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Technical Considerations for Seismic Isolation of Nuclear Facilities

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Technical Considerations for Seismic Isolation of Nuclear Facilities

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Prepared by
A. M. Kammerer[1], A. S. Whittaker[2], and M. C. Constantinou[2]

[1]Formerly, Office of Nuclear Regulatory Research
US Nuclear Regulatory Commission

[2]MCEER, University at Buffalo
State University of New York, 212 Ketter Hall
Buffalo, NY 14260

Ramón L. Gascot, Scott P. Stovall, and
Richard Rivera-Lugo, NRC Project Managers

Office of Nuclear Regulatory Research

ABSTRACT

Seismic isolation (SI) is a technology that has proven useful in constructing structures capable of withstanding intense earthquake ground motions. Although various techniques for isolating a structure from the effects of earthquake shaking have been known for centuries, the global use of modern seismic (or base) isolation devices has greatly expanded in recent decades. Seismically isolated structures now number in the thousands around the world and the technology has been shown to meet the performance needs of important non-nuclear structures. SI has been used in the design and construction of nuclear power plants (NPPs) in France and South Africa and was recently used to isolate new emergency response centers at NPPs in Japan. As a result, SI is currently being considered for nuclear facilities in the United States.

Base isolation, an application of SI, reduces the response of a structure to horizontal ground motion through the installation of horizontally flexible and vertically stiff seismic isolators between the superstructure and its foundation. The dynamics of the supported structure change such that the fundamental vibration period of the isolated structural system is significantly longer than that of the fixed-base (non-isolated) structure. This leads to significant reductions in the horizontal accelerations and forces transmitted to the isolated superstructure (and the systems and components therein). The reduction in horizontal acceleration is accompanied by an increase in lateral displacement between the foundation and the superstructure, which is accommodated by deformation of the isolators. SI devices are designed and tested to undergo safely and with significant margin the large horizontal deformations expected in design basis shaking.

This report was developed under the NRC’s Seismic Research Program Plan with the goal of providing technical information necessary for NRC staff to develop regulatory guidance on the use of SI technology. This report develops a performance-based and risk-informed design philosophy for SI derived based on NRC objectives and approaches. This design philosophy, in turn, leads to a set of recommended performance objectives and criteria that can serve as the foundation for future NRC guidance on the use of SI and related technology.

This report is the first comprehensive NRC technical document related to SI. As such, it provides background information on seismic isolators and isolation systems, a discussion of the history and performance of seismically isolated structures during earthquakes, and a summary of SI provisions in relevant existing codes and standards. It also provides a series of preliminary recommendations that have resulted from the NRC’s research program on SI.

The focus of this NUREG/CR is surface-mounted or near-surface-mounted large light water reactors. Although vertical isolation systems, the isolation of individual systems or components, and the isolation of deeply embedded reactors are not discussed in detail in this NUREG/CR, there is no technical reason that the principles and recommendations contained herein cannot be extended to these and other cases. However, additional considerations, constraints and recommendations may be needed. Some are discussed briefly in this report.

FOREWORD

Appendix A, "General Design Criteria for Nuclear Power Plants," to 10 CFR Part 50, General Design Criterion (GDC) 2, "Design Bases for Protection Against Natural Phenomena," requires, in part, that nuclear power plant structures, systems, and components (SSCs) important to safety must be designed to withstand the effects of natural phenomena (such as earthquakes) without loss of capability to perform their safety functions. Such SSCs must also be designed to accommodate the effects of, and be compatible with, the environmental conditions associated with normal operation and postulated accidents.

The U.S. Nuclear Regulatory Commission (NRC) initiated the current research investigating the use of seismic isolation (SI) technology in nuclear power plants in 2008 in response to activities ongoing within the US nuclear industry focused on using SI technology as a way to meet the seismic requirements noted above. The importance of developing specific criteria for applying this technology has been highlighted by lessons learned from recent large magnitude earthquakes that have strongly shaken both isolated structures and non-isolated operating nuclear power plants, especially in Japan. The NRC research program on SI has been conducted jointly by the NRC's Office of Nuclear Regulatory Research, the Lawrence Berkeley National Laboratory, and MCEER (www.buffalo.edu/mceer.html) at the University at Buffalo, State University of New York. The research program focused on several technical topics and the resulting recommendations are included in this document.

The research that resulted in this NUREG/CR was intended to meet two key objectives:

1. Collect and summarize existing technical information on seismic isolation (SI) technology to inform NRC staff.

2. Develop performance and design recommendations and technical considerations addressing the design, construction, and operational needs for SI systems that consider the seismic performance of structures, systems, and components (SSC)

The emphasis of the NUREG/CR is the seismic isolation of nuclear facilities, with a focus on base isolation of surface-mounted or near-surface-mounted large light water reactors. Issues relevant to the analysis and design of isolation systems for deeply embedded reactors, non-light water reactors, and for individual systems and components are not addressed in detail in this NUREG/CR. Similarly, vertical isolation is not discussed in detail in this NUREG/CR. For uses of SI technology beyond those considered in this NUREG/CR, additional constraints, recommendations and studies may be needed. Additional studies would address further development of the technical bases for guidance for those uses as their necessary details become known.

To maximize the utility of the NUREG/CR, references are made to relevant consensus standards such as those for seismic analysis of safety-related nuclear structures (ASCE 4) and for seismic design criteria for structures, systems and components in nuclear facilities (ASCE 43). Elements of the standards have been included into the recommendations and options proposed in this report where appropriate. Additional performance criteria were developed to meet the risk-informed goals of the NRC. The technical considerations, recommendations and performance criteria provided in this report, coupled with the applicable provisions in the standards referenced in this document, provide a technical basis to inform development of regulatory guidance on the use of SI technology in nuclear facilities.

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The United States Nuclear Regulatory Commission (NRC), Lawrence Berkeley National Laboratory (LBNL), and MCEER at the State University of New York, University at Buffalo conducted this research project on seismic isolation. This project is a part of a broader NRC initiative at LBNL related to advances in structural analysis and design of nuclear installations. Dr. Robert Budnitz served as the technical lead at LBNL for the projects that have comprised this initiative.

A broad research review panel, formed to provide input to the various LBNL projects, provided significant and important input to this study. The research review group was composed of Dr. Robert Kennedy (RPK Structural Mechanics Consulting), Dr. Nilesh Chokshi (NRC), Mr. Donald Moore (Southern Nuclear Operating Company), Dr. James Johnson (James J. Johnson and Associates), Mr. Antonio Godoy (James J. Johnson and Associates and International Atomic Energy Agency (retired)), and Prof. Jon Stewart of UC Los Angeles. Dr. Robert Kennedy (RPK Structural Mechanics Consulting) chaired the review group. Prof. Gregory Fenves of the University of Texas, Austin was a member of the panel early in the project. Prof. Bozidar Stojadinovic (Eidgenössische Technische Hochschule, Zürich) also contributed to the early phases of this project.

Professor Yin-Nan Huang (National Taiwan University) provided significant technical input and also reviewed early drafts of the report. Dr. Sanjeev Malushte (Bechtel) provided information on isolation of NPPs. Dr. Troy Morgan (formerly of Tokyo Institute of Technology, now of Exponent) assisted with review of the translation of JEAG 4614-2000, as well as information on earthquake experience in Japan. Additional external review was also provided at NRC requests by Dr. Cameron Black (SIE), Dr. Claudia Marin (Howard University), Dr. Nebojsa Orbovic (Canadian Nuclear Safety Commission), Dr. Ovidiu Coman (International Atomic Energy Agency), and Dr. Tory Iijima and Dr. Hiroshi Abe (both staff at the former Japan Nuclear Energy Safety (JNES) Organization, now a part of Japan's Nuclear Regulation Authority). Comments were provided by the late Dr. John Stevenson, Dr. Pierre Sollogoub, Dr. Nadim Moussallam, Dr. Renzo Medeot, Massimo Fornia and Hamid Ahmadi. The provision of comments does not imply endorsement by any external reviewer. Dr. Manish Kumar and Dr. Brian Terranova of the University at Buffalo assisted with the preparation of the final draft of this NUREG/CR and their technical contributions are gratefully acknowledged.

The project benefitted from an ongoing bi-lateral seismic research cooperation program between the former JNES and the NRC. Under this bi-lateral agreement, JNES staff attended research review panel meetings to provide seismic isolation insights from a Japanese perspective.

This NUREG/CR also benefitted from on-going interactions with the International Atomic Energy Agency (IAEA) International Seismic Safety Center Special Working Group on Seismic Isolation. Technical discussions among working group members improved the completeness of the guidance and the discussions of practice in other IAEA Member States.

NRC technical staff provided detailed review comments on this NUREG/CR at different times throughout the term of the project. Special thanks are due to Dr. Jose Pires, Senior Technical Advisor for Civil and Structural Engineering.

ABBREVIATIONS, ACRONYMNS, AND PARAMETERS

A	bonded area of the elastomer in a rubber isolator (or bearing or unit)
ACI	American Concrete Institute
ALMR	Advanced Liquid Metal Reactor
ANS	American Nuclear Society
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
AASHTO	American Association of State Highway and Transportation Officials
BDB	beyond design basis
β_{eff}	effective damping ratio for an isolator
CEN	Comité Européen de Normalisation
CRIEPI	Central Research Institute of the Electric Power Industry
CSDR	certified seismic design response spectrum
D	bonded diameter of a rubber bearing
DBE	Design Basis Earthquake
DFBR	Demonstration Fast Breeder Reactor
Δ_{max}	maximum positive horizontal displacement of an isolator during prototype testing
Δ_{min}	minimum negative horizontal displacement of an isolator during prototype testing
EDC	energy dissipation per cycle of loading of an isolator
EdF	Électricité de France
EERI	Earthquake Engineering Research Institute
EFR	European Fast Breeder Reactor
F_{max}	horizontal force corresponding to Δ_{max}
F_{min}	horizontal force corresponding to Δ_{min}
FOSID	first onset of significant inelastic deformation
FP	Friction Pendulum™ isolator
F_y	effective yield strength of an isolator
g	acceleration due to gravity (9.81m/s2)
G	shear modulus of an elastomer
GDC	General Design Criteria
GE	General Electric
GMRS	ground motion response spectra

GMRS+	ground motion response spectra +
H_D	mean annual hazard exceedance frequency
HCLPF	high confidence of low probability of failure
HDR	high damping rubber
HITEC	Highway Innovation Technology Engineering Center
IAEA	International Atomic Energy Agency
IBC	International Building Code
IRIS	International Reactor Innovate and Secure
JAERI	Japan Atomic Energy Research Institute
JEA	Japan Electrical Association
JEAG	Japan Electrical Association Guide
JNES	Japanese Nuclear Energy Safety Association
K_d	post-yield stiffness of an isolator
K_{eff}	effective (secant) horizontal stiffness of an isolator
K_u	initial stiffness of an isolator
LDR	low damping rubber
LNG	liquefied natural gas
LR	lead rubber
MAFE	Mean Annual Frequency of Exceedance
MCE	Maximum Considered Earthquake
MPa	Megapascal
μ	coefficient of sliding friction
n	number of isolators in an isolation system
n_r	number of layers of rubber in an elastomeric bearing
NUPEC	Nuclear Power Engineering Corporation
NPP	nuclear power plant
NRC	Nuclear Regulatory Commission
NUREG/CR	Nuclear Regulatory Report
P_{F^*}	annual frequency of unacceptable performance
P_F	annual frequency of failure for a structure or element
P_H	annual frequency of exceeding a designated hazard level
PRISM	Power Reactor Innovative Small Module
PSHA	probabilistic seismic hazard analysis
PTFE	Polytetraflouroethylene

PWR	Pressurized water reactor
Q_d	characteristic strength
R	effective radius of curvature of a sliding isolator
R_P	probability ratio, which is the ratio of H_D to P_{F*}
R_R	risk reduction factor
S	first shape factor for a rubber bearing
SAFR	Sodium Advanced Fast Reactor
SC-I	Seismic Category 1
SC-II	Seismic Category 2
SDC	Seismic Design Category
SI	seismic isolation
SIDRS	seismic isolation design response spectrum
SPRA	seismic probabilistic risk assessment
SSC	structures, system, and components
SSE	safe shutdown earthquake
STAR-LM	Secure Transportable Autonomus Reactor-Liquid Metal
S-PRIM	power reactor innovate small module
T_b	effective period of a seismic isolation system
t_r	thickness of a single layer of elastomer in a rubber isolator
T_r	total thickness of elastomer in a rubber isolator (= $n_r \times t_r$)
u_y	effective yield displacement of an isolator
US	United States
UHRS	uniform hazard response spectrum
W	weight supported by an isolator or the isolation system

1 INTRODUCTION

In recent years, there has been a surge in the development of structural engineering technologies for application to conventional (or non-nuclear) structures. Many of these new technologies have the potential to increase the seismic safety of new build nuclear power plants and to reduce the cost and time to build them. Seismic isolation (SI) is one such technology because of its proven ability to substantially reduce the seismic response of a structure, and by extension, the response of the structures, systems and components (SSCs) in a nuclear power plant (NPP).

Base isolation is an application of SI that reduces the response of a structure to horizontal ground motion through the installation of a horizontally flexible and vertically stiff layer of seismic isolation devices between the superstructure and its substructure. The dynamic response of the supported structure to earthquake shaking is changed by addition of isolators[1] such that the fundamental vibration period[2] of the entire isolated structural system is significantly greater than that of the original, non-isolated, structure. This leads to significant reductions in the accelerations and forces transmitted to the isolated superstructure (and to the components and systems contained therein) during strong earthquake shaking. The reduction in acceleration is accompanied by an increase in lateral displacement between the foundation and superstructure, which is accommodated in the isolators. Seismic isolators are designed and tested to undergo safely the expected horizontal deformations for design basis and beyond design basis ground motions, while supporting axial loads due to gravity and earthquake shaking.

The concept of seismic isolation is not new. Although various techniques for isolating a structure from the damaging effects of earthquake ground motion have been understood for centuries, modern seismic isolators and related codes and standards were actively developed in 1980's and 1990's for application to non-nuclear structures. Since that time, a significant number of conventional buildings, industrial structures, and bridges have been seismically isolated in the United States (US) and abroad. During the 1980's seismic isolation was also used in the design and construction of nuclear power plants (NPPs) in France and South Africa, and considered for nuclear facilities in Japan and the US. In the US, two seismically isolated reactor designs, Power Reactor Innovative Small Module (PRISM) Liquid-Metal Reactor and Sodium Advanced Fast Reactor (SAFR), were developed with Department of Energy support and examined by NRC in the mid 1980's and early 1990's. Although the downturn in nuclear power plant construction since the early 1990s hindered further development of SI technology for nuclear facilities in the US, the development and use of SI for conventional structures continued and the technology matured in the US and abroad. Recently regulatory guidance in Japan was changed to allow the use of SI technology at NPPs and new isolated emergency response centers have been constructed at NPP sites in Japan.

The purpose of this report is to provide the technical information necessary for NRC staff to develop potential new regulatory guidance on the use of SI technology. The study was performed under the NRC's Seismic Research Program Plan (2008-2011) and the reports presents a risk-informed, performance-based design philosophy for SI that is consistent with NRC objectives and approaches. This design philosophy, in turn, leads to a set of performance objectives and criteria

[1] In this report, the terms *isolator*, *bearing*, and *isolator unit* are used interchangeably. An isolation system is an assembly of seismic isolators, generally in one horizontal plane.
[2] The fundamental vibration period is the inverse of the fundamental frequency. Although frequency is typically used for the description of NPPs, period is typically used in the SI literature.

that could serve as the foundation for future NRC guidance on the use of SI and related technology. This report is the first of several detailed NRC technical documents related to application of SI to NPPs. As such, it provides introductory background information on SI, a discussion of the history and performance of isolated structures during past earthquakes, a summary of treatment of SI in codes and standards, and a series of recommendations and options that have resulted from the NRC's research program on SI.

This study focuses on base isolation of NPPs using horizontal isolation systems. Base isolation is provided in the horizontal direction only. This report addresses vertical SI, the isolation of individual systems or components, and the seismic isolation of embedded reactors and other advanced designs to a limited extent[3]. However, technical reasons for precluding these applications have not been identified. The principles and recommendations in this document could be extended to other cases, although additional considerations, constraints and recommendations may be applicable in these cases.

1.1 Potential Benefits of Seismic Isolation

Earthquake effects remain one of the principal contributors to the total risk of core damage and large radiation dose release in US nuclear power plants. Seismic isolation has the potential to significantly reduce the contribution of seismic hazard to the overall risk (e.g., Huang *et al.* (2008, 2009c)). SI can be used to effectively reduce the horizontal accelerations and deformations that develop in SSCs to values well below those in fixed-base NPPs for both design basis and beyond design basis shaking.

SI could benefit the analysis, design, review and construction of new build nuclear power plants by enabling the use of one (or more) certified plant design(s) across a broader range of site types and higher seismic hazard levels. SI systems would be used in areas of high seismicity where NPP design would be especially challenging. In this report it is assumed that a certified design would encompass the superstructure, and that site-specific isolation systems would be used to reduce transmitted accelerations to below the specific certified seismic design response spectrum (CSDRS) level. An isolated NPP taking advantage of the reduced earthquake effects on SSCs could be designed and certified to the CSDRS. As for non-isolated power plants, geotechnical considerations, including avoiding resonance between the isolation superstructure and the soil profile, would continue to be fully described as part of the certified design documentation.

1.2 Objective of the NUREG/CR

The objective of this report is to develop and summarize a set of technical considerations, recommendations and options that could serve as the basis for regulation and regulatory review of the design, construction, and operation of seismically isolated NPPs.

[3] Vertical isolation systems for *vibrating* machinery such as pumps and turbine generator sets have been deployed for many years.

The research effort, whose results are summarized in this report and Kumar *et al.* (2015a, 2015c), had two goals:

1. Collect and summarize existing technical information on seismic isolation (SI) technology to inform NRC staff.

2. Develop performance and design recommendations and technical considerations addressing the design, construction, and operational needs for SI systems that consider the seismic performance of structures, systems, and components (SSC).

