risk reduction factor, R_R, is defined as:

$$R_R = \frac{P_H}{P_F} \tag{5.1}$$

where P_H is the annualized probability of exceeding a designated hazard level and P_F is the annualized probability of failure for the structure or element. Typically, $R_R = 1$ for conventional structures, while $R_R >$ 1 for essential or critical structures (including nuclear facilities). Risk reduction factors for critical components are typically larger than those for non-critical ones. A satisfactory design would be one whose structural components have risk reduction factors greater than or equal to their targets.

5.2. Review of Design Standards for Seismically Isolated Conventional Structures

The following subsections summarize the design standards and codes that contain provisions for seismically isolated structures. Note that all of them contain deterministic procedures.

5.2.1. ASCE 7-10

Chapter 17 of ASCE 7-10 (*Minimum Design Loads for Buildings and Other Structures*) contains provisions addressing the use of seismic isolation in conventional buildings. It provides design displacements and shear forces for the isolation system, as well as other specific requirements for the design of seismically isolated structures. All other design requirements, including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal shear distribution, are the same as those for conventional, fixed-base structures.

Rather than addressing a specific method of seismic isolation, Chapter 17 of ASCE 7-10 provides general design requirements applicable to a wide range of possible isolation systems. Because the design requirements are general, testing of isolator units is required to confirm the mechanical properties used in design and to verify the overall adequacy of the isolation system. Use of isolation systems whose adequacy is not proved by testing is prohibited. In general, acceptable systems (1) remain stable when subjected to design displacements, (2) provide increasing resistance with increasing displacement, (3) do not degrade under repeated cyclic load, and (4) have quantifiable engineering parameters (such as force-deflection characteristics and damping).

The following subsections summarize these provisions. For more detailed information, please refer to Chapter 17 of ASCE (2010a).

5.2.1.1. Definitions

Chapter 17 of ASCE 7-10 defines four different isolator displacements, each of which is used for different parts of the design process.

> *Design displacement*: The design earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.
>
> *Total design displacement*: The design earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for design of the isolation system or an element thereof.
>
> *Maximum displacement*: The maximum considered earthquake (MCE) lateral displacement, excluding additional displacement due to actual and accidental torsion.
>
> *Total maximum displacement*: The maximum considered earthquake (MCE) lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system or elements thereof, design of structure separations, and vertical load testing of isolator units.

Chapter 17 of ASCE 7-10 also defines the various parts of an isolated structure (which are consistent with the definitions in Section 1.2 of this report):

> *Isolation interface*: The boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which moves rigidly with the ground.

Isolation System: The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system, energy-dissipation devices, and/or the displacement restraint system if such systems and devices are used to meet the design requirements of this chapter.

Isolator Unit: A horizontally flexible and vertically stiff structural element of the isolation system that permits large lateral deformations under design seismic load. An isolator unit is permitted to be used either as part or, or in addition to, the weight-supporting system of the structure.

5.2.1.2. General Design Requirements

Chapter 17 of ASCE 7-10 specifies general design requirements for seismically isolated structures. Isolated structures designed in accordance with the provisions of ASCE 7 are expected:

1. To resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components, or building contents; and
2. To resist major levels of earthquake ground motion without failure of the isolation system, significant damage to structural elements, extensive damage to nonstructural components, or major disruption to facility function.

As a result, isolated structures are expected to perform much better than fixed-base structures during moderate and major earthquakes. The following paragraphs summarize several of the general design requirements contained in Section 17.2 of ASCE 7-10. For a complete list of these provisions, please refer to Section 17.2 of ASCE (2010a).

ASCE 7-10 requires that the isolation system:

- Not include a displacement restraint that limits lateral displacement due to the maximum considered earthquake to less than the total maximum displacement (unless specific requirements are met);
- Provide a wind-restraint system to limit lateral displacement;
- Produce a restoring force; and
- Provide for other environmental conditions including aging effects, creep, fatigue, operating temperature, and exposure to moisture or damaging substances.