To the extent possible, this report refers to relevant standards, including those for seismic analysis of safety-related nuclear structures, ASCE 4-16 (ASCE, 2017) and for seismic design criteria for structures, systems and components in nuclear facilities, ASCE 43-05 (ASCE, 2005). However, additional performance criteria and guidelines were developed to address the NRC's performance goals and objectives. This report was also designed to be a useful reference to those engineers planning to use SI in their designs, as well as regulators reviewing applications that employ SI.

1.3 Scope

To meet the objectives listed above, this report provides introductory background information on SI, a discussion of the history and performance of seismically isolated structures during past earthquakes, a summary of the treatment of SI in relevant codes and standards, and a series of preliminary recommendations that have resulted from the NRC's research program on SI.

The technical considerations presented herein apply specifically to the base isolation of nuclear facilities. The isolation of individual systems and components has not been discussed in detail, although nothing has been identified to preclude a priori other uses of SI technology, and indeed, many of the principles set forth in the NUREG/CR would be relevant and appropriate. In the discussion that follows it is assumed that the isolators will be installed in a radiation-free environment; the prolonged effects of radiation exposure on isolators is not discussed. If isolators were to be exposed to radiation, other considerations such as possible related changes in material properties, additional in-service monitoring, and consideration of other relevant conditions would have to be addressed.

This NUREG/CR assumes the isolation of surface or near-surface-mounted, safety-related structures such as large light water reactors. Issues specific to the analysis and design of isolation systems for deeply embedded reactors, or components thereof, are not addressed in detail in this report because those designs will likely differ significantly from the NPPs assumed here. However, this focus should not be interpreted to suggest that SI technology cannot address design challenges related to small modular reactors and deeply embedded reactors.

The SI system is treated in this document as part of the structural system of a NPP. The risk-informed design of the superstructure of the NPP continues to be governed by the performance objectives defined in ASCE 43-05, amended as appropriate by NRC guidance. However, as described in Section 8 of this report, recommended performance criteria for the individual isolator units, the isolation system, the stop and umbilical lines are provided for both a design basis ground motion response spectrum and a beyond design basis ground motion response spectrum. The latter provides criteria for review of the performance of the isolations system to assure that safety-related functions can be achieved with margin that addresses intended safety goals. As described in Section 8, the recommended performance criteria for the isolation system differ from those provided in ASCE 43-05, which were developed for non-isolated light water reactors and targeted NPP-level risk goals. The criteria herein also targets NRC performance objectives (similar to those in ASCE 43-05), but are applicable to the isolation system and can be adapted to a wider range of NPP designs.

Three types of seismic isolators are in the scope of this study. These are Low Damping Rubber (LDR) bearings, Lead Rubber (LR) bearings, and spherical sliding Friction Pendulum™ bearings. The rubber bearings utilize low damping natural rubber as the base compound with predictable and reproducible behavior. All three types of isolators provide a restoring force to re-center the isolation system after an earthquake. These are desired features for use in nuclear power plant applications. These three types of bearings have been widely used in the US for critical structures and have a proven track record. The scope of this report is therefore limited to these three types of isolators. Discussion of the behavior, mechanical properties, modeling, structural response analysis, design, qualification and in-service monitoring issues for these isolators are provided in this report.

To the extent possible, the technical considerations provided herein are compatible with other resource documents on the seismic isolation of infrastructure, such as the standard for minimum design loads for buildings and other structures, ASCE 7-10, (ASCE, 2010), and the specification for the seismic isolation design for bridges (AASHTO, 2010). These standards are described in this report. However, some of the recommendations presented in this report differ from those provided in these codes to address NRC's use of risk-informed performance goals and objectives. Significant effort has been made to coordinate the contents of this report with the seismic isolation provisions in ASCE 4-16.

2 A BRIEF HISTORY OF SEISMIC ISOLATION

2.1 Seismic Isolation of Non-Nuclear Structures

The idea of decoupling a structure from the ground to avoid the damaging effects of earthquake shaking by rocking or sliding date back to the Classical period of Greece and Persia (600 to 300 BC) and to 12th century Japan. As a strategy to protect structures from earthquakes, base or seismic isolation (SI) emerged formally in 1870 with a proposal for a double concave rolling system (US Patent No. 99,973; Fenz and Constantinou (2006)) and returned more than 30 years later (in the early 20th century) with several proposals for sliding systems (e.g., Naeim and Kelly (1999)). However, it is only in the past 30 years that advances in materials technology and manufacturing have enabled the fabrication of isolators of a sufficient size and quality suitable for protection of critical structures. Buckle and Mayes (1990) and EERI (1990) provide much information on developments in the field of seismic isolation in the period 1970 through 1990.

The development of elastomeric and sliding isolators for bridges in the 1950s was a precursor to development of modern seismic isolators. Researchers in the 1970s in New Zealand (e.g., Blakeley *et al.* (1979); Skinner *et al.* (1993)) developed and tested simple low damping and lead-rubber bearings and models supported on such bearings. Since that time, researchers and engineers in dozens of countries have refined isolation technology and implemented it buildings, bridges and infrastructure. Key developments in the past 30 years have occurred in the United States (US) and Japan.

Since the first application of a pendulum-type isolation system to a building in Ashgkabad, Turkmenistan (in the former USSR) in 1955, more than 6000 buildings have been constructed or retrofitted worldwide with seismic isolation (SI) technology. 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 (Japan) earthquakes. After the 1995 Kobe earthquake, the use of SI in Japan expanded quickly because academicians, design professionals, and regulators acknowledged that base isolation could eliminate structural damage and loss of life, even in severe earthquake shaking. Although the US has been one of the forerunners in creating and testing SI technology, implementation in the US has proceeded more slowly, with applications focused on new bridges, new critical facilities, and existing architecturally and historically important structures. The difference in the pace of adoption of SI in Japan and the US can be attributed to many factors, including construction practices, performance expectations for typical building and bridge construction, and regulatory environments.

2.1.1 Seismic Isolation of Non-Nuclear Structures in the United States

In the US, engineers have implemented SI in many different types of structures, ranging from highway bridges to government buildings to offshore oil and gas platforms. Many of these structures support essential or critical services (e.g., hospitals and emergency response centers) and, therefore, require higher levels of performance during earthquake shaking than typical (non-essential) structures. Modern US design codes for non-nuclear structures (e.g., ASCE 7-10 (ASCE, 2010) and the AASHTO *Guide specification for seismic isolation design* (AASHTO, 2010)) include mandatory language that describes how the seismic isolation system is to be analyzed and designed, and individual isolators or bearings are to be tested. Chapter 5 summarizes these design procedures.

It is generally accepted that seismically isolated buildings will perform markedly better than modern, non-isolated (or fixed base) buildings. The performance goal for the latter is a low probability of collapse in the event of maximum considered earthquake (MCE) shaking. Non-repairable damage is tacitly accepted. In stark contrast, the structural and mechanical components in a seismically isolated building, if analyzed, designed, and detailed correctly, should not be damaged by MCE shaking. Because there is a cost premium to construct a seismically isolated building, SI is only considered in the US for those new buildings with substantially higher performance requirements than the minimum associated with codes of building practice.

Most of the applications of seismic isolation to date by US design professionals have involved critical structures. Some examples are provided in Figure 2-1 for construction in regions of high seismic hazard. Figure 2.1(a) is a photograph of an offshore platform in the Sakhalin II natural gas field that was seismically isolated using 4 single concave Friction Pendulum (FP) bearings (Clarke *et al.*, 2005). Figure 2-1(b) is a photograph of the US Court of Appeals Building in San Francisco that was damaged in the 1989 Loma Prieta earthquake and retrofitted using FP bearings. Figure 2-1(c) is a photograph of the San Francisco City Hall, which was also damaged in the 1989 Loma Prieta earthquake and retrofitted using lead-rubber bearings. Figure 2-1(d) is an approach to the Golden Gate Bridge, which was also retrofitted using lead-rubber bearings. Each of these structures is required to remain operational after rare or very rare earthquake shaking, which is a performance expectation that is much higher than for commercial buildings designed to ASCE 7-10.

2.1.2 Seismic Isolation of Non-Nuclear Structures in Japan

As noted above, the use of SI technology in Japan rapidly increased since the 1995 Kobe earthquake. This rapid growth means that any count of the number of applications of SI technology in Japan quickly becomes out of date. However, the number of isolated structures is in the thousands. SI technology has been applied to all kinds of kinds of structures, bridges, and industrial and liquefied natural gas facilities in Japan. The use of SI technology in non-nuclear structures is generally governed by the professional codes, most notably those developed by the Japan Society of Seismic Isolation and the Architectural Institute of Japan. Most applications in Japan have used elastomeric isolators.

2.1.3 Seismic Isolation of Non-Nuclear Structures in Other Countries

There have been many applications of seismic isolation outside of the US and Japan. A few are described below to illustrate isolation of other types of critical structures. Figure 2-2 presents applications in Greece, South Korea and Turkey. Figure 2.2(a) is a photograph of FP bearings mounted on pedestals prior to the installation of the basemat for a large liquefied natural gas (LNG) tank at Revithoussa, in Greece. Figure 2-2(b) is a photograph of the isolators, installed atop pedestals (to enable isolator inspection and replacement) and below the basemat at Revithoussa. Figure 2-2(c) shows elastomeric bearings installed below LNG tanks in Inchon, South Korea (also atop pedestals). Figure 2-2(d) is a photograph of the roof of the Ataturk International Airport in Istanbul, Turkey, which is seismically isolated from the superstructure below with single concave FP bearings. The applications in Greece and Turkey are in regions of high seismic hazard. The seismic hazard at Inchon is low to moderate.

(a) Sakhalin II platform (courtesy of Shell)   (b) US Court of Appeals, San Francisco

(c) San Francisco City Hall   (d) Golden Gate bridge, San Francisco

Figure 2.1 Examples of Application of Seismic Isolation of Important and Critical Structures and Infrastructure

2.2 Response of Isolated Non-Nuclear Structures during Past Earthquakes

Significant earthquakes in the US, Japan and Turkey in the past 20 years have provided significant data, information, and lessons to enable the evaluation of the performance of seismically isolated structures. Some of the notable case studies are described below.

2.2.1 1994 Northridge Earthquake, California

The 1994 Northridge earthquake shook several seismically isolated structures in the Los Angeles area of California. One isolated structure was the University of Southern California University Hospital, an 8-story structure with highly irregular plan (Asher *et al.*, 1990). During the Northridge earthquake, the peak acceleration recorded at the foundation level of the building was 0.37g. The isolation system, a combination of LR and LDR bearings, reduced the peak horizontal acceleration at the roof to 0.21g. The hospital was undamaged by the significant earthquake shaking (e.g., Asher *et al.* (1995); Nagarajaiah and Xiaohong (2000)).

(a) LNG tank base, Revithoussa, Greece

(b) FP bearings below LNG tank basemat

(c) Rubber bearings, Inchon, South Korea

(d) Ataturk Airport, Istanbul, Turkey

Figure 2.2 Other Applications of Seismic Isolation in Critical Facilities

2.2.2 1995 Kobe Earthquake, Japan

The 1995 Kobe earthquake shook several isolated structures in Kobe, Japan. Higashino and Okamoto (2006) report the behavior of two seismically isolated buildings in the epicentral region: the Post and Telecommunication Building located 35 km from the epicenter of the earthquake, and an office building located in Kobe.

The Post and Telecommunication building is a six-story structure located near Kobe. The framing system is composed of steel reinforced concrete columns and H-shaped steel beams. The isolation system is composed of natural rubber bearings, lead-rubber bearings and steel dampers. The peak horizontal floor accelerations were a factor of 3 smaller than the peak horizontal foundation accelerations.

The three-story office building in Kobe was isolated with high-damping rubber bearings. The isolation system was not as effective as that at the Post and Telecommunication building, with peak horizontal floor accelerations smaller than the peak horizontal foundation acceleration in one direction and of the same magnitude as the peak horizontal foundation acceleration in the perpendicular direction.

2.2.3 1999 Duzce earthquake, Turkey

The construction of the 2.3-km long seismically isolated Bolu viaduct in Turkey was nearly completed when it was shaken by the 1999 M7.2 Duzce earthquake. The seismic isolation system failed and the bridge narrowly avoided total collapse (see Figure 2-3) due to excessive superstructure movement (Roussis *et al.*, 2003). An evaluation of the *design* of the isolation system revealed that it did not meet AASHTO requirements for the site of the viaduct. The failed isolation system had insufficient displacement capacity and lacked restoring force capability. This type of isolation system would not meet the requirements of this NUREG/CR. The viaduct was subsequently retrofitted with Friction Pendulum™ spherical sliding bearings with a sufficiently large displacement capacity.

2.2.4 2003 Tokachi-Oki Earthquake, Hokkaido, Japan

Higashino and Okamoto (2006) documented the response of four seismically isolated buildings that experienced the 2003 Mw 8.0 Tokachi earthquake in Japan. These buildings employed elastomeric isolation systems and supplemental damping devices. All four sites experienced peak ground accelerations between 0.22 g and 0.30 g, which were significantly greater than those observed in the isolated structures. The maximum isolator displacements ranged between 12 cm and 30 cm, and no damage was reported in any of the four buildings.

2.2.5 2004 Chuetsu Earthquake, Niigata, Japan

At least seven isolated structures in the Niigata Prefecture of Japan were subjected to strong shaking from the 2004 Mw 6.8 Chuetsu earthquake. These buildings were isolated with either elastomeric bearings or elastomeric bearings in combination with flat sliders. One of the isolated buildings, which suffered no damage to its structural framing, nonstructural systems and contents, was reportedly used as an evacuation facility for patients of a nearby hospital (Higashino and Okamoto, 2006). This building's isolation system experienced a maximum displacement of approximately 15 cm. Sensors were installed above and below the isolators, and the records indicate the seismic isolation system reduced the horizontal inertial forces by 75% (Tamari *et al.*, 2006).

2.2.6 2005 Fukuoka Earthquake, Japan

At least nine isolated buildings were located in the region struck by the 2003 Mw 7.0 Fukuoka Japan earthquake in 2003 (Kani *et al.*, 2006). The isolation systems in these buildings consisted of elastomeric bearings and supplemental damping devices. There was no damage to the structural framing and non-structural components aside from an entry stair (damaged due to insufficient vertical clearance of the concrete slab above the isolation interface) and handrails at an expansion joint. Isolator displacements ranged from 12 cm to 30 cm, and base accelerations were reduced by 50% to 70% in the superstructure with respect to the foundation below the isolators.

(a) Excessive superstructure movement (b) Damaged energy dissipater

Figure 2.3 Failed Isolation System in the Bolu Viaduct in Turkey

2.2.7 2011 Tohoku Earthquake and Tsunami, Miyagi, Japan

A devastating Mw 9.0 earthquake struck the coast of the Miyagi Prefecture in Japan on March 11, 2011. Although some areas were affected by significant ground shaking, the majority of property damage and casualties were a result of the tsunami triggered by the undersea earthquake. Saito *et al.* (2011) reported a post-earthquake survey of 17 seismically isolated buildings in the Tohoku region, 16 in the Miyagi prefecture and one in Yamagata prefecture. All 17 buildings had elastomeric isolation systems of either high-damping rubber or natural rubber in combination with metallic yielding or viscous dampers. The National Institute for Land and Infrastructure Management (2011) documented the building damage survey.

There was no structural damage observed in any of the surveyed isolated buildings. Peak isolator displacements measured between 20 cm and 40 cm. Not all 17 buildings were instrumented with accelerometers but those that were exhibited basement peak accelerations of around 0.40 g or less. Non-structural damage included crushing of ceiling panels near expansion joints, loss of ceramic tiles on an exterior wall, and movement or overturning of some free-standing contents such as refrigerators. Minor shear cracking was observed in a basement wall of one building but this was attributed to local soil subsidence of around 10 cm. No cracking or bulging of the rubber bearings was observed. In one building, cracking was observed in a shaped supplemental lead damper[1].

Two full-scale demonstration buildings were constructed side-by-side at Tohoku University to judge the efficacy of seismic isolation. One was constructed with a fixed-base and the other was isolated. The two buildings experienced moderate shaking in this earthquake. The peak horizontal roof accelerations in the isolated building were less than one half those in the conventionally constructed building (Nakamura *et al.*, 2011).

[1] Lead dampers of this type are not used in US practice

Damage was observed at the expansion joints in some of the buildings due to either poor detailing or the sacrificial nature of the joint. Ground subsidence around some isolated buildings was a potential contributor to this damage.

2.2.8 Summary of Earthquake Experience in Non-Nuclear Structures

The examples presented above of the performance of isolation systems during moderate-to-severe shaking illustrate the following key points that are addressed in the following chapters of this report.

1. The use of robust and well-designed seismic isolators deliver the performance expected of a seismically isolated structure.
2. The underestimation of seismic hazard or use of inappropriate analysis methods can lead to unacceptable performance of seismically isolated (and conventional) structures.

Careful attention must be paid to the characterization of the design basis and beyond design basis ground motions, appropriate nonlinear methods of analysis must be employed (unless the isolators are low damping rubber bearings), sufficient displacement capacity must be provided in the bearings for design basis and beyond design basis shaking, and isolators must be tested to demonstrate adequate displacement capacity for beyond design basis shaking and the clearance to the stop (see Chapter 8 for a discussion of the stop). Particular attention should be paid to expansion joints and other non-structural components sensitive to damage due to displacement at the isolation interface. Regulatory and peer review of the analysis and design of seismically isolated nuclear structures should be mandatory, as is currently the case in the US for isolated buildings and bridges.

2.3 Nuclear Applications of Seismic Isolation

2.3.1 Introduction

Seismic isolation was applied to the construction of nuclear power plants (NPPs) in Cruas, France and Koeberg, South Africa in 1983 and 1984, respectively (Malushte and Whittaker, 2005). To date, six NPP units have utilized SI. All six were constructed in France and South Africa as described below. The Tokamak fusion reactor and the Jules Horowitz research reactor, both being constructed in Caderache, France are being seismically isolated.

In response to the 2006 changes to the Japanese Seismic Nuclear Regulations and the 2007 Niigataken-Chuetsu-Oki earthquake that impacted the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP), Japanese utilities decided to seismically isolate emergency operations buildings. As a result, seismically isolated emergency response centers have now been constructed at the sites of Japanese NPPs.

While only a handful of publically available light water reactor designs include seismic isolation (these include the IRIS (International Reactor Innovative and Secure) and 4S (Super Safe, Small and Simple)), most of the fast reactors were designed to include seismic isolation. These include the ALMR (Advanced Liquid Metal Reactor), S-PRISM (Power Reactor Innovative Small Module), DFBR (Demonstration Fast Breeder Reactor), DFBR (Demonstration Fast Breeder Reactor), STAR-LM (Secure Transportable Autonomous Reactor-Liquid Metal) and EFR (European Fast Breeder Reactor). None of these designs has yet been licensed.

2.3.2 France and South Africa

There are currently six seismically isolated reactors globally. They are located at two sites and all six are Pressurized Water Reactor (PWR) units constructed in the 1980s. Four reactors are located at the Cruas-Meysse site in France and two are located at the Koeberg site in South Africa. Isolators in these NPPs use neoprene as the elastomer.

According to Électricité de France, seismic isolation was implemented at the Cruas-Meysse NPP to allow use of a standard plant design (Labbe, 2010). The isolation system allowed the Cruas-Meysse NPP, which was to be designed for a peak ground acceleration (PGA) of 0.3g, to be constructed using a standard 900 MWe plant, which was designed for a PGA of 0.2g. Use of seismic isolation allowed the NPP to be constructed without an increase in the volume of steel and concrete in the structures and also with fewer seismic restraints and supports for equipment on the nuclear island. Additionally, the qualification tests performed for the 900MWe standard design could be used. Their experience indicated that the cost savings that resulted from use of the standard design was greater than the increased cost of design and construction of the seismic isolation system.

At the Cruas-Meysse plant, each of the four units was constructed on 1,800 neoprene pads. Each pad measures 500 mm x 500 mm x 65 mm. The seismic hazard levels used for design at the Cruas-Meysse site is moderate by international standards with a safe shutdown earthquake (SSE) ground motion anchored to a peak ground acceleration of 0.20g.

In Koeberg, each of the two units is isolated on a total of 2000 neoprene pads. Each pad measures 700 mm x 700 mm x 100 mm. The SSE peak ground acceleration used in design was 0.30g. The pads at the Koeberg NPP are also equipped with flat sliders on the top surface. The sliders consist of a lead-bronze alloy lower plate and a polished stainless steel upper plate. The sliding feature was implemented so that the lateral force transmitted to the reactor vessel is limited to the frictional resistance of the sliding interface.

Seismic isolation is also being deployed in the Jules Horowitz Reactor, now under construction, and at the Georges Besse II Enrichment Facility, both in France. Seismic isolation, similar to that used at the Cruas-Meysse NPP, was also used to protect three spent fuel storage pools at the La Hague reprocessing plant.

The synthetic rubber isolators used in these NPP applications have not been accepted by the US technical community. Because of the reported long-term changes in the mechanical properties of the elastomers, they are not recommended for application to a US nuclear facility. The synthetic rubber (a neoprene) used in the French isolators, has stiffened significantly (37%) over time, changing the properties of the isolation system. The isolator properties are monitored and isolators are changed out as necessary (Labbe, 2010). The bimetallic interface used in the South African isolators is no longer considered viable for use in seismic bearings because the mechanical properties of such interfaces can change substantially with time (Lee, 1993)[2].

2.3.3 United States

In the late 1980s, General Electric (GE) submitted a pre-application review to NRC for its seismically isolated Power Reactor Innovative Small Module (PRISM) reactor as described in

[2] This NUREG/CR strongly recommends the use of seismic isolators with a proven track record in the United States (i.e., low damping rubber, lead-rubber, and Friction Pendulum™ bearings) for applications to US NPPs. Recommended minimum requirements to bring a new seismic isolation technology (e.g., synthetic rubber isolators) to the NRC are provided in Section 3.3.

NUREG 1368 (NRC, 1994). Although this review found sufficient data existed at that time to warrant further development of a design application, the PRISM project did not continue for non-technical reasons. Mayes *et al.* (1990) summarizes the design and the projected cost savings due to SI. Since that time, no application has been submitted to NRC for a seismically isolated NPP. As a result, no NRC regulatory guidance for design and evaluation of seismically isolated nuclear structures was prepared.

However, both the NRC and the Department of Energy have recently performed and supported research that has developed the technical basis for the recommendations provided in this NUREG/CR. Chapter 12 of ASCE 4-16 (ASCE, 2017) provides guidance for design of seismically isolated nuclear facility structures in accordance with performance and risk goals defined in ASCE 43-05 (ASCE, 2005) which is itself based on performance criteria in US Department of Energy Standard DOE-STD-1020-2002 (DOE, 2002).

2.3.4 Japan

In 2006, the Nuclear Safety Commission of Japan revised the regulatory guidance for seismic design of Japanese NPPs. This revision removed the specification stating that buildings are to remain essentially rigid and explicitly allowed for the use of seismic isolation. In 2007, the Niigataken-Chuetsu-Oki Earthquake caused strong shaking at the Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP). While the earthquake did not damage any safety-related systems, it did cause significant damage to the plant’s emergency operations center. As a result, several Japanese utilities, including the Tokyo Electric Power Company, began a program to seismically isolate all of their emergency operations center buildings. With the exception of the emergency response centers discussed above, there are no seismically isolated safety-related nuclear facilities in Japan at this time.

3 BASICS OF SEISMIC ISOLATION

3.1 Introduction

A seismic isolation (SI) system reduces the response of a structure to horizontal ground motion. Base isolation[1] is achieved typically by installing horizontally flexible and vertically stiff seismic isolators (also typically called isolation bearings or isolator units) between the superstructure and its foundation. The isolators serve two key functions: 1) supporting gravity loads, and 2) protecting of the supported structure and its contents from the damaging effects of horizontal earthquake shaking. In non-nuclear buildings, isolators are required to protect the structure from earthquake ground motions with an annual frequency of exceedance of 4×10^{-4} (return period of 2500 years). Isolators are typically installed immediately below columns or walls. In isolated nuclear power plants (NPPs), consistent with Regulatory Guide 1.208 (NRC, 2007a), seismic protection is required for ground motion levels with an annual frequency of exceedance of less than 1×10^{-4} (a return period of greater than 10,000 years).

Isolators beneath the NPPs will likely be located around the perimeter of the structure and below the internal structure as a minimum, although the use of a thick and strong basemat above the isolators enables more flexibility with respect to isolator location than is possible in commercial buildings constructed using beams, columns and spread footings.

Figure 3-1 is a schematic cross section through a seismically isolated nuclear power plant (NPP) structure. Table 3-1 provides definitions of terms used in this NUREG/CR. The isolators are installed atop pedestals in a space below the basemat. The figure identifies the superstructure (entire structure above the isolators, in this case a containment structure), the basemat (structural foundation above the isolators), an isolator, a pedestal, the foundation (including and below the pedestals), and the isolation gap (or moat) within which the isolated superstructure can move without restriction. Pedestals are used to facilitate inspection and possible replacement of an isolator.

The addition of horizontally flexible elements (isolators) below a stiff superstructure increases the fundamental period of vibration of the overall superstructure. Typical superstructures in NPPs have fundamental periods of the order of 0.1 second. The addition of seismic isolators will generally increase this period to 2 or more seconds. Figure 3-2(a) illustrates the benefits of seismic isolation using an acceleration response spectrum; the fundamental periods of the superstructure and isolated superstructure are shown together with the reduction in horizontal acceleration enabled by the addition of the isolation system. Figure 3-2(b) shows a displacement spectrum and identifies the increase in displacement associated with the addition of the isolation system. Hysteretic damping resulting from yielding lead (in the lead-rubber bearing) or friction (in the Friction Pendulum™ bearing) is used to reduce the isolator displacement. Supplemental fluid viscous dampers (or lead dampers commonly used in Japan) can also be used to reduce isolator displacements. Nearly all of the displacement in the isolated structure develops over the height of the isolators. The isolators are designed and tested to simultaneously accommodate large displacements and axial loads as described later in this report.

The structural framing above (basemat) and below (foundation including the pedestal and moat walls) the isolators must be designed and detailed to accommodate the forces delivered by the isolators. These forces include axial loads, shearing forces, and bending moments, where the

[1] Base isolation is only one application of SI and is the focus of this report. However, other forms of isolation are also used and include floor and equipment isolation.

bending moment around each horizontal axis can be calculated as sum of the product of the shear force and bearing height and the product of the axial load and horizontal displacement. The foundation must be sufficiently stiff to ensure that the load of the superstructure is appropriately distributed to the individual isolators.

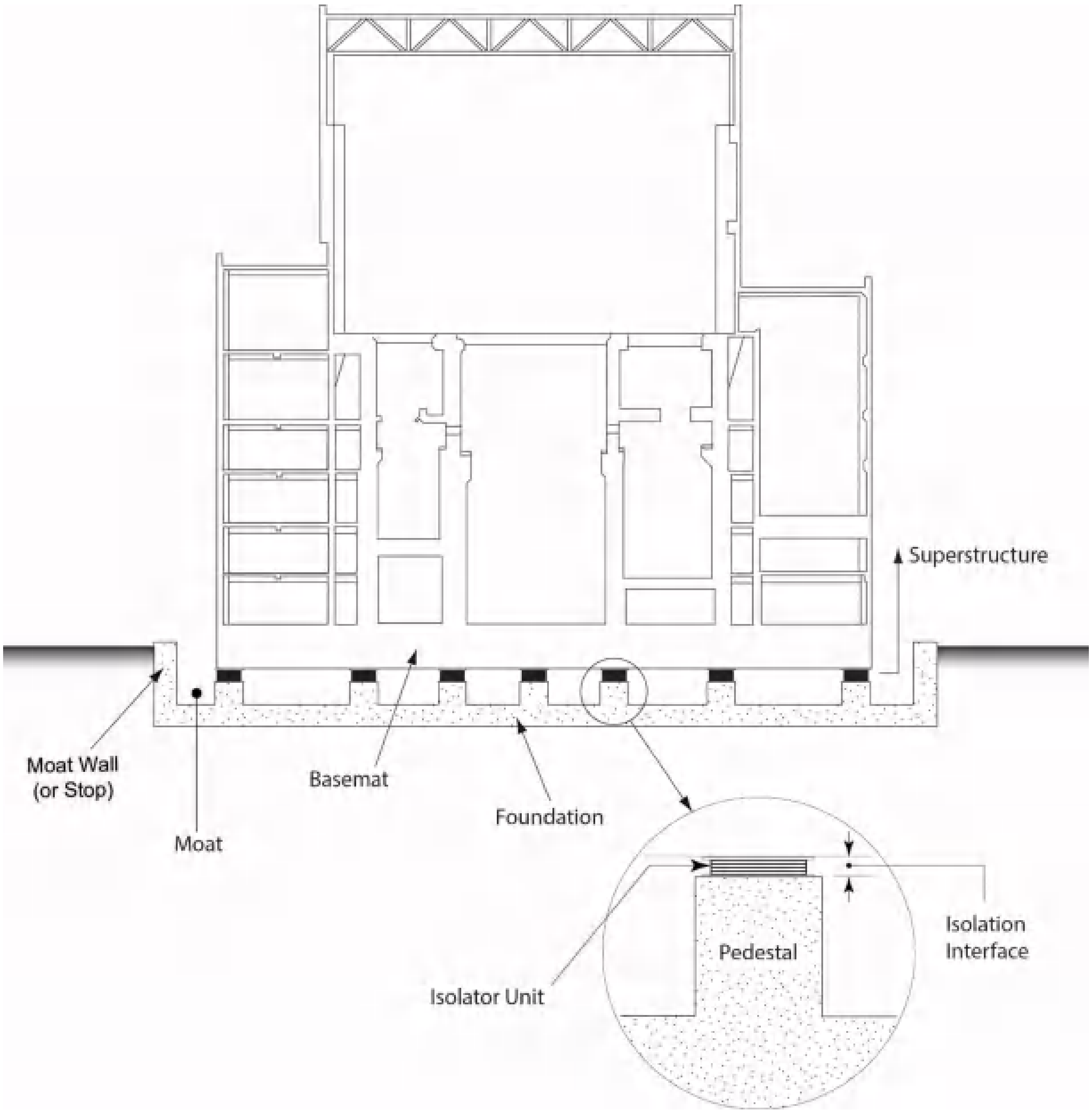
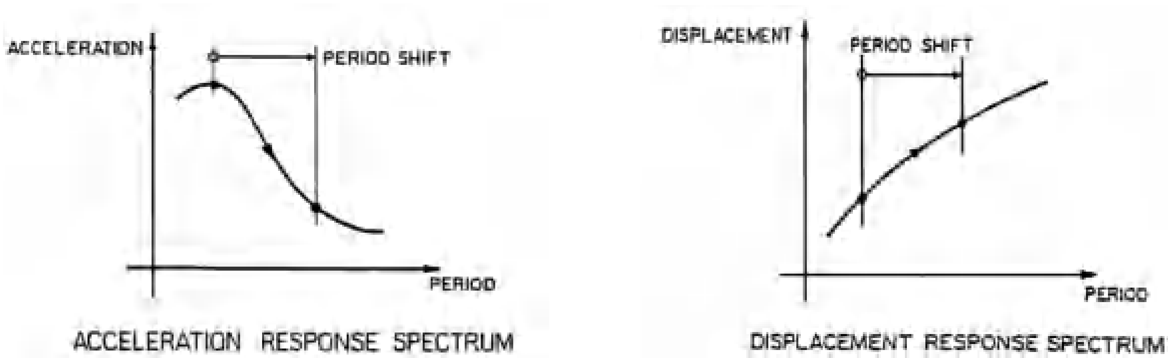


Figure 3.1 Cross Section of a Seismically Isolated Nuclear Structure

Table 3-1 Definitions of Terms

Term	Definition
Basemat	Thick reinforced concrete diaphragm immediately above the isolation system
Drift	Relative horizontal displacement between any two (initially) vertically aligned points in the structure
Effective damping ratio	The value of equivalent viscous damping corresponding to the energy dissipated during cyclic response of the isolation system.
Effective horizontal stiffness	The value of the lateral force in the isolation system, or an element thereof, divided by the corresponding lateral displacement.
Foundation	Reinforced concrete foundation (including pedestals) below the isolators that support the isolators. The moat walls are part of the foundation.
Isolation interface	The interface between the isolated superstructure and the supporting (non-isolated) foundation.
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 (or isolators or bearings)	A horizontally flexible and vertically stiff structural component of the isolation system that permits large lateral deformations under earthquake shaking.
Moat (or isolation gap)	A space or gap around the perimeter of the isolated superstructure in which the superstructure can move without restriction.
Pedestal	The vertical support below the isolator units that allows for access to the isolators for inspection, maintenance, replacement, and other actions
Stop	A structure, or series of structures, designed to prevent excessive displacement of the isolation system. A moat wall could serve as the stop. A stop is sometimes called a “fail-safe” system in international guidance.
Superstructure	The superstructure is composed of all structural elements above the isolation system (e.g., slabs, beams, columns, walls). For a conventional LWR, the structural framing would include primary and secondary containment, internal structure to support the power generation and safety-related components and systems, and the basemat (or diaphragm) immediately above the isolation system.
Umbilicals	Umbilical lines are nonstructural components and systems (mainly distribution systems) that cross the isolation interface and must sustain the large isolator displacements (or deformations) associated with design basis and extended design basis ground motions. Examples of umbilical lines could include high-pressure steam lines from the power reactor to the turbines and cables located on trays or in ducts from emergency power systems located off the nuclear island to the power reactor.



a. Acceleration response b. Displacement response

Figure 3.2 Effect of Seismic Isolation on Spectral Demand (Courtesy of Dynamic Isolation Systems)

3.2 Seismic Design Practice

Historically, seismic design practice for fixed-base commercial buildings has been deterministic in nature with forces in the structural elements being calculated by the analysis of a linearly elastic model of a structure using either equivalent lateral forces or a response spectrum. The performance expectation for these buildings is a low probability of collapse for earthquake shaking with a return period of approximately 2,500 years. There is no associated performance check. ASCE 7-10 (ASCE, 2010) provides mandatory provisions for the seismic analysis and design of fixed-base (non-isolated) and isolated buildings.

Performance-based earthquake engineering for buildings evolved in the 1990s to enable engineers to explicitly design and detail structures for multiple performance levels. Nonlinear analysis (both static and dynamic) was codified to enable engineers to estimate displacements and drifts in buildings, which were then attached to descriptions of performance that ranged from “fully functional” to “heavily damaged but without collapse”. However, during that time the checking process remained deterministic and goals related to fully probabilistic design criteria could not be realized. ASCE 41-13 (ASCE, 2013) provides provisions and commentary on these first generation tools for performance-based earthquake engineering of buildings.

Second generation tools for performance-based earthquake engineering of buildings have been published recently. These tools enable the calculation of loss (direct, indirect and casualties) in a building for an intensity of earthquake shaking, an earthquake scenario, or on an annualized basis. The calculation process, which is described in Yang *et al.* (2009) and the *Guidelines for seismic performance assessment of buildings* FEMA (2013) involves the use of nonlinear response-history analysis, Monte Carlo analysis, fragility functions, damage states, and consequence functions. Huang *et al.* (2008, 2011a, 2011b) developed a companion process for safety-related nuclear structures.