Furthermore, ASCE 7-10 requires that individual elements of the isolation system:

- Be stable under the design vertical load when subjected to a horizontal displacement equal to the total maximum displacement; and

- Be designed to avoid local uplift unless the resulting deflections do not cause overstress or instability of the isolator units or other structure elements.

Chapter 17 of ASCE 7-10 also requires that elements and components of the seismically isolated structure that cross the isolation interface be designed to withstand the total maximum displacement (induced by the MCE). Minimum separations between the isolated structure and surrounding retaining walls or other fixed obstructions should be greater than the total maximum displacement.

5.2.1.3. Hazard Characterization

ASCE 7-10 specifies the MCE as an event having 2% probability of exceedance in 50 years, which results in a return period of approximately 2,500 years. The design earthquake is taken as two thirds of the MCE.

Chapter 17 of ASCE 7-10 permits the use of either design spectra or ground motion histories to characterize the earthquake hazard for a seismically isolated structure at a particular site. Where design spectrum procedures are appropriate, isolated structures that do not require site-specific ground motion procedures can be analyzed using the same design spectrum as for conventional structures. Where response history procedures are used, ground motions should consist of pairs of appropriate horizontal ground motion acceleration components that shall be selected and scaled from individual recorded events. Appropriate ground motions shall be selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the MCE. Where the required number of recorded ground motion pairs are not available, appropriate simulated ground motion pairs shall be used to make up the total number required.

5.2.1.4. Analysis Procedure Selection

Chapter 17 of ASCE 7-10 specifies two different types of analysis procedures: equivalent lateral force and dynamic. Equivalent lateral force procedure is permitted only if specific criteria are satisfied. Please refer to Section 17.4.1 of ASCE (2010a) for a list of these criteria. Dynamic analysis procedures are required if the criteria specified in Section 17.4.1 of ASCE 7-10 are not satisfied. There are two types of dynamic analysis procedures: response spectrum and response history. Response spectrum analysis procedures are permitted only if specific criteria are satisfied. Please refer to Section 17.4.2.1 of ASCE (2010b) for a list of these criteria.

The next two sections discuss the requirements for equivalent lateral force and dynamic analysis procedures, respectively.

5.2.1.5. Equivalent Lateral Force Procedure

The equivalent lateral force procedure outlined in Section 17.5 of ASCE 7-10 is a prescriptive set of design requirements for seismically isolated structures satisfying the criteria specified in Section 17.4.1 of ASCE 7-10. It provides equations for calculating minimum lateral displacements (Section 17.5.3) and minimum lateral forces (Section 17.5.4) to be used in the design of seismically isolated structures. Minimum lateral displacements are computed using the minimum effective stiffness of the isolation system as established by testing in accordance with the requirements of Section 17.8 of ASCE 7-10. Minimum lateral forces, on the other hand, are computed using the maximum effective stiffness of the isolation system. The base shear for the structure above the isolation interface can be reduced by the R factor, which for an isolated structure ranges between 1 and 2. The base shear for the isolation system and the structure below the isolation interface, however, cannot be reduced by the R factor. The maximum story drift of the structure above the isolation interface must be less than 1.5 percent of the story height.

5.2.1.6. Dynamic Analysis Procedures

Section 17.6 of ASCE 7-10 outlines the requirements of dynamic analysis. In general, the requirements are less prescriptive than those of an equivalent lateral force analysis. Section 17.6 requires that the isolation system be modeled using deformational characteristics developed and verified by testing in accordance with the requirements of Section 17.8 of ASCE 7-10. The isolation system should be modeled with sufficient detail to:

a. Account for the spatial distribution of isolator units;
b. Calculate translation, in both horizontal directions, and torsion of the structure above the isolation interface considering the most disadvantageous location of eccentric mass;
c. Assess overturning/uplift forces on individual isolator units; and
d. Account for the effects of vertical load, bilateral load, and/or rate of loading if the force-deflection properties of the isolation system are dependent on one or more of these attributes.

A linear elastic model of the isolated structure is permitted provided that:

1. Stiffness properties assumed for the nonlinear components of the isolation system are based on the maximum effective stiffness of the isolation system; and
2. All elements of the seismic force-resisting system of the structure above the isolation system remain elastic for the design earthquake.