Nuclear power plant structures have been designed traditionally to remain essentially elastic for design basis earthquake shaking but detailed to be ductile using the provisions of ACI 349 for shaking more intense than the design basis (i.e., beyond design basis earthquake shaking). This approach differs from the design of fixed-base (non-nuclear) buildings wherein substantial damage is expected for design basis shaking.

3.3 Types of Seismic Isolators Used for Base Isolation

Three types of seismic isolators are considered sufficiently well characterized for use in NPPs in the United States (US): 1) currently Low Damping Rubber (LDR) bearings, 2) Lead Rubber (LR) bearings, and 3) spherical sliding isolators, specifically, the Friction Pendulum™ (FP) family of bearings. These types of isolators have been subjected to dynamic component tests involving compressive/tensile axial loading and bi-directional shearing, as well as system-level tests on earthquake simulators involving three translational components of motion. These isolator types have been accepted by the US technical community and implemented in critical structures. Importantly, these types of bearings are considered to be analyzable, which is a prerequisite for use in US NPPs.

The rubber (LDR and LR) bearings utilize low damping natural rubber as the base compound. The LR bearing is a LDR with a central lead core. The FP bearing is a spherical sliding bearing that involves movement of articulated slider(s) coated in a high load, low friction composite material across a polished stainless steel surface(s). Each type of bearing has been widely used in the US and abroad for critical applications and has a proven track record. The mechanical properties of these three types of isolator are not expected to change over the lifetime of the nuclear facility. Chapter 4 describes these three types of bearings.

Chapter 4 also describes the high-damping rubber bearing, a fourth type that is not currently considered appropriate for use in nuclear facilities in the US because mechanical properties vary by compound, the compounds exhibit undesirable properties that make them *unanalyzable* (e.g., scragging and aging), and they cannot be modeled before prototype testing (e.g., Thompson *et al.*, 2000). Synthetic rubbers, such as neoprene, are also currently not used in US practice for any type of SI application for similar reasons. Improvements and standardization, including the use of non-proprietary compounds, may overcome the hurdles that preclude the use of high-damping and synthetic rubber bearings in US NPPs.

Other types of isolators may be acceptable for nuclear applications in the US at a later time. The following qualification tasks should be accomplished before a new type of bearing is used to isolate an NPP in the US.

1. Dynamic testing of full-scale (prototype) isolators for compressive and tensile axial loads and bidirectional horizontal motion at amplitudes of displacement expected for beyond design basis ground motions in regions of moderate and high seismic hazard;
2. Development of verified and validated numerical models capable of predicting the results of dynamic testing of prototype isolators, including deterioration of hysteresis due to energy dissipation during earthquakes;
3. Demonstration through basic chemistry, laboratory tests and field applications that the mechanical properties of the isolators do not change by more than 20% over the design lifetime in the temperature range of 40ºF to 80ºF;
4. System-level testing of the isolation system using three translational components of earthquake ground motion; and
5. Verification and validation of numerical tools and codes to predict the seismic response of the isolation system.

Tasks 1 through 5 should be performed by experienced personnel independent of the isolator manufacturer; the results shall be peer reviewed. Accelerated aging tests should not be used in Task 3 to characterize the evolution of the mechanical properties of elastomeric bearings (Constantinou *et al.*, 2007). Short-term testing of small rubber coupons cannot be used to predict the evolution of the mechanical properties of full-size elastomeric bearings cured under very different conditions. Isolation systems involving both elastomeric and spherical or flat sliding bearings should not be used. Ideally, new isolators should be implemented in mission-critical structures prior to application to NPPs. Chapter 9 addresses additional considerations for the use of seismic isolation, including quality control, quality assurance, and in-service inspection.

Figure 3.3 illustrates the assumed (shearing) force-lateral displacement relationship for a typical SI unit. The LR and FP bearings exhibit this force-displacement relationship (or hysteresis). The isolator is designed to have significant initial stiffness (K_u) to limit horizontal displacements induced by frequent loadings such as wind. If the displacement exceeds the yield capacity of the isolator (u_y), its horizontal stiffness decreases substantially (to K_d). This reduced stiffness provides the flexibility necessary to achieve seismic isolation during an earthquake. The characteristic strength of the isolator (or zero-displacement force intercept) is denoted Q_d; the yield strength of the isolator is denoted F_y. Lead rubber and FP bearings exhibit this force-displacement relationship (or hysteresis). The cyclic response of the LDR bearing is essentially linear over a wide range of shear strain (lateral displacement) in the elastomer. In some applications, particularly in Japan, the isolator is coupled with a separate external damping mechanism.

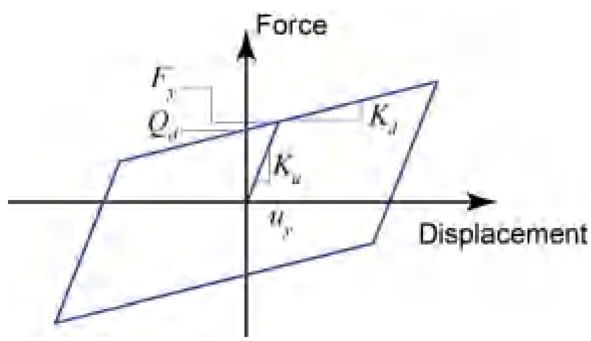


Figure 3.3 Force-Displacement Relationship for an Isolator

3.4 Construction of Seismic Isolators

As noted previously, the two main categories of seismic isolators currently considered appropriate for nuclear applications in the US are elastomeric (rubber) and sliding. The construction of each is briefly summarized below. Chapter 4 of this report discusses the properties of each in more detail. Both types of isolators are passive; that is, they do not have any internal sources of energy or controllers and control systems to actively modify their behavior. Actively controlled isolators (and supplemental damping systems) are not considered appropriate for safety-related nuclear structures because of concerns regarding long-term reliability, reliability in loss of onsite and offsite power conditions, need for maintenance, and need to replace controllers and computers. Importantly, isolators in nuclear structures must provide sufficient restoring force to re-center the supported structure after design basis shaking; the restoring force should be intrinsic to the isolator and not provided by external springs.

The first category of isolator is the laminated rubber (elastomeric) bearing. The laminated rubber bearing is composed of alternating layers of rubber and steel plates. The elastomeric material is typically natural rubber, which has low stiffness and high deformation capacity. The low stiffness and high deformation capacity of rubber enables the construction of seismic isolators. The horizontal stiffness depends primarily on the shear modulus of the rubber, the bonded area of the rubber (i.e., the area of rubber bonded to the steel plates), the total thickness of rubber, and the shear strain in the elastomer (which is related to the lateral displacement of the bearing).

The rubber is bonded to thin steel plates (or shims) using proprietary procedures, which typically involve the use of adhesives and the application of temperature and pressure. The steel shims do not affect the horizontal stiffness of the bearing, but prevent the elastomer from bulging when subjected to axial forces. The rubber provides the restoring force. The shape and geometry of the bearing, its aspect ratio (i.e., height-to-diameter), and imposed lateral displacement are important in determining the stability (or maximum vertical load) of the bearing. In low-damping and high-damping rubber bearings, energy dissipation is provided by the elastomer. In lead-rubber bearings, in which natural rubber forms the basis of the elastomer, energy dissipation is provided primarily by a cylindrical lead core or plug installed in the center of the bearing. The equivalent viscous damping in a low-damping elastomer typically ranges between 2% and 5% of critical. The equivalent viscous damping in a high-damping elastomer typically ranges between 8% and 13% of critical. The equivalent viscous damping in a lead-rubber bearing generally ranges between 15% and 30% of critical, depending upon the diameter of the lead core, the geometry of the bearings, and the imposed lateral displacement.

The second category of isolator is the spherical sliding bearing. The Friction Pendulum™ family of bearings is the only spherical sliding isolator considered appropriate for possible US NPP applications at this time. Sliding on a spherical contact surface enables the seismic isolation and provides a restoring force. Friction between the contact surfaces while sliding provides the energy dissipation. Vertical loads are transferred across the contact surfaces. A variant on the sliding bearing is the rolling bearing, which requires the construction and joining of two unidirectional rolling bearings to provide horizontal isolation in all horizontal directions. Springs and supplemental damping devices are required for both rolling bearings and flat sliding bearings to provide the requisite restoring force and energy dissipation.

3.5 Base Isolation of Nuclear Facilities

Base isolation offers significant potential benefits and challenges that applicants should factor into a decision whether to seismically isolate a nuclear facility. The potential benefits include:

- **Safety**: A properly engineered seismically isolated superstructure will, with high confidence, experience significantly smaller inertial forces than the comparable fixed-base superstructure. The significant reduction in inertial forces reduces earthquake demand on the primary and secondary structures, components and systems. Smaller forces on the structure enables improved safety (less damage in beyond design basis shaking) and a reduction in the size (and cost) of many internal structures. The reduction in horizontal inertial forces can substantially decrease the amplitude of the floor acceleration response spectra used for design and qualification of the components and systems. Taken together, the reduction in inertial forces and deformations in the isolated superstructure enabled by seismic isolation can substantially reduce the contribution of seismic hazard to the annual frequency of core damage and large radiation dose release, which increases safety. Huang *et al.* (2008) also observed that seismic isolation is an effective strategy to protect the internal systems and components from the effects of ground shock caused by near-by explosions.
- **Reliability**: Reliability is greatly enhanced by dynamic testing of prototype isolators to forces and displacements expected in beyond design basis earthquake shaking.
- **Economy**: Although seismic isolation adds a safety-related structural system between the substructure and superstructure, the cost premium could be offset by the savings realized through reductions in strength, size and complexity of the internal structure and substantial reductions in demand imposed on other structures, systems and components (SSCs). The savings would result from the smaller inertial forces and deformations associated with

isolation, noting that nearly all of the displacement of the isolated structure occurs in the isolation system. A significantly larger economy may be derived from broader use of certified designs or from simplified, standardized small modular reactor designs.

There are challenges associated with the base isolation of nuclear facilities, including:

- **Vertical acceleration:** Although modern isolation systems substantially decouple the superstructure from horizontal ground shaking, none mitigates response to vertical ground motion. Therefore, the superstructure itself and the internal systems and components attached to the superstructure will remain exposed to high vertical accelerations. This is sometimes addressed through vertical isolation of important equipment. Furthermore, if uplift is allowed in some forms of the FP system or substantial tension is allowed in elastomeric systems, re-engagement following uplift or tension may involve 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.* (2009a) discuss the effects of vertical acceleration on the response of seismically isolated structures.
- **Nonlinearity:** With the exception of LDR bearings, seismic isolators are nonlinear elements and require nonlinear response-history analysis to determine displacement demands for design basis and beyond design basis shaking. Linear elastic methods of analysis, including both time and frequency domain analysis approaches, are not appropriate for the final design of such isolation systems. Chapter 7 of this report discuss the issues of modeling and analysis in more detail.
- **Relative displacements across the isolation interface:** Any safety-related component or distribution system (e.g., pipes, cable trays, cable ducts and conduits) that crosses the isolation interface must be designed to accommodate the displacements expected in the isolators during design basis and beyond design basis ground motions.
- **Moat:** A moat (or seismic gap) must be provided to allow unobstructed movement of the isolated superstructure during earthquake shaking. The moat or seismic gap must be kept clear of any objects that may obstruct the movement of the isolated superstructure.

3.6 Floor and Equipment Isolation

Although base isolation is predominantly used in new construction, floor and equipment isolation can be used to improve seismic performance of both new and operating NPPs. An important design consideration for floor and equipment isolation is the coupling of horizontal and vertical movements. Vertical isolation can be used for both floors and equipment, but the potential for lateral deflection of vertical springs must be addressed through the use of a system to restrain or limit lateral movement. Rotational motion of floor and equipment isolation systems must also be addressed. Of significant concern is very tall equipment. The analysis of floor and equipment isolation systems should use in-structure response time series obtained from the analysis of appropriate structural models and input ground motions. Differential movement of SSCs across an isolation interface must be addressed.

4 MECHANICS OF SEISMIC ISOLATORS

4.1 Basic Mechanics of Seismic Isolators

This section introduces the basic mechanics of the two main categories of seismic isolators used for seismic isolation (SI), namely elastomeric and spherical sliding bearings. The remainder of the chapter provides information on the properties and performance of elastomeric and spherical sliding bearings.

4.1.1 Elastomeric Bearings

Elastomeric bearings typically use either natural or synthetic rubber[1] as their elastomeric material. Rubber has high elastic deformation capacity and very high elongation-at-break. It is also virtually incompressible. Detailed information on the mechanics of rubber bearings can be found in Kelly (1993), Naeim and Kelly (1999), and Constantinou *et al.* (2007), and is not repeated here.

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, damping, 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 or shims. These shims, which do not affect the horizontal stiffness of the bearing, substantially increase the vertical stiffness of the bearing by confining the rubber and preventing it from bulging. Insufficient vertical stiffness of isolators may result in rocking of an isolated superstructure during an earthquake. Figure 4.1 shows a typical cross-section through an elastomeric bearing.

The most important mechanical property of an elastomeric bearing (with no lead plug) is its effective (secant) horizontal stiffness, K_{eff}, which can be expressed as (e.g., Naeim and Kelly (1999)):

$$K_{eff} = \frac{GA}{T_r} \tag{4.1}$$

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 to 0.70 MPa (65 to 100 psi) at 100% shear strain[2] (Constantinou *et al.*, 2007). Only this type of rubber, which exhibits low damping, should be used at this time to construct elastomeric bearings for US NPPs.

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[1] Natural rubber is derived from latex and becomes the chemical called "isoprene" if purified. Synthetic or artificial rubber is produced by the polymerization of different monomers, including isoprene.
[2] Shear strain is calculated by dividing the isolator horizontal displacement by total thickness of rubber

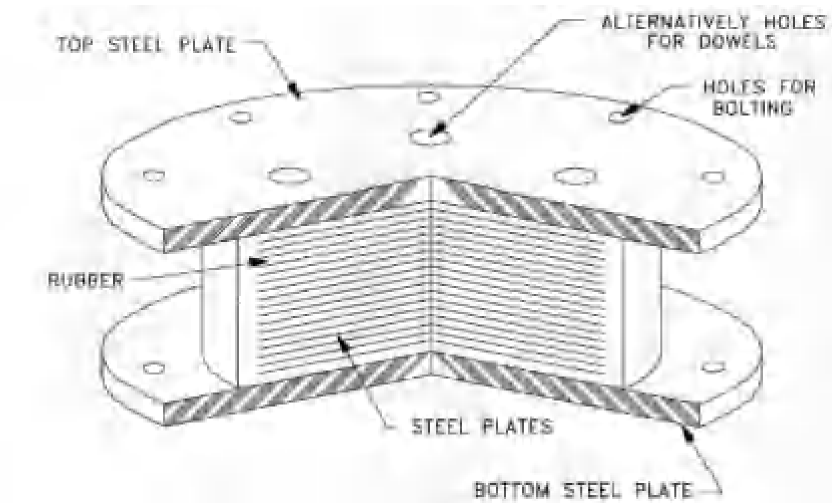


Figure 4.1 Cross-Section of a Typical Elastomeric Bearing (Constantinou *et al.* 2007)

The effective period of an isolation system composed on *n* identical isolators, T_b can be expressed as:

$$T_b = 2\pi\sqrt{\frac{W}{ngK_{eff}}} \tag{4.2}$$

where W is the total weight supported by the isolation system, g is the acceleration caused by gravity (9.81 m/s^2 or 32.2 ft/s^2), n is the number of (identical) isolators, K_{eff} is the effective horizontal stiffness of one isolator, and all other terms are defined previously. Because shear modulus depends on shear strain, the period of the isolation system depends on the lateral displacement of the isolation system. Changes in the vertical load (supported weight) due to vertical accelerations or rocking and/or overturning forces can also affect the response of the isolation system.

Shear strain, vertical stiffness, and isolator stability calculations make use of a variable known as the shape factor, S, which is calculated as

$$S = \frac{D}{4t_r} \tag{4.3}$$

where D is the bonded diameter, and t_r is the thickness of a single rubber layer. See Naeim and Kelly (1999) for details.

The shape factor for a seismic isolation bearing is typically greater than 10 and sometimes greater than 30. High shape factors help minimize shear stress caused by compression and also increase the buckling load capacity of the bearing (e.g., Constantinou *et al.* (2007)). Low shape factor bearings were studied by Tajirian *et al.* (1990) for nuclear applications for the purpose of attenuating the effects of vertical earthquake shaking. The vertical stiffness of a low shape factor bearing is less than that of a high shape factor bearing with an identical bonded diameter and total thickness of rubber. Low shape factor bearings creep more than high shape factor bearings and their use in building and nuclear structures was never pursued.

4.1.2 Spherical Sliding Bearings

Similar to an elastomeric bearing, the most important mechanical property of a spherical sliding bearing is its horizontal stiffness, K_{eff}. The *sliding* horizontal stiffness of a Friction Pendulum™ bearing, is:

$$K_{eff} = \frac{W}{R} \tag{4.4}$$

where W is the vertical load supported by the bearing and R is the effective radius of curvature of the bearing. (The sliding stiffness is different from the secant stiffness, which is used for equivalent linear calculations, as described later.) The sliding period of single concave Friction Pendulum™ isolation system, T_b, is

$$T_b = 2\pi\sqrt{\frac{R}{g}} \tag{4.5}$$

The sliding period depends solely on the effective radius of curvature of the sliding surface, which should be identical for all bearings in a FP isolation system.

4.2 Mechanical Properties of Seismic Isolators

Although there are many types of seismic isolators, only the LDR, LR and spherical sliding (FP) bearings are currently considered sufficiently well characterized for use in nuclear facilities in the US. This section provides information on the mechanical properties of these types of isolators. Information on high damping rubber bearings is provided for completeness.

The text of this section is based on multiple sources, including Zayas *et al.* (1987, 1990), Kelly (1993), Skinner *et al.* (1993), Naeim and Kelly (1999), Constantinou *et al.* (2007) and Fenz and Constantinou (2008a, 2008b). Constantinou *et al.* (2007) and Naeim and Kelly (1999) provide substantial information on the construction, analysis and design of elastomeric and sliding isolation systems.

4.2.1 Low Damping Bearings

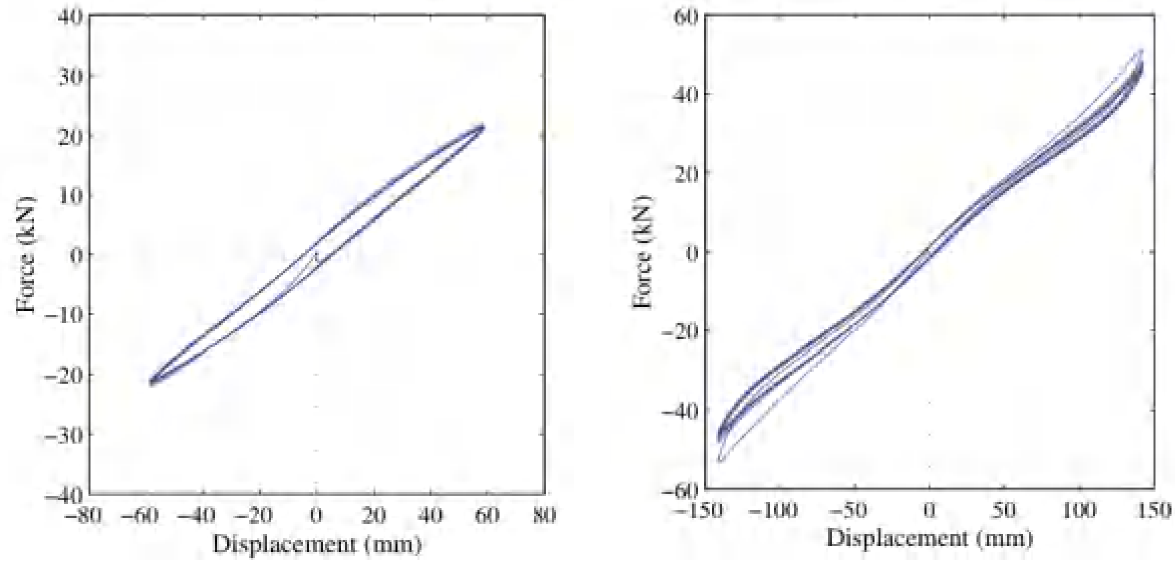
Low damping rubber (LDR) bearings use natural rubber as the elastomer.

4.2.1.1 Mechanical Properties of a Low Damping Rubber Bearing

The following three sections discuss the general behavior of LDR bearings when subject to shear, compression, and tension loads. The mechanical properties of a specific LDR depend on many factors, including the composition of the rubber, the conditions during vulcanization, and the geometry of the bearing. Therefore, the hysteretic behavior of a project-specific LDR bearing should be established by full-scale dynamic testing of prototype isolators.

4.2.1.1.1 Shear

Figure 4.2 shows the horizontal force-displacement relation for a LDR bearing at two levels of shear strain. This LDR bearing had a diameter of 250 mm, a shape factor of 9.8, an effective shear modulus of 0.65 MPa, and an effective damping ratio of less than 5%. The bearing was tested at a frequency of 1Hz, at an ambient temperature of 20ºC, with an axial load of 28kN. Figure 4.2(a) presents results of a test at a shear strain of 75%. Figure 4.2(b) presents results of a test at a larger shear strain: 175%. For both tests, the behavior is essentially linear elastic. However, at a strain of approximately 175% the horizontal stiffness of the bearing is observed to increase, although the increase is small. In general, low damping natural rubber stiffens at shear strains greater than 200% (Morgan *et al.*, 2001; Thompson *et al.*, 2000). At shear strains less than 200%, LDR bearings can be typically be modeled as linear elastic elements. The shear modulus of LDR ranges from 0.45 to 0.70 MPa (65 to 100 psi) at 100% shear strain.



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

Figure 4.2 Force-Displacement Relationship for a Low Damping Rubber Bearing (Constantinou *et al.*, 2007)

The area enclosed by the force-displacement relationship (hysteresis loop) can be used to approximate the damping ratio of the bearing. The hysteresis loops of Figure 4-2 enclose a small area, reflecting the fact that LDR bearings have little inherent damping, which is typically between 2% and 4% of critical (Kumar *et al.*, 2015d).

4.2.1.1.2 Compression

LDR bearings (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 (e.g., Naeim and Kelly (1999); Constantinou *et al.* (2007)).

4.2.1.1.3 *Tension*

For small tensile stresses, the axial stiffness in tension is similar to the stiffness in compression. As the tensile stress increases, the bearing begins to develop small cracks in the volume of the rubber, which is a process called cavitation. This typically occurs at tensile stress equal to about 3G, where G is the effective shear modulus of rubber, thus ranging from 1.5 to 2.5 MPa (0.20 to 0.35 ksi) depending on the composition of the rubber. Once cavitation occurs, confinement is lost and the tensile stiffness of the bearing decreases by several orders (Constantinou *et al.* (2007); Kumar *et al.* (2014, 2015d)). The substantial reduction of the vertical stiffness effectively allows for *uplift* of the elastomeric bearing. However, only bearings of high quality construction are capable of sustaining the resulting significant rubber extension without rupture.

Kumar *et al.* (2014, 2015d) performed experiments on elastomeric bearings, subjecting them to shearing, axial tension, and axial compression forces and displacements, for the purpose of characterizing behavior under extreme loadings. The cavitation pressure of 3G and the need for high quality construction were confirmed. Numerical models to describe behavior of LDR and LR bearings under extreme loadings were developed, verified and validated.

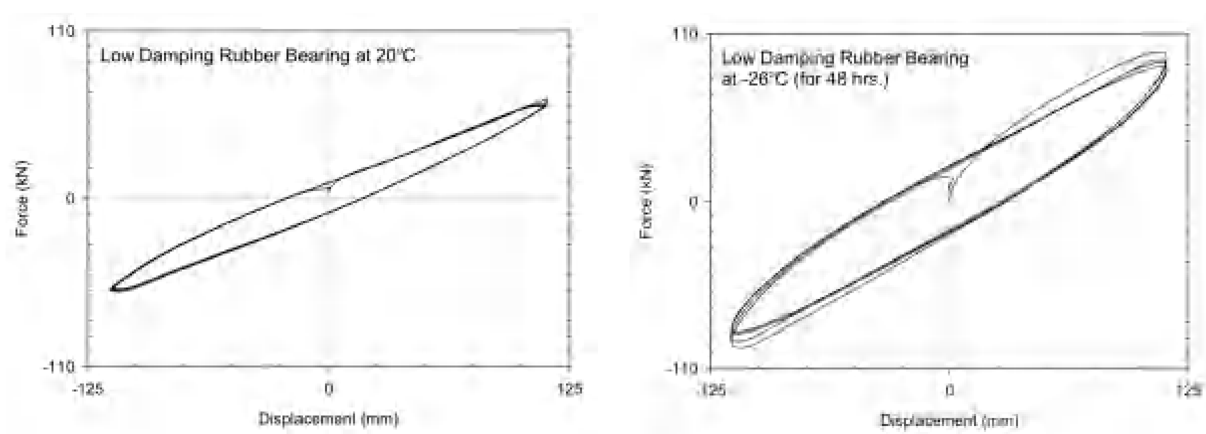
4.2.1.2 Variation in Mechanical Properties of LDR Bearings

The mechanical properties of LDR bearings can be affected by a number of variables. The following sections describe the effect of different variable on the mechanical properties of a LDR bearing. The effect of each variable should be established for the specific LDR bearing proposed for construction.

4.2.1.2.1 *Temperature*

Low temperatures substantially increase the stiffness, strength, and damping of LDR bearings. Figure 4.3 demonstrates this effect on a LDR bearing with a height of 411 mm, a width of 411 mm, and a shape factor of 10.7. The bearing was tested at a frequency of 0.35 Hz, with an axial pressure of 6.9 MPa, and a displacement amplitude of 113 mm. Figure 4.3(a) shows the force-deformation relation for an LDR bearing at 20ºC (68ºF), a typical operating temperature for bearings with interior exposure (i.e. inside a building or structure). Figure 4.3(b) shows the force-displacement relationship for an LDR bearing at –26ºC (–15ºF) after exposure for 48 hours. Both stiffness and damping increase by approximately 50% to 60% as a result of the drop in temperature. When the temperature falls below the glass transition temperature of natural rubber (approximately –55ºC or –67ºF), 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 LDR. As the duration of exposure increases, both stiffness and damping increase. This time-dependent stiffening is more pronounced for LDR 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 LDR, producing only minor reductions in both quantities. It is important to note that for most nuclear applications, the isolation system is likely to reside within a controlled environment and large variations in ambient temperature are not expected.



(a) Hysteresis at 20ºC (b) Hysteresis at –26ºC

Figure 4.3 Effect of Temperature on the Mechanical Properties of a Low Damping Rubber Bearing (Constantinou *et al.*, 2007)

4.2.1.2.2 Axial Load

The mechanical properties of a LDR bearing are sensitive to the axial load carried by the bearing. In general, as the axial force in the bearing increases, its horizontal stiffness decreases. The phenomenon is predictable by simple models of mechanics and is important when the axial load approaches the buckling load. This stiffness reduction is most pronounced in bearings with low shape factors because they have low axial load capacity (Ryan *et al.*, 2005).

4.2.1.2.3 Load History and Loading Rate

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 scragging is insignificant for LDR bearings fabricated with natural rubbers provided that the effective shear modulus of rubber is greater than about 0.45 MPa (65 psi).

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

4.2.1.2.4 Aging and Environmental Effects

Natural rubber can stiffen over time if 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 (Morgan *et al.*, 2001; Thompson *et al.*, 2000). If the natural rubber is fully cured during bearing fabrication, the age-related stiffening of the bearing will be minor.

The mechanical properties of natural rubber can degrade with prolonged exposure to oxygen and ozone. In bulk rubber (such as the rubber in a LDR bearing), only a thin 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).

4.2.2 Lead Rubber Bearings

A LR bearing is essentially a low damping rubber bearing with a vertical hole into which a lead plug is inserted; see Figure 4.4. This lead core, which has a diameter that is typically between 15% to 33% of the total bonded diameter, increases the damping (energy dissipation) of the bearing. The amount of energy dissipated by a LR bearing depends on the size of its lead core and the confinement of the core. LR bearings have been constructed with multiple lead cores but their behavior is currently not well understood. Only LR bearings with a single, central lead plug are considered sufficiently well characterized for use in US NPPs at this time.

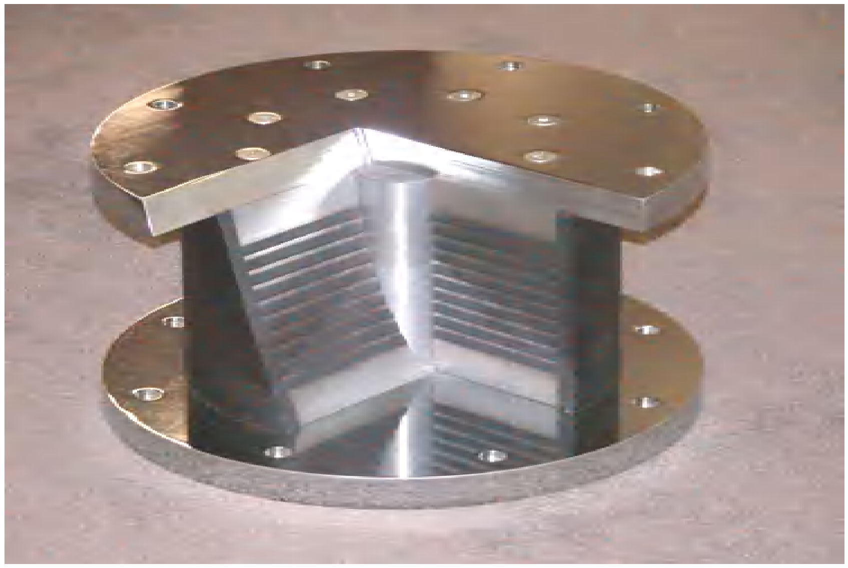


Figure 4.4 Cut-Away View of a Lead Rubber Bearing (Courtesy of Dynamic Isolation Systems, Inc.)

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 recover its original mechanical properties following inelastic action (Skinner *et al.*, 1993).

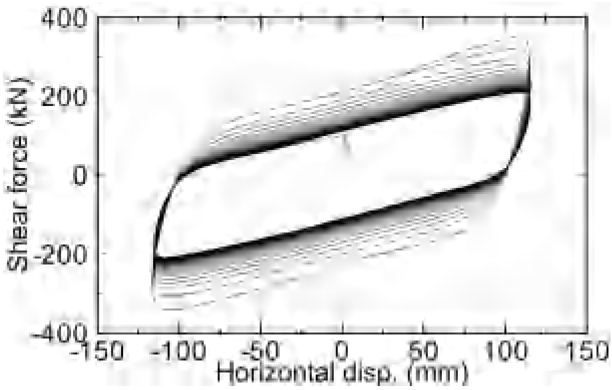
4.2.2.1 Mechanical Properties of LR Bearings

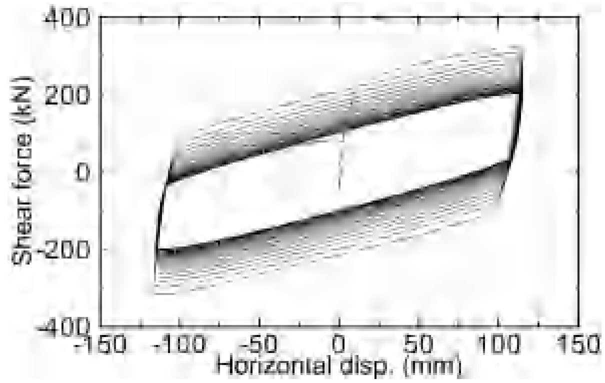
The following three sections discuss the general behavior of a LR bearing when subject to shear, compression, and tension loads. The mechanical properties of a specific LR bearing depend on many factors, including the composition of the rubber, the conditions during vulcanization, and the geometry of the bearing and lead plug. The behavior of a project-specific LR bearing should be established by full-scale dynamic testing of prototype isolators.

4.2.2.1.1 *Shear*

Figure 4.5 shows the horizontal force-displacement relationship for a LR bearing. This LR bearing has a diameter of 184 mm, and the diameter of the lead plug was 38 mm. The bearing was tested at a frequency of 0.5 Hz, to a shear strain of 250%, under an axial pressure of 6.9 MPa. Figure 4.5(a) shows the actual force-deformation relation of an LR bearing tested in the laboratory. Figure 4.5(b) shows the analytical force-displacement relationship when accounting for heating of the lead core (Kalpakidis *et al.*, 2010; Kumar *et al.*, 2014). In Figure 4.5(b), the characteristic strength (Q_d) is governed by the dynamic yield strength, size, and instantaneous temperature of the lead, and degree of confinement of the lead core. The post-yield stiffness K_d is a function of the shear modulus of the rubber (G), the bonded area (A), and the total thickness (T_r) of the rubber: see Equation 4.1. The maximum design shear strain of a LR bearing varies by manufacturer but generally ranges between 125% and 200% (Constantinou *et al.*, 2007).

The force-displacement relationship (hysteresis loop) for an LR bearing (Figure 4.5) encloses more area than a LDR bearing (Figure 4.2), reflecting the fact that the energy dissipation (damping) is much greater in a LR bearing than a LDR bearing.





(a) Measured response (b) Idealized response

Figure 4.5 Force-Displacement Relationship for a Large Lead Rubber Bearing (Kalpakidis *et al.*, 2010; Kumar *et al.*, 2014)

4.2.2.1.2 *Compression*

The lead core does not significantly affect the behavior of the LR bearing in compression. Accordingly, the behavior of the LR bearing in compression is similar to that of the LDR bearing.

4.2.2.1.3 *Tension*

The lead core does not significantly affect the behavior of the LR bearing in tension. Accordingly, the behavior of the LR bearing in tension is similar to that of the LDR bearing.

4.2.2.2 Variation in Mechanical Properties of LR Bearings

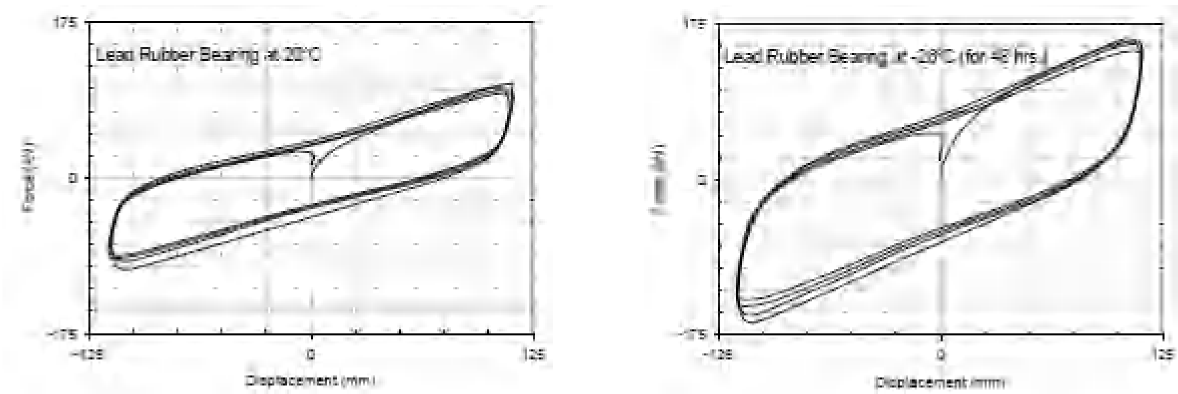
The mechanical properties of LR bearings can be affected by a number of variables. The following sections describe the general effect each variable has on the mechanical properties of a LR bearing. The effect of each variable should be established for the specific LR bearing used in design.