Section 17.6 of ASCE 7-10 specifies that the design earthquake be used to calculate the total design displacement of the isolation system and the lateral forces and displacements in the isolated structure. The MCE shall be used to calculate the total maximum displacement of the isolation system.

If a response spectrum analysis is permitted, the total design displacement and total maximum displacement shall include simultaneous excitation of the model by 100 percent of the ground motion in the critical direction and 30 percent of the ground motion in the perpendicular, horizontal direction. The maximum displacement of the isolation system shall be calculated as the vector sum of the two orthogonal displacements. The maximum story drift of the structure above the isolation system calculated by response spectrum analysis shall not exceed 1.5 percent of the story height.

Where a response history analysis is performed, Section 17.6 of ASCE 7-10 requires a suite of no less than three appropriate ground motions be used in the analysis. Each pair of ground motion components shall be applied simultaneously to the model considering the most disadvantageous location of eccentric mass. The maximum displacement of the isolation system shall be calculated from the vector sum of the two orthogonal displacements at each time step. If at least seven ground motions are used, ASCE 7-10 permits the average value of the response parameter of interest to be used for design. If fewer than seven ground motions are analyzed, the maximum value of the response parameter of interest must be used. The maximum story drift of the structure above the isolation system calculated by response history analysis shall not exceed 2.0 percent of the story height.

Lateral displacements and forces obtained from a dynamic analysis (either response spectrum or response history) cannot be less than the minimum values specified in Section 17.6.4 of ASCE 7-10. These minimum values are computed using equations similar to the ones from the equivalent lateral force procedure of Section 17.5.

5.2.1.7. Design Review

Section 17.7 of ASCE 7-10 requires that an independent engineering team review the design of the isolation system. This team should include persons experienced in seismic analysis methods and the theory and application of seismic isolation.

5.2.1.8. Testing

Section 17.8 of ASCE 7-10 requires that the deformation characteristics and damping values of the isolation system used in the design and analysis of seismically isolated structures be based on tests of a selected sample of the components. Prototype tests must be performed separately on two full-size specimens (or sets of specimens, as appropriate) of each predominant type and size of isolator unit in the isolation system. Section 17.8.2.2 of ASCE 7-10 prescribes the sequence and number of cycles for each test. In addition, Isolator units should be tested to determine if their force-deflection characteristics vary with vertical load, rate of loading, or bilateral load.

Using data recorded during the prototype tests, the deformation characteristics and damping values of the isolation system can be determined. Section 17.8.5 of ASCE 7-10 contains equations for computing the minimum and maximum effective stiffness and the effective damping of the isolation system.

5.2.2. NEHRP Recommended Provisions

The seismic isolation provisions of the NEHRP Recommended Provisions form the basis of the seismic isolation provisions in ASCE 7. Please refer to Section 5.2.1 of this report for a summary of these provisions.

5.2.3. AASHTO Guide Specification for Seismic Isolation Design

The *Guide Specifications for Seismic Isolation Design*, published by American Association of State Highway and Transportation Officials (AASHTO 2010), contains provisions for the seismic isolation of bridge structures. The isolation system in a bridge is typically installed at the top of the bridge piers (just below the bridge deck). During an earthquake, the isolators limit the amount of force that can be transferred from the deck to the piers and foundation. This force reduction helps the piers and foundation remain essentially elastic during the design basis earthquake. In contrast, a bridge without isolation is expected to undergo large inelastic deformation and suffer permanent damage.

Many of the provisions in the AASHTO *Specifications* are similar to those contained in Chapter 17 of ASCE 7-10. The AASHTO *Specifications*, however, provide a systematic approach for determining bounding values of the mechanical properties of isolators for design and analysis. For this purpose, AASHTO defines system property modification factors that account for the effects of temperature, aging, scragging, velocity, cumulative travel, and contamination. These factors are based on Constantinou et al. (1999, 2007) and Thompson et al. (2000).

AASHTO *Specifications* impose an additional prototype bearing test requirement aimed at verifying the ability of a bearing to sustain thermal and traffic induced small-amplitude motions. The additional test consists of uniform amplitude cycling of a bearing to a cumulative travel of 1 mile. While such tests are appropriate for bridge applications, they may not be appropriate for nuclear power plants as the bearings, located under the power plant are not expected to undergo significant temperature changes or motion due to moving loads in the isolated superstructure.

5.3. Review of Design Standards for Safety-Related Nuclear Structures

The following subsections summarize the design standards and codes that contain provisions for safety-related nuclear structures. The first subsection addresses design standards containing deterministic procedures, while the second subsection addresses design standards containing probabilistic procedures.

5.3.1. Deterministic Procedures

5.3.1.1. ASCE 4-10

ASCE 4 (*Seismic Analysis of Safety-Related Nuclear Structures and Commentary*) identifies four steps in the design and construction process that lead to the reliable performance of safety-related nuclear structures in an earthquake:

1. Definition of the seismic environment;
2. Analysis to obtain response information;
3. Design or evaluation of the various structural elements; and
4. Construction.

ASCE 4 focuses on Step 2. While this version of the standard is deterministic, the prescribed procedures are consistent with the overall probabilistic goals for design of nuclear structures. Section 7.7 ("Seismically Isolated Structures") of ASCE 4-10 identifies specific requirements and techniques for dynamic response evaluation, design, and testing of seismically isolated nuclear structures. The following subsections summarize many of these provisions. Please refer to ASCE (2010b) for additional information.

The provisions in Section 7.7 of ASCE 4-10 are written for the three types of isolators considered appropriate for nuclear structures: low damping bearings, lead rubber bearings, and sliding bearings with restoring force provided by gravity (including Friction Pendulum™ bearings). The two performance objectives specified in Section 1.3 of ASCE 43-05 (summarized below) form the basis of the provisions contained in Section 7.7 of ASCE 4-10:

1. 1% probability of unacceptable performance for 100% design basis earthquake (DBE) shaking; and
2. 10% probability of unacceptable performance for 150% DBE shaking.

To achieve these two performance objectives, ASCE 4 specifies three performance statements:

1. Individual isolators shall suffer no damage in DBE shaking;
2. The probability of the isolated nuclear structure impacting the surrounding structure (or moat) for 100% (150%) DBE shaking is 1% (10%) or less; and
3. Individual isolators shall sustain gravity and earthquake-induced axial loads at 90^{th} percentile lateral displacements consistent with 150% DBE shaking.

Performance statement 1 can be realized by production testing of each isolator supplied to a project for median DBE displacements and co-existing gravity and earthquake-induced axial forces. Analysis can be used in support of performance statement 2 provided that the isolators are modeled correctly and the ground motion representations are reasonable. Performance statement 3 can be realized by prototype testing of a limited number of isolators for displacements and co-existing axial forces consistent with 150% DBE shaking, noting that an isolation system may contain on the order of a hundred isolator units and failure of the system would involve the simultaneous failure of a significant percentage of the isolators.

5.3.1.1.1. General Design Requirements

ASCE 4 requires that the design of the isolation system account for the effects of vertical and lateral loads induced by earthquakes and wind, as well as other effects including aging, creep, and operating temperature and moisture. Best-estimate mechanical properties for the isolator units are used for DBE analysis. The mechanical properties of the isolation system should not vary over the lifespan of the structure by more than ±20% from the best-estimate values (with 95% probability).

Furthermore, ASCE 4 requires that the design provide access for inspection and replacement of all components of the isolation system. A registered design professional should develop programs for monitoring the isolation system and inspecting individual isolators. The engineer of record may choose to store spare isolators for periodic retesting. Such isolators should be stored next to installed isolators and placed under similar compressive loads. These spare isolators should be retested with the same test machine and protocol used for the initial tests.

5.3.1.1.2. Seismic Input

The procedure for determining the seismic input for an isolated nuclear structure is the same as for a conventional fixed-base nuclear structure. Please refer to Section 2 of ASCE 4-10 for additional information.