4.2.2.2.1 *Temperature*

As described in Section 4.2.1.2.1, changes in temperature significantly affect the mechanical properties of LDR. Because a LR bearing is essentially an LDR bearing with a lead core, the observations of Section 4.2.1.2.1 also apply to the rubber portion of a LR bearing.

Figure 4.6 shows the force-displacement relationship for a LR bearing tested at two different ambient temperatures. This bearing had a bonded width of 381 mm, a total rubber thickness of 196 mm, a shape factor of 10.7, and a lead plug with a diameter of 70 mm. The bearing was tested at a frequency of 0.35 Hz, at a displacement amplitude of 113 mm, under a bearing pressure of 6.7 MPa.

Low temperatures can substantially increase the stiffness, strength, and damping of LR bearings. The changes in behavior seen in Figure 4-6 are due primarily to changes in the shear modulus and damping of the elastomer and secondarily due to changes in the strength of lead. The changes in the properties of the elastomer can be seen in the loops of Figure 4-3.



(a) Hysteresis at 20°C (b) Hysteresis at –26°C

Figure 4.6 Effect of Temperature on the Mechanical Properties of a Lead Rubber Bearing (Constantinou *et al.*, 2007)

The mechanical properties of lead are affected by changes in temperature, particularly at high temperatures. As lead heats, its effective yield stress decreases, which reduces the amount of energy dissipated per cycle (Constantinou *et al.*, 2007). This is observed in the loops of Figure 4-5(a). In this case, the duration of testing is short so that conduction of heat through the steel end and shim plates is insignificant and all the generated heat increases the temperature of the lead plug. Accordingly, there is substantial reduction of strength in a few cycles of motion. Under these conditions, the rise in temperature of the lead is related to the distance travelled and inversely related to the height of the lead core but is not related to the diameter of the lead core (Kalpakidis and Constantinou, 2008, 2009).

If the bearing undergoes repeated cycles of motion, conduction of heat through the steel end and shim plates becomes important and the mechanical properties of the bearing stabilize. Kalpakidis *et al.* (2010) describes the process in detail. Figure 4.7 illustrates this effect. The bearing in Figure 4.7 was subjected to 25 cycles of motion at ±75% shear strain. The bonded diameter of the bearing was 483 mm and the diameter of lead plug was 140 mm. The bearing was tested at a frequency of 0.35 Hz and under a bearing pressure of 8.0 MPa. 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 after which both the yield strength of lead and the energy dissipated by the bearing in a cycle of loading do not change.

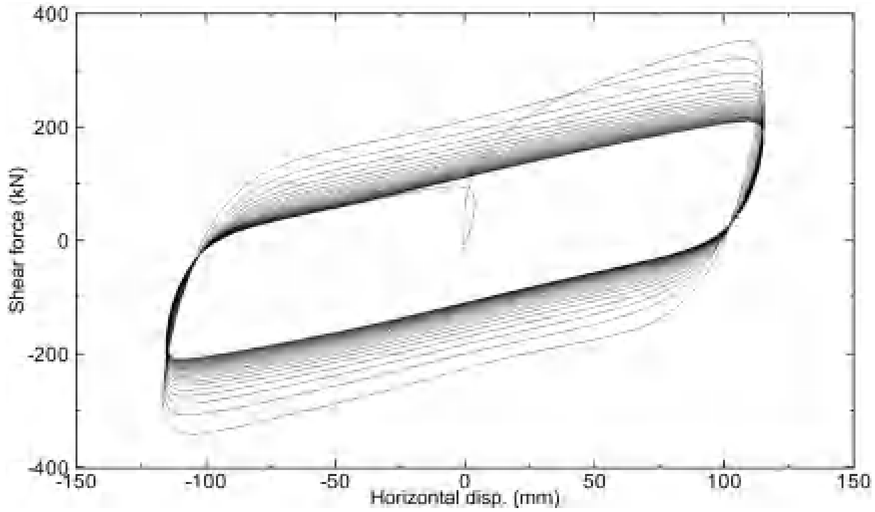


Figure 4.7 Force-Displacement Relationship for a Lead Rubber Bearing Subjected to 25 Cycles of Motion at ±75% Shear Strain (Reproduced from Kalpakidis et al. (2010))

4.2.2.2.2 Axial Load

Constantinou *et al.* (2007) report that compressive axial load does not affect significantly the mechanical properties of a LR bearing. An increase in axial compressive force will lead to a decrease in horizontal stiffness and an increase in damping, although the effects are generally minor unless the horizontal displacement of the bearing approaches its bonded diameter.

4.2.2.2.3 Load History and Loading Rate

The mechanical properties of a LR bearing depend on loading history. If the loading rate is high, the strength of the lead plug and energy dissipated per cycle both decrease as noted above. The greater the number of cycles of loading, the greater the reductions in strength and energy dissipated per cycle.

4.2.2.2.4 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 4.2.1.2.4 of this report.

4.2.3 High Damping Rubber Bearings

High damping rubber (HDR) bearings use natural rubber with additives (or fillers) as the elastomer. These additives modify the mechanical characteristics of the rubber, including its hardness, stiffness, damping, elongation-at-break, creep and relaxation properties. The most important consequence of adding these fillers is the increase in damping of the elastomer. HDR bearings have damping ratios greater than 7% of critical, with some compounds providing more than 13% damping.

As noted previously, HDR bearings are not considered appropriate at this time for use in US NPPs, due to scragging and unpredictable changes in mechanical properties over time, but are discussed here for completeness.

4.2.3.1 *Mechanical Properties of High Damping Rubber Bearings*

The following three sections discuss the general behavior of HDR bearings when subjected to shear, compression, and tension loads. The mechanical properties of a specific HDR bearing depend on many factors, including the composition of the rubber, the conditions during vulcanization, and the geometry of the bearing.

4.2.3.1.1 Shear

Figure 4.8 shows the force-displacement relationship for a HDR bearing. At small shear strains, the horizontal stiffness of the bearing is relatively high, which helps control wind-induced displacements. At moderate shear strains, the horizontal stiffness decreases, which provides the structure with the horizontal flexibility required during the design basis earthquake. At large strains (approximately 180%), the horizontal stiffness of the bearing increases, which could help limit maximum horizontal displacements. The maximum shear strain depends on the compounding of the rubber, but generally ranges from 200 to 350% (Constantinou *et al.*, 2007). The shear modulus of HDR typically ranges from 0.35 to 1.40 MPa (50 to 200 psi).

The larger area enclosed by the force-displacement relationship (hysteresis loop) in Figure 4.8, by comparison with that of Figure 4-2, shows the increase in damping with respect to the LDR bearing. Notice the large amount of scragging (degradation in response with repeated cycling to a given displacement or shear strain). Tests by Thompson *et al.* (2000) and Morgan *et al.* (2001) suggest that the amount of scragging increases as the volume fraction of fillers increase. Scragging effects are typically pronounced in HDR bearings.

4.2.3.1.2 Compression

The addition of fillers does not significantly affect the behavior of a HDR bearing in compression. Therefore, the behavior of a HDR bearing in compression is similar to a LDR bearing in compression. Refer to Section 4.2.1.1.2 for more information.

4.2.3.1.3 Tension

The addition of fillers does not significantly affect the behavior of a HDR bearing in tension and so its behavior is similar to that of a LDR bearing as described in Section 4.2.1.1.3.

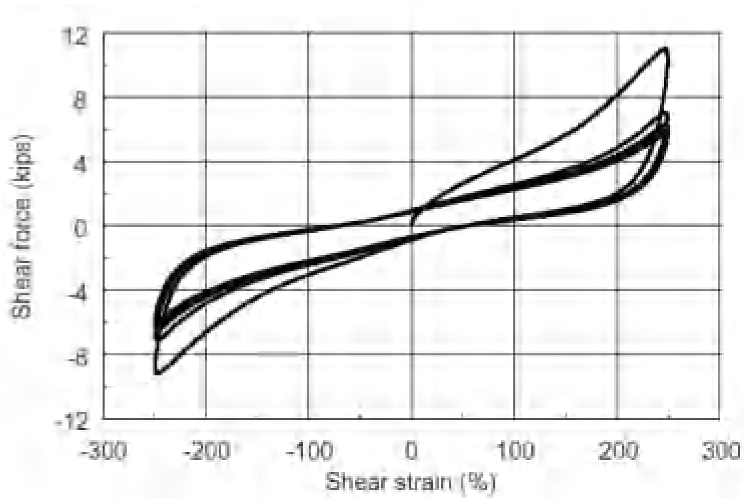


Figure 4.8 Force-Displacement Relationship for a High Damping Rubber Bearing (Thompson *et al.*, 2000)

4.2.3.2 *Variation in Mechanical Properties of HDR Bearings*

The mechanical properties of a HDR bearing are affected by many variables. The following sections describe the general effect each variable has on the mechanical properties of a HDR bearing.

4.2.3.2.1 *Temperature*

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

4.2.3.2.2 Axial Load

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

4.2.3.2.3 Load History and Loading Rate

The mechanical properties of a HDR bearing 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 (the reduction in stiffness after the first cycle is termed scragging). By the third cycle the response of a HDR bearing stabilizes (see Figure 4.8). The bearing will recover its virgin or unscragged properties over time (Constantinou *et al.*, 2007; Thompson *et al.*, 2000)

Tests performed by HITEC (1999), Thompson *et al.* (2000), Morgan *et al.* (2001), Constantinou *et al.* (2007) and others show that the mechanical properties of HDR are sensitive to changes in loading rate. In general, as the loading rate (or frequency of excitation) increases, both the horizontal stiffness and damping increase moderately.

4.2.3.2.4 Aging and Environmental Effects

The higher levels of damping in HDR are sometimes a result of the incomplete initial curing of the elastomer. Over time, the rubber will continue to cure, with the ongoing vulcanization of the rubber matrix occurring more rapidly in the first few years after the rubber is compounded, and slowing over time as the free sulfur is consumed (e.g., Thompson *et al.* (2000)). 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 bulk rubber such as the rubber in a HDR bearing, only a thin surface area is affected by exposure to chemicals in the environment, light and water. To prevent these effects, HDR bearings are normally fabricated with a cover layer of rubber containing various anti-ozonants and anti-oxidants with higher resistance to environmental effects (Constantinou *et al.*, 2007).

4.2.4 Spherical Sliding Bearings

The only type of spherical sliding bearing currently considered sufficiently well characterized for US NPP applications is the Friction Pendulum™ (FP) family of bearings and so the discussion below focuses on this type of sliding isolator. Figure 4.9 presents components of two FP bearing designs. Figure 4.9(a) shows a single concave FP bearing design. Its components include a housing plate, a concave dish with a thin 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 4.9(a), would be 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 sliding period of bearing (see Equation 4.5). The single concave FP bearing can be installed with the sliding surface facing up or down, depending upon the designer's decision to resist the earthquake-induced moment in the structural component below or above the isolator, respectively. Care must be taken in the construction of the FP bearing to ensure uniform support of the thin stainless steel inlay on the concave dish.

Figure 4.9(b) shows a triple concave FP bearing. 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 to achieve a user-specified hysteresis loop. The sliding period of the bearing depends on the lateral displacement and is calculated using the radii of the sliding surfaces of the inner and outer plates (Fenz and Constantinou, 2008a, 2008b). Information on the triple concave FP bearing is provided in Fenz and Constantinou (2008a, 2008b), Sarlis and Constantinou (2013), and Sarlis *et al.* (2013).

(a) Components of a single concave FP bearing

(b) Components of a Triple FP bearing

Figure 4.9 Friction Pendulum™ Bearings (Courtesy Earthquake Protection Systems, Inc.)

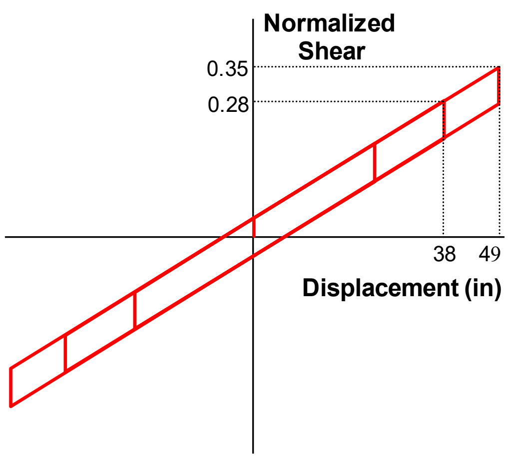
4.2.4.1 *Mechanical Properties of the FP Bearing*

The following three sections discuss the general behavior of the FP bearing when subject to shear, compression, and tension loads. The mechanical properties of a specific FP bearing depend on a number of factors, including the type of sliding surface and the geometry of the bearing. The hysteretic behavior of a FP bearing should be established by full-scale dynamic testing.

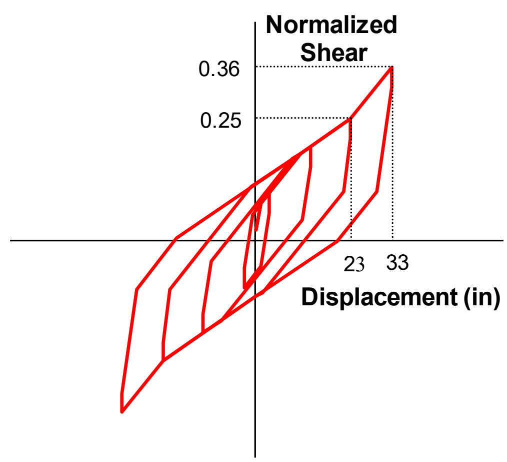
4.2.4.1.1 Shear

Figure 4.10(a) shows the force-deformation relation for a typical single concave FP bearing. Notice that the response is approximately bilinear and very similar to that of an LR bearing (see Figure 4.5). The initial high horizontal stiffness of the single concave FP bearing 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 4.4). The area enclosed by the hysteresis loop depends on the coefficient of sliding friction and contact pressure.

Figure 4.10(b) shows the force-deformation relation for a typical triple concave FP bearing. The response of a triple concave bearing is similar to that of an HDR bearing but the response of the triple concave bearing is repeatable and predictable. The piece-wise linear hysteresis is achieved by varying the radius and coefficient of sliding friction on the interfaces seen in Figure 4.9(b). At small displacements, the horizontal stiffness of the bearing is relatively high, which helps control wind-induced displacements. At moderate displacements, the horizontal stiffness decreases, which provides the structure with the horizontal flexibility required during the design basis earthquake. At large displacements, the horizontal stiffness of the bearing increases, which may help control displacements in beyond design basis shaking. Sarlis *et al.* (2013) report the results of earthquake-simulator testing of triple concave FP bearings.



(a) Single concave



(b) Triple concave

Figure 4.10 Hysteresis Loops for Friction Pendulum Bearings

4.2.4.1.2 Compression

The compression stiffness of a FP bearing is very high, reflecting the materials used for much of its construction, namely, carbon steel, stainless steel or ductile cast iron. Most FP bearings can be assumed to be rigid in the vertical direction.

4.2.4.1.3 *Tension*

FP bearings typically have no resistance to tensile loadings.

4.2.4.2 Variation in Mechanical Properties of a FP bearing

The mechanical properties of FP bearings can be affected by a number of variables. The following sections describe the general effect each variable has on the mechanical properties of the FP bearing. The effect of each variable should be established by dynamic testing for the specific FP bearing used in design.

4.2.4.2.1 *Temperature*

The mechanical properties of a FP bearing are somewhat sensitive to changes in ambient temperature. In general, as the ambient temperature increases, the coefficient of sliding friction decreases. Figure 4.11 supports this observation based on data for a particular interface consisting of a woven liner in contact with polished stainless steel. Low temperatures have a marked effect on the values of friction at low velocity. As the sliding velocity increases, frictional heating will increase the temperature of the sliding surface(s) and reduce the coefficient of sliding friction (Constantinou *et al.*, 2007).

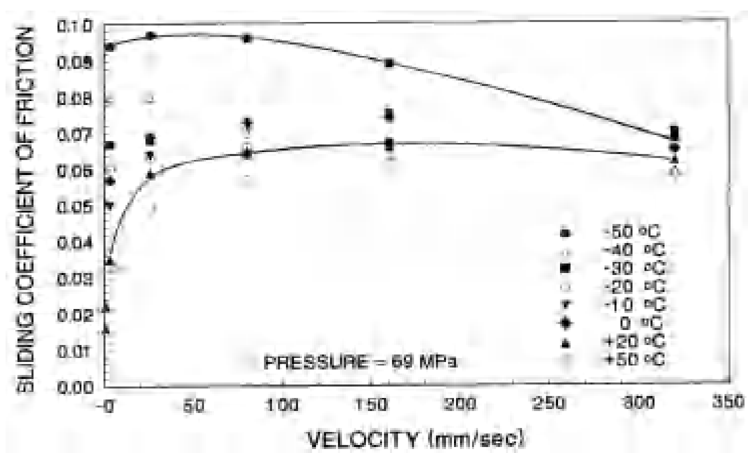


Figure 4.11 Effect of Ambient Temperature and Velocity on the Coefficient of Sliding Friction (Constantinou *et al.*, 2007)

4.2.4.2.2 Axial Load

The lateral force in a FP bearing is directly proportional to the axial load. Both the zero-displacement force intercept, or strength of the bearing, Q_d in Figure 3-3, and its post-yield stiffness, K_d in Figure 3-3, are proportional to the axial load. Another effect of axial load is that an increase in pressure on the articulated slider(s) will lead to a reduction in the coefficient of sliding friction (Constantinou *et al.*, 2007). Figure 4.12 shows the relationship between apparent (contact) pressure and the inverse of the coefficient of sliding friction. The data in Figure 4.12 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 FP bearing).