5.3.1.1.3. Dynamic Analysis

ASCE 4 allows only response history analysis procedures for isolated nuclear structures. No fewer than 11 sets of three-component ground motions should be used for analysis. Each set should be selected and scaled in accordance with Section 2 of ASCE 4-10. Each set of ground motion components should be applied simultaneously to the model. The maximum horizontal displacement of the isolation system can be calculated from the vector sum of the two orthogonal components at each time step. The peak value of each response parameter of interest for design should be captured for each set of ground motions. The 80^{th} percentile value of each response parameter from the 11+ sets of analyses shall be used for design.

ASCE 4 requires that the isolation system be modeled using best-estimate non-linear force-displacement relationships for the isolator units developed by analysis and verified by dynamic testing. Linear force-displacement relationships are permitted only for low damping bearings. The isolation system should be modeled with sufficient detail to:

a. Account for the spatial distribution of isolator units;
b. Calculate translational and torsional displacements of the structure above the isolation interface; and
c. Assess axial forces on individual isolator units.

ASCE 4 permits the superstructure to be modeled using best-estimate elastic properties.

5.3.1.1.4. Displacement and Forces For Design

ASCE 4 requires that the superstructure, foundation, and isolator units be designed for the forces and displacements computed by response history analysis. The connections of the isolators to the superstructure and foundation should be designed for the un-factored loads and lateral forces associated with best-estimate material properties of the isolators, and for the displacements associated with 10% probability of exceedance in 150% DBE shaking, using best-estimate material properties and strength reduction factors equal to unity.

ASCE 4 does not explicitly provide guidance for estimating the design displacement capacity of isolator units. Two approaches are proposed in a supporting report (Huang, et.al, 2009). The first approach is performed at a design-basis earthquake level. It involves computing the maximum isolator displacement demand form 11 appropriately scaled ground motion records, computing its statistical characteristics and determining the appropriate scale factor to satisfy the ASCE 4 probabilistic performance objectives. Authors of the supporting report (Huang, et.al, 2009) suggest that is lieu of further computation, the median of the maximum displacement demand at DBE level can be multiplied by 3 to compute the design displacement demand for isolator units. The second approach is performed at the beyond design-basis earthquake level. It involves selection of 30 ground motions, their appropriate scaling, and selection of 30 instantiations of isolator devices by varying their properties within the expected property value ranges. Isolator unit design displacement demand is then computed from the statistical distribution of the maximum horizontal displacement demand computed form at least 30 analyses selected from the total 900 possible ground motion-isolator unit combinations to satisfy the probabilistic performance objectives of ASCE 4.

In addition, ASCE 4 requires that any seismic joints and/or structures, systems, or components that cross the isolation interface be designed and detailed to accommodate a horizontal displacement with no more than 10% probability of exceedance in 150% DBE shaking, accounting for variability in both spectral demand and isolator mechanical properties. ASCE 4 permits this horizontal displacement to be taken simply as three times the design displacement.

5.3.1.1.5. Design Review

ASCE 4 requires a design review of the isolation system and related test programs by a review team consisting of one or more members experienced in the application of seismic isolation and large-scale testing of isolator units. The review should include, but is not limited to, the following:

a. Review of displacement and force calculations for the isolator units and all associated structures, systems, and components;
b. Review of the prototype test program; and
c. Review of the production (quality control) test program.

5.3.1.1.6. Testing of Prototype and Production Isolators

ASCE 4 specifies and requires two different series of tests: prototype tests and production (quality control) tests.

Prototype tests need to be performed separately on two full size specimens of each predominant type and size of isolator unit in the isolation system. Prototype test specimens may not be used in construction. ASCE 4 specifies a minimum sequence of tests for prototype isolators. Additional testing may be required on a case-by-case basis. Prototype tests are not required if an isolator unit is of similar size and of the same type and material as a prototype isolator unit that has been tested previously using the specified sequence of tests.

Production (quality control) tests need to be performed on each isolator unit in the isolation system. ASCE 4 specifies a minimum sequence of tests for production isolators.

5.3.1.2. NRC RG 1.165

NRC Regulatory Guide 1.165 (*Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion*) outlines a deterministic procedure for selecting ground motion for the DBE, also referred to as the safe shutdown earthquake. NUREG RG 1.165 has been superseded by RG 1.208.