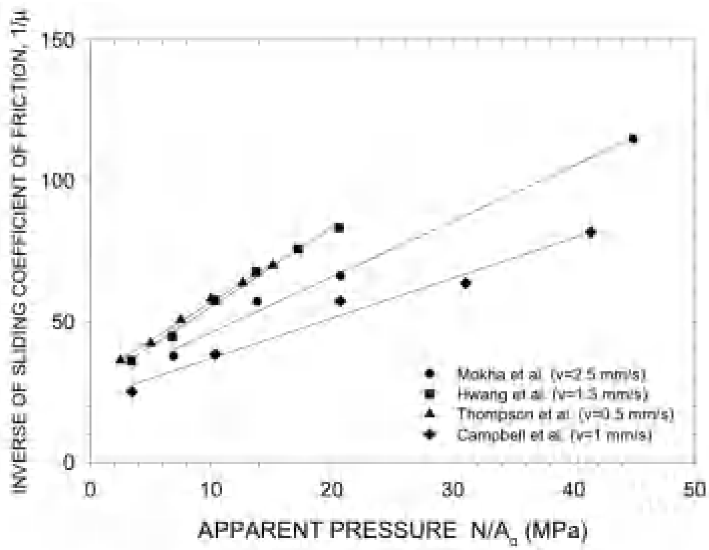


Figure 4.12 Effect of Apparent Pressure on the Coefficient of Sliding Friction (Constantinou *et al.*, 2007)

4.2.4.2.3 Load History and Loading Rate

The mechanical properties of a FP bearing depend somewhat on its load history. The passage of the articulated slider(s) over the sliding surfaces deposits a very thin layer of composite material on the sliding surface that will reduce the coefficient of sliding friction by a small percentage. Data on wear of FP bearings are limited to a small number of laboratory tests. Constantinou *et al.* (2007) note that wear of the composite liners in two tests of FP bearings was insignificant. In the first test, after 10,000 cycles and 1520 meters of travel, the liner lost 2% of its thickness. In the second test, after 20,000 cycles and 3240 meters of travel, the liner lost 20% of thickness. Note that these tests had a focus on bridge applications for which isolator movement due to thermal cycling and live loading are significant. Isolators for nuclear facility applications will not experience such daily movements.

The mechanical properties of the FP bearing are sensitive to loading rate. In general, as the loading rate (velocity) increases from very slow to fast, the coefficient of friction increases to an asymptotic value at a velocity of approximately 100 mm/sec. Figure 4.11 demonstrates this effect (specifically the curve corresponding to 20ºC). At very high velocities (500 mm/sec or greater), there is substantial frictional heating, which may reduce the thickness of the composite surface and will reduce the coefficient of sliding friction (Constantinou *et al.*, 2007). Heating of the sliding interface will reduce the coefficient of sliding friction, with an effect on isolator hysteresis similar to that observed in the LR bearing when the temperature of the lead core increases.

4.2.4.2.4 Aging and Environmental Effects

Identical to elastomeric bearings, sliding bearings in NPPs will be installed in a dry, air-conditioned space, with limited exposure to environmental effects. Regardless, the mechanical properties of the FP bearing are generally not sensitive to environmental effects. The materials used in the construction of the FP bearing are not susceptible to aging.

The outer surfaces of the FP bearing are typically coated in an epoxy-based paint to protect against environmental effects. A perimeter elastomeric liner is used to prevent the ingress of contaminants into the isolator internals. Corrosion of the stainless steel sliding surfaces is highly unlikely, especially given the installation of the protective liner, the corrosion properties of the stainless steels used (Constantinou *et al.*, 2007), and the installation of the isolation system in a dry, air-conditioned space.

5 GUIDANCE ON ANALYSIS AND DESIGN OF ISOLATION SYSTEMS IN US CODES AND STANDARDS

This chapter discusses the current state of practice for the analysis and design of seismically isolated structures in the United States (US). Section 5.1 discusses deterministic and probabilistic design procedures. Current design standards for isolation of non-nuclear structures are reviewed in Section 5.2. Section 5.3 discusses design standards for safety-related nuclear structures.

5.1 Approaches to Analysis and Design

Two types of procedures can be used to analyze and design a structure: 1) deterministic, and 2) probabilistic. Most current structural codes and standards are deterministic, though recently there has been increased interest in developing probabilistic design procedures. The NRC adopts elements of a risk-informed, performance-based approach based on ASCE 43-05 (ASCE, 2005) as described in RG 1.208 (NRC, 2007a). The next two subsections discuss each type of design procedure.

5.1.1 Deterministic

Deterministic seismic design procedures involve the calculation of demands on structural components and non-structural components for a chosen intensity of shaking, and the checking of each component to ensure that demand is less than capacity (i.e., demand-capacity ratio of less than 1.0). The intensity of the shaking can be represented by horizontal and vertical response spectra or 3-component sets of ground motions. Analysis of the structure is performed using the chosen representation of shaking to compute demands, in terms of forces and/or deformations (displacements). Randomness in ground motion can be considered (Jayaram *et al.*, 2011). Uncertainty in capacities can be addressed using resistance factors. A probability of unacceptable performance for a chosen intensity of shaking can be calculated using the distributions of demand and capacity.

5.1.2 Probabilistic

Probabilistic seismic design procedures utilize both a frequency of earthquake shaking and a frequency of unacceptable performance.

Instead of using demand-capacity ratios as noted above, the probabilistic procedures of ASCE 43-05 use 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 annual frequency of exceeding a designated hazard level and P_F is the annual frequency of unacceptable performance (i.e., failure) for the structure or element. Typically, R_R is between 1 and 2 for conventional buildings, and R_R is substantially greater than 1 for essential or critical structures, including nuclear facilities. An acceptable design in the probabilistic space is one whose structural and non-structural components have risk reduction factors greater than or equal to their target values.

5.2 Summary of Design Standards for Non-Nuclear, Seismically Isolated Structures

The following subsections summarize the design standards and codes that contain provisions for non-nuclear, seismically isolated structures. Note that most of the guidance for non-nuclear structures is based on deterministic design procedures.

5.2.1 ASCE 7-10

Chapter 17 of ASCE 7-10 (ASCE, 2010), *Minimum design loads for buildings and other structures*, contains provisions addressing the use of seismic isolation (SI) in non-nuclear 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 (non-isolated), fixed-base structures. These requirements alone are not sufficient to meet NRC's objectives.

Rather than addressing a specific method of SI, 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 for design. Use of isolators 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 substantially under repeated cyclic load, and (4) have quantifiable engineering parameters (such as force-displacement characteristics and damping).

The following subsections summarize these provisions. For more detailed information, refer to Chapter 17 of ASCE 7-10.

5.2.1.1 Definitions

Chapter 17 of ASCE 7-10 defines four different isolator displacements[1], each of which is used for different parts of the design process.

> **Design displacement:** The design earthquake lateral displacement, excluding additional displacement due to natural and accidental torsion, required for design of the isolation system and the supported structure.
>
> **Total design displacement:** The design earthquake lateral displacement, including additional displacement due to natural 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 natural and accidental torsion.
>
> **Total maximum displacement:** The maximum considered earthquake (MCE) lateral displacement, including additional displacement due to natural 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.

[1] These definitions differ from the definitions provided in Chapter 8 because different approaches are used for analysis and design.

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 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 (in ASCE 7-10).
>
> **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 7-10 are expected:

a. To resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components, or building contents; and

b. 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, non-nuclear isolated structures are expected to perform much better than non-nuclear, fixed-base structures during moderate and major earthquakes[2]. 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, refer to Section 17.2 of ASCE 7-10.

ASCE 7-10 requires that the isolation system:

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

[2] These overall performance goals for seismically isolated non-nuclear structures are not sufficient to meet NRC's performance objectives.

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 accommodate the total maximum displacement. Minimum separations between the isolated structure and surrounding retaining walls or other fixed obstructions should be greater than the total maximum displacement to allow for unobstructed movement of the isolated structure.

5.2.1.3 Hazard Characterization

ASCE 7-10 specifies MCE ground motion as a risk-adjusted, maximum direction spectrum. It is derived from a Uniform Hazard Response Spectrum with a 2% probability of exceedance in 50 years, which corresponds to a return period of approximately 2,500 years. In the near-fault region, the MCE spectral ordinates are capped by a deterministic limit, which effectively reduces the return period of the MCE shaking. Non-isolated buildings designed per ASCE 7-10 are assumed to have a 10% probability of collapse given MCE shaking. The risk target for a non-isolated building is a 1% probability of failure in 50 years. The design earthquake ground motion spectrum is taken as two thirds of the MCE ground motion spectrum.[3]

Ground motion is characterized in ASCE 7-10 using response spectra for analysis and design of typical buildings. Pairs of appropriately selected and scaled horizontal ground motion are used for response-history analysis. Appropriate ground motions are selected from events having magnitudes, fault distance, and source mechanisms that are consistent with those that control the MCE ground motion.

5.2.1.4 Analysis Procedure Selection

Chapter 17 of ASCE 7-10 specifies two different types of analysis procedures for seismically isolated structures: equivalent lateral force and dynamic. The equivalent lateral force procedure can only be used for final design if specific, restrictive criteria are satisfied. Otherwise, response-spectrum or response-history dynamic analysis, is required. Response-spectrum analysis procedures are permitted only if certain criteria are met. Nonlinear isolation systems, such as those constructed with lead-rubber (LR) or spherical sliding (FP) bearings, are routinely analyzed by response-history procedures.

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 and minimum lateral forces to be used for the design of seismically isolated structures. Minimum values of lateral displacement are computed using the minimum effective stiffness of the isolation system as established by testing. Minimum values of lateral force are computed using the maximum effective stiffness of the isolation system for design basis

[3] This design ground motion is far more frequent than the ground motion response spectrum developed for nuclear power plants as described in Regulatory Guide 1.208 (NRC, 2007a), which has an unreduced annual frequency of exceedance of 1×10^{-4} multiplied by a design factor that is always equal to or greater than 1 at every spectral frequency.

shaking. The base shear for the design of the structure above the isolation interface can be reduced from that delivered by the isolation system by factor of between 1 and 2. The structure below the plane of the isolation system is designed for the base shear delivered by the isolation The maximum drift in any story 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 provides requirements for dynamic analysis of seismically isolated structures. The requirements are less prescriptive than those of the equivalent lateral force procedure. Section 17.6 requires that the isolation system be modeled using deformational characteristics developed and verified by testing. The isolation system should be modeled with sufficient detail to (ASCE, 2010):

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 (ASCE, 2010):

a. Stiffness properties assumed for the nonlinear components of the isolation system are based on the maximum effective stiffness of the isolation system; and

b. All elements of the seismic force-resisting system of the structure above the isolation system remain elastic for the design earthquake ground motion.

If a response-spectrum analysis is permitted for the superstructure, the total design displacement and total maximum displacement should include simultaneous excitation of the model by 100 percent of the ground motion in the one horizontal direction and 30 percent of the ground motion in the perpendicular, horizontal direction. The maximum displacement of the isolation system is calculated as the vector sum of the two orthogonal displacements.

Where a response-history analysis is performed, a suite of no less than three appropriate sets of ground motions is used for analysis. Each pair of ground-motion components is applied simultaneously to the model considering the most disadvantageous location of eccentric mass. The maximum displacement of the isolation system is calculated from the vector sum of the two orthogonal horizontal displacements at each time step. If at least seven pairs of ground motions are used for analysis, the mean maximum value of the response parameter of interest can be used for design. If fewer than seven ground motions are used, the peak maximum value of the response parameter of interest must be used.

The lateral displacements and forces obtained from either response-spectrum or response-history analysis cannot be less than a specified fraction of the values calculated using the equivalent lateral force procedure. These requirements date to the early provisions for seismic isolation design when dynamic analysis was rarely performed and the technical community had little confidence in the results of dynamic analysis.

5.2.1.7 Design Review

Section 17.7 of ASCE 7-10 requires that an independent engineering team review the analysis and 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 isolators. 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. The sequence and number of cycles for each test are specified.

The deformation characteristics and damping values of the isolation system can be determined from data generated from prototype tests. Section 17.8.5 of ASCE 7-10 provides equations for computing the minimum and maximum effective stiffness and the effective damping of the isolation system.

5.2.2 AASHTO Guide Specification for Seismic Isolation Design

The American Association of State Highway and Transportation Officials published the *Guide specifications for seismic isolation design* in 2010 (AASHTO, 2010). The specification is a significant update to its predecessor, which was published in 1999. The isolation system in a bridge is typically installed just below the deck, at the top of the bridge piers and the abutments. The isolators limit the force that can be transferred to the substructures, enabling the piers, abutments and their foundations to remain essentially elastic during design basis shaking. In contrast, a non-isolated bridge is expected to undergo inelastic deformations and suffer damage during design earthquake shaking.

Many of the provisions in the AASHTO guide specifications are similar to those in Chapter 17 of ASCE 7-10. One important difference is the AASHTO use of system property modification factors to account for the effects of temperature, aging, scragging, velocity, cumulative travel, and contamination on the response of the isolated structure. This approach is based on the work of Constantinou *et al.* (1999), Constantinou *et al.* (2007), Thompson *et al.* (2000) and Warn and Whittaker (2006b), among others, and is now acknowledged in Section 17.1.1 of ASCE 7-10.

The AASHTO guide specifications require prototype-bearing tests that are specific to bridges, namely, to demonstrate that the bearing can sustain thermal and traffic induced movements over the design life of the bridge. Although such tests are not relevant to isolators proposed for NPPs, similar tests might be appropriate for isolation systems deployed to protect components and systems in CIS, where thermal and vibration-induced movements may be experienced.

5.3 <u>Summary of Design Standards Applicable to Isolated, Safety-Related Nuclear Structures</u>

The following subsections summarize the design standards and codes that contain provisions for safety-related nuclear structures. The Nuclear Regulatory Commission (NRC) utilizes these standards, with modifications as needed that are documented in the Standard Review Plan and Regulatory Guides. It is assumed in this report that the seismic isolation system and isolator units[4] will be treated as structural components. A seismically isolated nuclear facility structure has three elements (see Figure 3-1): the superstructure, the isolation system, and the foundation embedded into the underlying soil. Many of the design and quality assurance criteria of Chapter 9 are drawn in part from ASCE 7-10 and ASCE 43-05 (ASCE, 2005)[5], although the performance expectations of ASCE 7-10 (buildings) and ASCE 43-05 (primarily informed by DOE Standard 1020) are different from those of the NRC.

5.3.1 ASCE 43-05

ASCE 43-05 is a consensus standard that contains more stringent seismic design criteria than standards for conventional buildings and bridges. It provides the basis for risk-informed, performance-based seismic design of a range of safety-related nuclear structures, and is not limited to NPPs. ASCE 43-05 is intended for use with ASCE 4-16 (ASCE, 2017), ACI 349 (ACI, 2013), ANSI/AISC N690 (AISC, 2012), ASCE 7-10 (ASCE, 2010), and other standards and codes. ASCE 43-05 contains no specific provisions for seismically isolated structures.

ASCE 43-05 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* (ANS, 2010), to define five Seismic Design Categories (SDCs) and four Limit States. SDCs range from 1 to 5. SDC 1 corresponds to non-nuclear structures; SDC 5 corresponds to hazardous or critical structures such as NPPs. Limit States range from A to D. Structures assigned to Limit State A are expected to experience large permanent distortion and damage when subjected to design earthquake shaking, whereas structures assigned to Limit State D are expected to remain essentially elastic for design earthquake shaking. The provisions in ASCE 43-05 address the four limit states but apply them only to SDCs 3, 4, and 5, because these are associated with nuclear facility structures, systems and components. See Table 1-1 of ASCE 43-05 for details.

The SDC is used to establish the hazard exceedance frequency for the design basis earthquake (DBE) ground motion. The exceedance frequency decreases with increasing SDC, meaning that a more critical structure (e.g. NPPs in SDC 5) is designed for a greater intensity of shaking than a less critical structure (e.g., SDC 3). The limit state is used to establish acceptance criteria for the components of the structure.

ASCE 43-05 defines the Design Basis Earthquake (DBE) spectrum as the product of a Design Factor and a Uniform (or Equal) Hazard Response Spectrum (UHRS) developed for the site using probabilistic seismic hazard analysis. For SDC 5 (i.e., NPPs), the UHRS has an annual frequency of exceedance of 10^{-4}. The Design Factor is calibrated to achieve a maximum annual frequency of unacceptable performance (first onset of significant inelastic deformation at the component level) of 10^{-5} in non-isolated structures, accounting for the conservatisms inherent in the design process and capacities made possible through the use of ductile detailing. The provisions of ASCE 43-05 form the technical basis for the development of the Ground Motion Response Spectrum (GMRS) in USNRC Regulatory Guide 1.208 (NRC, 2007a).

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[4] The isolator units should also be treated as Seismic Category I components, although their behavior is required to be non-linear.

[5] As described in Section 9.2.1, quality control procedures in 10 CFR 50 Appendix B also apply to isolator units.

As stated in Section 1.3 of ASCE 43-05, 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 (as defined in Section 2.0 of ASCE 43-05).
2. Less than 10% probability of unacceptable performance for 150% of the DBE ground motion (as defined in Section 2.0 of ASCE 43-05).

The ASCE 43-05 methodology simultaneously achieves these two objectives that are in turn expected to achieve the target performance goal, which is expressed as an annual frequency of non-exceedance. Design for DBE shaking as defined above, using the procedures of ASCE 4-16 (ASCE, 2017) and materials standards such as ACI 349 (ACI, 2013) is expected to achieve both performance objectives and the target performance goal. The performance goals, P_F, hazard exceedance probabilities, H_D, and the probability ratios, R_P, for SDC 3, 4 and 5 are presented in Table 5-1 below.

Table 5-1 Target Probability Goals in ASCE 43-05 for Nuclear Structures

	Seismic design category		
	SDC 3	**SDC 4**	**SDC 5**
P_F	10×10^{-5}	4×10^{-5}	1×10^{-5}
H_D	40×10^{-5}	40×10^{-5}	10×10^{-5}
$R_P = H_D/P_{F*}$	4	10	10

5.3.2 ASCE 4-16

Standard ASCE/SEI 4-16 provides detailed guidance on seismic isolation of nuclear facilities. Chapter 12 of the Standard is devoted to seismic isolation. Mandatory language and commentary are provided to enable analysis, design, review and performance assessment of isolated nuclear structures, where the focus is horizontal isolation of large nuclear facilities such as power plants, containment vessels, turbine buildings, and emergency response centers. Topics addressed in the chapter include a) general requirements, b) characterization of seismic inputs, c) methods of dynamic analysis, d) calculation of displacements and forces for design, e) peer review, and f) requirements for testing of prototype and production isolators.