5.3.1.3. ACI 349

The American Concrete Institute (ACI) publishes ACI 349, *Code Requirements for Nuclear Safety-Related Concrete Structures*. This standard should be referenced if the superstructure or foundation of the isolated structure is constructed with concrete. In particular, this standard may guide the design of the anchorage of the isolator unit plates into the foundation and the isolated superstructure mats.

5.3.1.4. ANSI/AISC N690

The American National Standards Institute (ANSI) and the American Institute of Steel Construction (AISC) publish ANSI/AISC N690, *Specifications for the Design, Fabrication, and Erection of Steel Safety Related Structures for Nuclear Facilities*. This standard should be referenced if the superstructure or foundation of the isolated structure is constructed with steel. In particular, this standard may guide the design of the isolator unit anchor plates (usually made of steel) and the connection of these plates to the foundation and superstructure mats.

5.3.2. Probabilistic Procedures

5.3.2.1. ASCE 43-05

ASCE 43 (*Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities*) is a consensus standard that contains more stringent seismic design criteria than building codes for conventional structures. It provides a rational basis for the performance-based, risk-consistent seismic design of safety-related nuclear structures. ASCE 43 is intended for use with ASCE 4, ACI 349, ANSI/AISC N690, ASCE 7, and other standards and codes.

ASCE 43 follows the graded approach outlined in ANSI/ANS Standard 2.26 (*American National Standard for Design Categorization of Nuclear Facility Structures, Systems, and Components for Natural Phenomena Hazards*) to define five “Seismic Design Categories” (SDCs) and four “Limit States.” SDCs increase from 1 to 5. SDC 1 corresponds to conventional structures while SDC 5 corresponds to hazardous or critical structures (such as nuclear reactor structures). Limit States range from A to D. Structures assigned to Limit State A are expected to experience large permanent distortion during the DBE, while structures assigned to Limit State D are expected to remain essentially elastic. The provisions in ASCE 43 cover all Limit States but apply only to SDCs 3, 4, and 5.

The SDC is used to set the DBE: the higher the SDC, the larger the design basis earthquake. The Limit State is used to set the analysis methodology, design procedures, and acceptance criteria for the structure. Regardless of the SDC or Limit State, the structure is designed to have 1% probability of unacceptable behavior in the DBE, and 10% probability of unacceptable behavior when the DBE is amplified by 150% (which is commonly referred to as the beyond design basis event).

ASCE 43-05 contains no specific provisions for seismically isolated structures.

5.3.2.2. NRC RG 1.208

This section will contain a brief summary of NRC Regulatory Guide 1.208, *A Performance-Based Approach to Define the Site-Specific Earthquake Ground Motion*.

5.3.2.3. ANSI/ANS 2.26, 2.27 and 2.29

This section will contain a brief summary of ANSI/ANS 2.26, 2.27 and 2.29 as they pertain to seismic isolation.

6. CONCLUSIONS AND RECOMMENDATIONS

The renaissance of nuclear energy is the catalyst behind the surge in exploring novel civil/structural technologies. Seismic isolation is one such civil/structural technology: it enables engineered modification of the seismic response of a structure and its systems and components. Reduction of seismic risk and reduction of cost and time to build are the two principal incentives for license applicants to seriously consider seismic isolation of new nuclear power plant structures. The objective of this report is to summarize the technical considerations that will serve as the basis for design regulation and regulatory review of seismically isolated nuclear power plant structures.

This report presents the technical consideration for analysis and design nuclear power plants and safety-related nuclear facility structures using seismic isolation. It is intended to serve as a reference to those engineers planning to use seismic isolation in their designs, as well as regulators reviewing applications that employ seismic isolation hardware and systems.

The technical considerations presented in this document apply to the seismic isolation of nuclear power plants and safety-related nuclear facility structures and not their subsystems or individual components (such as equipment). It is assumed that any hardware used in the seismic isolation system will be installed in a low- or no-radiation environment. Therefore, the effect of radiation on the response of seismic isolation hardware is not discussed in this report.

To the extent possible, the technical considerations provided herein will be compatible with other resource documents on the seismic isolation of infrastructure, such as ASCE 7 *Minimum design loads for buildings and other structures* (ASCE 2010a), ASCE 4 *Seismic analysis of safety-related nuclear structures* (ASCE 2010b), and the AASHTO *Guide Specification for seismic isolation design* (AASHTO 2010).

Seismic hazard remains of the three major contributors to the total risk of nuclear reactor core damage and large radiation dose release. Seismic isolation is a technology that can significantly reduce the contribution of seismic hazard to the overall risk exposure of nuclear power plant structures. It can be used to, with high confidence, limit the inertial forces and the vibrations transmitted from the ground to the isolated nuclear facility superstructure. Furthermore, seismic isolation are engineered devices: the engineering properties of isolator units produces in the same series vary less than the mechanical properties of conventional steel and concrete structural elements, and significantly less than the variations in ground motion demand. Seismic isolation devices can be engineered to control the seismic excitation of the isolated superstructure at essentially the same low level for a wide variety of potential nuclear power plant sites. This enables modular design, fleet rather than individual plant licensing, and modular construction of nuclear power plants. While seismic isolation adds a structural system between the substructure and superstructure, the added cost and complexity is very likely to be compensated for by the savings realized through the increase of safety margins despite the smaller strength, size and complexity of the superstructure. However, a significantly larger economy can be derived from simplified, standardized designs made to suite modular construction methodology. Today, modular construction is being implemented at the level of components and systems: use of seismic isolation enables expansion of the modular concept to the entire isolated superstructure and provides the all-important cost driver for use of seismic isolation in new nuclear power plants.

Use of seismic isolation systems brings about a number of challenges. Seismic hazard analysis for seismically isolated structures must consider the long fundamental vibration period of such structures, typically 2 seconds or longer. Thus, ground motion records must be filtered differently to provide reliable data in this long-period (low-frequency) range, ground motion attenuation relations must be developed accordingly, and design spectra and ground motion selection procedures must be modified. Maximum horizontal deformation of the isolator units occurs under combined effects of the horizontal components of ground motion and the relative eccentricity of the isolated superstructure with respect to the isolation system. Effect of the vertical motion of the isolated superstructure on the isolator units must be accounted for, regardless of the cause: overturning moment in the superstructure, ground motion in the vertical direction, or the rotational (rolling) components of the ground motions. Therefore, ground motion selection and scaling must account for all components of ground motion. Modeling of the response of seismically isolated structures must directly account for the three-dimensional non-linear behavior of seismic isolators under both design-basis and beyond-design-basis earthquake ground motions. This is particularly challenging for the traditional frequency-domain methods used to evaluate the effects of soil-structure interaction on the structural demands and floor spectra. To overcome this challenge, it is imperative to develop effective time-domain non-linear modeling and analysis methods to account for soil-structure interaction between the foundation, the seismic isolation system, and the isolated superstructure, and to develop validated and verified models of the isolator units.

Non-redundancy of the seismic isolation system is a profound challenge. The seismic isolation system comprises a large number of essentially identical seismic isolation units that act in parallel. The performance of the seismic isolation system will not be significantly affected by failure of a one or even a few seismic isolation units or components. However, the seismic isolation layer connects the isolated superstructure to the substructure forming a series system (Figure 1). As such, the seismic isolation layer is not redundant. Therefore, engineering measures must be taken to prevent simultaneous or cascading failure of many seismic isolation units due to exceeding their deformation or force capacities. Such engineering measures include fail-safe system such as deformation limiters achieved by physical (bumpers, stoppers, walls) or mechanical (increasing isolator unit stiffness, slider breaks) means, and additional passive or semi-active dampers. These measures, when engaged, may result in impact and/or gradual increase in forces transmitted to the isolated superstructure. Fulfilling of performance statement 3 may require engineering measures to prevent simultaneous or cascading failure of many seismic isolation units due to exceeding their deformation or force capacities. Such engineering measures include fail-safe system such as deformation limiters achieved by physical (bumpers, stoppers, walls) or mechanical (increasing isolator unit stiffness, slider breaks) means, and additional passive or semi-active dampers. These measures, when engaged, may result in impact and/or gradual increase in forces transmitted to the isolated superstructure. Possible failure of isolator devices to carry gravity load may impose additional demands on the superstructure isolation mat and the foundation mat. Assuming that simultaneous failure of multiple isolators is prevented by the fail-safe systems described above, it may be sufficient to design the superstructure and the foundation mats to span the gravity loads over a single isolator. Ultimately, the design objective is to provide sufficient space for the isolation system to move without obstacles and for graceful degradation of the seismic isolation layer in extreme situations: a cliff-edge sudden seizure of its isolation function must be avoided.

ASCE 4-10 provisions for determining the design displacement demand for isolator units, and therefore for initial sizing of the isolator units are adequate. The applicant is expected to demonstrate that the nuclear facility and its seismic isolation system satisfy NRC requirement for a high confidence of low probability of core damage and large radiation release. The applicant may choose to satisfy this requirement by appropriately decreasing the probability of undesirable isolator system and isolator unit performance for beyond design-basis earthquake ground motions while keeping the remainder of the foundation and superstructure in essentially elastic response states. This can be achieved by appropriately increasing the isolator deformation capacity and, simultaneously, increasing the distance between the isolated superstructure and the physical motion limiters. On the other hand, the applicant may choose to show by modeling and analysis, or by testing, that the isolated nuclear facility structure can sustain the impact against physical limiters associated with seismic demands at the HCLPF level and maintain adequate safety margins against core damage and large radiation release. In either case, the applicant should demonstrate that the ability of the isolator devices and isolation system to transmit the gravity load of the isolated superstructure to the foundation is maintained.

Further research to investigate a number of challenges identified above is recommended. The nuclear facility engineering community will benefit by having a portfolio of validated and verified computer models of base isolator units likely to be used in the US suitable for use in existing software. These models, phenomenological, mechanistic or finite element, should model the coupled non-linear response of isolator units to combined three-directional displacement input. Such models enable investigation of the effects of horizontal-direction impact of the isolated superstructure against the physical limiters in beyond-design basis cases when the limiters are used to prevent simultaneous failure of multiple isolation units. In addition, such models enable investigation into the effects of vertical motion of the isolated superstructure due to vertical excitation or overturning due to horizontal excitation, propagation of vertical excitation into the isolated superstructure and its effect on equipment, and the benefits and costs of seismic isolation in the vertical direction. Finally, validated and verified models of isolator units enable analysis to determine the sensitivity of the seismic response of the isolated nuclear facility to variability in isolator unit mechanical properties, both due to unit-to-unit variation in production and due to property evolution over time.

Integrated research efforts may also be needed to examine the existing NRC regulatory framework to identify regulatory impediments for licensing of seismically isolated nuclear facilities and to propose modifications based on a risk-informed performance-based framework, if necessary. Similarly, integrated research is needed to develop verified and validated soil, foundation, isolator, and superstructure mode to assess the effect of soil-structure interaction on the seismically isolated nuclear facility structures.

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Definitions

This section contains definitions of basic concepts and terminology associated with seismic isolation and is consistent with the definitions of ASCE 4 to the extent possible.

Design displacement	The design basis earthquake horizontal displacement at the center of mass of the isolation system computed per ASCE 4.
Drift	
Effective damping	The value of equivalent viscous damping corresponding to energy dissipated during cyclic response of the isolation system.
Effective stiffness	The value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.
Isolation interface	The interface between the isolated superstructure and the supporting (non-isolated) substructure.
Isolation system	A collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system and all connections to other structural elements.
Isolator unit	A horizontally flexible and vertically stiff structural component of the isolation system that permits large lateral deformations under earthquake shaking.
Maximum displacement	The earthquake-induced horizontal displacement at the center of mass of the isolation system corresponding to 150% design basis earthquake shaking.
Substructure / Foundation	Primary structural systems and components that support the isolator units
Superstructure	Primary and secondary structural systems and components above the isolation interface