The scope of the chapter is broader than seismic analysis of isolated nuclear power plants. The text on design, construction and testing of isolators in the chapter will be moved to Chapter 10 of ASCE 43-05, which is currently under revision, and then removed from ASCE 4 in the revision following ASCE 4-16.

The mandatory language presented in Chapter 12 of ASCE 4-16 is very similar to the performance criteria recommended in this report. The major difference between ASCE 4-16 and this report, which is specific to nuclear power plants, is the definition of the beyond design basis earthquake, which is relevant to the calculation of seismic risk, which is calculated at the component level for DOE facilities and at the plant level for NRC-regulated NPPs. In ASCE 4-16, the intensity of the beyond design basis earthquake is set at 150% of the design basis earthquake, and the target performance goal for an SDC 5 facility (see Section 5.3.1) is a mean annual frequency of unacceptable performance of 1×10^{-5}. As described in Chapter 8, the proposed mean annual frequency of unacceptable performance for an isolated NPP is an order of magnitude smaller, requiring a different definition of the beyond design basis earthquake.

5.3.3 NRC RG 1.208

USNRC Regulatory Guide (RG) 1.208, *A performance-based approach to define the site-specific earthquake ground motion* (NRC, 2007a), describes the current NRC approach for developing a site-specific ground motion response spectrum. The ground motion response spectrum is used as the design basis for new nuclear power plants. RG 1.208 is based in part on provisions in ASCE 43-05.

5.3.4 Concrete Structures

This report assumes that the design of reinforced concrete structures and foundations would follow the provisions of ACI 349 (ACI, 2013) for nuclear safety-related concrete structures other than containments, and ASME Boiler and Pressure Vessel Code Section III, Division 2 (also ACI 359) (ASME, 2001b) for concrete containments, with the exceptions and additions in RG 1.142 (NRC, 2001) and RG 1.136 (NRC, 2007b).

5.3.5 Steel Structures

This report assumes that the design of safety-related steel structures other than containments would follow the guidance in Sections 3.8.3 and 3.8.4 of the Standard Review Plan (NUREG 0800) (NRC, 2013). It also assumes that the design of steel containments would follow the provisions in ASME Boiler and Pressure Vessel Code Section III, Division 1 (ASME, 2001a) with the exceptions and additions in RG 1.57 (NRC, 2013).

6 INTERNATIONAL GUIDANCE FOR SEISMIC ISOLATION OF NUCLEAR FACILITIES

6.1 Regulatory Guidance for Seismic Isolation of Nuclear Facilities in Japan

As discussed in Sections 2.1.2 and 2.3.4, seismic isolation (SI) is widely used in Japan and has recently been used in the construction of new emergency response center buildings at sites of Japanese NPPs. In 2006, the Nuclear Safety Commission of Japan revised the regulatory guidance for seismic design of Japanese NPPs. This revision removed the specification stating that buildings are to remain essentially rigid and explicitly allowed for the use of seismic isolation. As a result, the former Japan Nuclear Energy Safety (JNES) organization, now a part of the Nuclear Regulatory Authority of Japan (NRAJ), initiated a program to develop new regulatory guidance for the design of isolated nuclear power plants (NPPs) in Japan.

Japan has a long history of research and development focused on seismic isolation technology for NPPs. The Japan Atomic Energy Research Institute (JAERI) conducted studies on equipment base isolation systems from 1987 to 2000. The Nuclear Power Engineering Corporation (NUPEC), and later JNES, continued JAERI's work from 2001. NUPEC also studied floor isolation. The University of Tokyo's Institute of Industrial Science studied equipment isolation. The Central Research Institute of the Electric Power Industry (CRIEPI) continues to study fast breeder reactor building isolation. The Japan Electric Association (JEA) has formulated design guidelines for seismically isolated buildings (see Section 6.1.2).

6.1.1 JNES-RC-2013-1002

Under a bi-lateral cooperative research agreement, the NRC and JNES exchanged technical information related to the seismic safety of nuclear power plants, which included information related to seismic isolation. Through this agreement, the NRC was provided with an English translation of draft JNES guidance on seismic isolation entitled *Technical review guidelines for structures with seismic isolation: JNES-SS-1101* (JNES (2010) in Japanese and JNES (2011) in English). The final version of the guidance was published in January 2014, *Proposal of technical review guidelines for structures with seismic isolation: JNES-RC-2013-1002* (JNES, 2014). Report JNES-SS-1101 was the first regulatory guidance developed by JNES for the seismic isolation of NPPs.

The JNES drew on experience from other non-nuclear organizations that research and publish guidance on the use of SI, such as the Architectural Institute of Japan (which formulated SI guidelines in 2000), the Japan Society of Civil Engineers (which formulated SI guidelines in 2000), and the Japan Society of Seismic Isolation (which formulated guidelines in 2006). This chapter discusses the similarities and differences between the JNES 2014 guidance and this NUREG/CR.

JNES-RC-2013-1002 discusses regulatory positions and their technical basis and identifies relevant regulatory review elements. The difference in purpose and content between JNES-RC-2013-1002 and this report challenge a direct comparison. At a high level, there are several key differences between the two approaches that are discussed below. Key differences are summarized in Table 6-1.

This report describes three types of SI systems that its authors consider currently acceptable for possible use in US NPPs and two that are not. The JNES guidance does not specify the types of SI systems considered. Additionally, JNES-RC-2013-1002 discusses the design of vertical isolation systems and the isolation of floors and equipment, which are not addressed in detail in this report. The use of equipment and floor isolation can be used in both new and operating reactors, and is discussed as an option for operating NPPs for cases where the seismic hazard has increased beyond the NPP's original design basis.

JNES-RC-2013-1002 is generally based on deterministic approaches, whereas this report is based on risk-informed, probabilistic approaches. The determination of input ground motions for new US NPPs is based on the probabilistic techniques in Regulatory Guide 1.208 (NRC, 2007a). The design ground motion for Japanese NPPs (NSC, 2006) is determined using a deterministic framework. The design of the isolation system per JNES-RC-2013-1002 can be performed using either a margins approach or seismic probabilistic risk assessment (SPRA)[1] techniques. In contrast, this report provides performance-based risk-informed criteria for design.

The JNES guidance and this report address beyond design basis loadings differently. JNES-RC-2013-1002 presents an approach that provides a margin for design basis shaking: on isolator capacity, clearance to adjacent structure, and on crossover piping capacity. No specific margin is identified and so the potential for unexpected consequences during beyond design basis shaking such as overloading of isolators, pounding between structures, excess stresses on umbilical (crossover) systems, and obstacles in the path of the upper basemat or isolator units, is not formally addressed. By contrast, this report recommends the inclusion of a physical stop in isolated structural designs to a) limit the possible range of motion of the isolations system in beyond design basis shaking, and b) reduce the seismic risk associated with isolator failure to less than 1×10^{-6} per year. The inclusion of a stop requires that it be designed for impact loadings. Section 5.3 of JNES-RC-2013-1002 introduces an *excessive displacement stopper* but provides neither a basis for its design nor its location with respect to the isolated structure.

Generally, the recommendations provided in this report are much more detailed than in JNES-RC-2013-1002 in terms of providing specific analysis and design approaches, as well as detailed performance criteria. The approach to numerical analysis differs from JNES-RC-2013-1002, which allows for the components of motion to be treated separately. The design of a system per JNES-RC-2013-1002 uses the absolute sum of vertical and horizontal motions or SRSS if the peaks are not close in time. This report recommends a fully coupled analysis for all but a few special cases. Both documents treat the design of the superstructure using conventional methods once the foundation input response spectra are developed. There are a number of similarities in the criteria and both approaches require consideration of the construction, operation and maintenance of the isolators and isolation system of an NPP at the time of design.

Both JNES-RC-2013-1002 and this report address:

- Performance of umbilical lines (called crossover structures in the JNES guidance)
- Rocking and rotation
- Testing of isolation units to determine or verify mechanical properties
- Incorporation of variability of properties in analyses
- Validation of analytical models against test data
- Use of earthquake records rich in long-period motion
- Other external events and loading conditions

[1] Seismic probabilistic risk assessment is called seismic probabilistic safety assessment (SPSA) in Japan. SPSA in Japan are conducted as described in *Probabilistic Safety Assessment Implementation Standard on Earthquakes of Nuclear Power Stations*: AESJ-SC-P006" (AESJ, 2007)

Both the guidance in JNES-RC-2013-1002 and the recommendations of this report include:

- Design of the isolation system to assure that vertical load bearing capacity is maintained at all times, including under extreme loading conditions
- The isolation systems be treated as safety-related unless they support a non-safety related structure
- Inspection and maintenance programs be developed and isolators be replaceable if needed
- A post-earthquake inspection program be developed
- Seismic monitoring equipment be installed
- Implementation of quality control systems consistent with safety-related equipment be developed

6.1.2 JEAG 4614

The JEA, a membership organization supporting the electrical industry in Japan, has developed a number of technical guidelines related to the design of nuclear power facilities. One such document is JEAG 4614-2000 *Technical guidelines on seismic base isolation systems for structural safety and design of nuclear power plants* (JEA, 2000). The NRC was also provided with JEAG 4614-2000 in Japanese. The NRC translated JEAG 4614 for internal use and its contents are introduced below. These guidelines apply to nuclear power facilities incorporating laminated rubber seismic isolation bearings and any associated energy dissipation devices. However, these guidelines may be adapted to any class of isolation hardware meeting the appropriate performance specifications described therein.

According to these guidelines, an NPP must be designed such that both the facility personnel and the general public are not exposed to excessive radiation in the event of any natural or human-based disaster, including earthquake, flood, tsunami, wind, freezing, snow accumulation, landslides, lightning, aircraft impact, bursting of dams, and explosions. To achieve this objective, the JEA guidelines are broadly organized to address the following:

a. Classification of Seismic Isolated Nuclear Power Facilities

b. Seismic Isolation Design and Evaluation Methodology

c. Load Combinations and Required Margins of Safety

d. Performance Requirements for Seismic Isolation Bearings and Damping Devices

e. Design Requirements for Secondary Systems (e.g., machinery, plumbing, etc.)

f. Quality Control and Maintenance Requirements for Seismic Isolation Hardware

A complete and direct comparison between this report and the JEA Guidelines is challenging because JEAG 4614 has been published only in Japanese and provides highly detailed guidance. A general comparison indicates that the guidance provided by JEAG 4614-2000 has many commonalities with design requirements for seismic isolation systems in non-nuclear structures in the US. However, there are fundamental differences in the definition of seismic hazard. In the JEA guidelines, the seismic demand for the linear static procedure does not appear to vary by geographic location and does not appear to correspond to a specific mean annual frequency of exceedance. For response-history analysis, seismic demand is defined using three acceleration records scaled to a minimum value of spectral velocity, without a description of the technical basis for such an approach. This contrasts with the approach recommended in this NUREG/CR, which operates in a risk-informed, performance-based framework.

6.2 Guidance for Seismic Isolation of Nuclear Facilities in France

Currently, there is no regulatory guidance for the seismic isolation of nuclear facilities in France. Instead, designs are based on an enhancement to European Standard 1337 (CEN, 2005) and European Standard 15129 (CEN, 2009). Construction and QA/QC requirements are provided to address the testing and replacement of bearings.

6.3 Guidance for Seismic Isolation of Nuclear Facilities by the International Atomic Energy Agency

At the time of this writing, a special Working Group on Seismic Isolation of NPPs is developing an international guidance document on the use of SI. The document is expected to provide guidance on technical areas where consensus exists among the member states. It is also expected to contain a series of appendices that will describe current practice in different member states.

Table 6-1 Comparison of JNES-RC-2013-1002 and this NUREG/CR Report

JNES-RC-2013-1002	This report
Does not specify types of isolators. Provides design and review criteria to address a broad range of possible SI approaches.	Specifies three types of isolators (low damping natural rubber, lead rubber, and spherical sliding) as generally appropriate and two types (synthetic rubber and high-damping rubber) as inappropriate. Does not preclude other types of isolators but provides a list of qualification activities to be undertaken to demonstrate the appropriateness of new isolator designs.
Deterministic design with design criteria provided up to DBE	Performance-based
Consideration of residual risk using seismic probabilistic safety risk analysis	Recommendations for beyond-design-basis ground motions
Focused on foundation isolation for new NPPs and equipment and floor isolation for existing NPPs	Focuses on foundation isolation for new NPPs, but does not preclude other uses
Includes a discussion of both horizontal and vertical isolation	Focuses on horizontal isolation with limited discussion of vertical isolation
Prefers time-history method and allows for modified SRSS (all maximums combined)	Identifies three methods and recommends time-domain non-linear 3D modeling of the isolated superstructure in most cases

7 MODELING AND ANALYSIS OF SEISMICALLY ISOLATED NUCLEAR STRUCTURES

This chapter provides recommendations and options for analysis of seismically isolated NPPs (Section 7.1) and modeling of seismic isolators and systems (Section 7.2).

Section 7.1 provides three analysis procedures for seismically isolated NPPs. Each procedure involves soil-structure-interaction (SSI) analysis. Section 7.1 directs the reader to consensus standards such as ASCE 4-16 (ASCE, 2017) and ASCE 43-05 (ASCE, 2005), and United States Nuclear Regulatory Commission (NRC) documents for information.

Section 7.2 discusses mathematical models for seismic isolator units. This section does not provide details on modeling non-isolated components of NPP structures (e.g., reinforced and prestressed concrete, structural steel). Instead, it directs the reader to consensus standards such as ACI 349 and AISC N690 for information.

7.1 Analysis of Seismically Isolated Structures

7.1.1 Overview

Three methods of analysis of seismically isolated structures are presented below: 1) coupled time domain, 2) coupled frequency domain, and 3) multi-step. Each is described in detail in the following subsections and each is consistent with the requirements of Chapter 12 of ASCE 4-16. Three-dimensional models should be analyzed using the simultaneous input of three translational components of ground motion. Models of the superstructure, foundation and soils should conform to standard NRC practice. Numerical models of the isolators should conform to the guidance provided below. The treatment of natural and accidental torsion should conform to standard NRC practice.

A coupled time-domain analysis involves a nonlinear time-domain analysis of the soil-foundation-isolator-superstructure system. Nonlinear finite elements should be used for all components in the mathematical model that are expected to respond inelastically. Coupled time-domain analysis is the preferred method of analysis and there are no restrictions on its use in terms of isolator types or non-linear models response.

A coupled frequency-domain analysis involves a frequency-domain analysis of the soil-foundation-isolator-superstructure system. Equivalent linear properties are used for all components in the mathematical model. This analysis procedure can be used for proportioning a nuclear facility equipped with low damping (natural) rubber (LDR) bearings, if the bearings can be modeled accurately as linear viscoelastic elements over the range of shear strain expected for the chosen intensity of shaking. This analysis procedure can be used in the first step of the multi-step method if nonlinear bearings (i.e., lead-rubber or Friction Pendulum™) are being used.

The multi-step method involves two analyses: 1) propagation of rock outcrop ground motion into a model of a soil-foundation-isolator-superstructure system for the purpose of generating a Seismic Isolation Design Response Spectrum (SIDRS) at the level of the foundation (see Figure 3-1), and 2) nonlinear response-history analysis of a model of the isolated superstructure using three component acceleration time series consistent with the SIDRS. This analysis procedure can be used for LDR, lead-rubber (LR) and spherical sliding (Friction Pendulum) isolation systems.

The impact on the response of an isolated structure resulting from variations in isolator mechanical properties should be evaluated. Huang *et al.* (2009a) presents a procedure for such computations that avoids the need for Monte Carlo analysis.

Large displacements across the isolation plane may be expected for design basis and beyond design basis ground motions. The computer code used for the analysis must accommodate large displacements and second-order effects.

The analyst should verify the adequacy of the numerical models of the soils, SSCs, and isolators, after the analysis is completed, to ensure that assumptions made are not violated. Examples include a) strain range over which equivalent linear isolator response is maintained, b) tensile pressures (loadings) on elastomeric and sliding bearings, c) displacements sufficient to trigger vertical and horizontal coupling of elastomeric isolators, and d) ensuring inelastic action is confined to the isolation system (e.g., Politopoulos and Sollogoub (2005)).

7.1.2 Coupled Time-Domain Analysis

A coupled time-domain analysis requires the analyst to build finite element models of the soil, foundation, isolators, and superstructure.

Meshing of the soil, foundation and superstructure should conform to consensus standards such as ASCE 4-16 (ASCE, 2017), and NRC guidance. Mesh refinement for the soil should be performed and the analyst should demonstrate the chosen element type and size is sufficient to transmit frequencies across the range of interest (e.g., Bolisetti *et al.* (2016)). Appropriate and validated nonlinear constitutive models for soils should be used. Multiple soil meshes should be constructed for analysis to represent lower bound, best estimate and upper bound mechanical properties.

Mesh sizes for the foundation and superstructure should be similar to those used for non-isolated construction and be capable of transmitting frequencies across the range of interest for SSCs.

Isolators will generally be modeled as beam elements connecting the foundation (or pedestal) to the basemat (see Figure 3-1). Each isolator in the isolation system should be modeled explicitly. The models used for the isolators should capture the behaviors identified in Section 7.2.3.

Time-domain analysis requires the analyst to input acceleration time series around the boundary of the finite element model. The number of sets of three-component acceleration time series to be input to the model is dependent on the goal of the analysis. If the goal is to establish mean estimates of response (e.g., horizontal displacement of the isolation system, floor acceleration response spectra, shearing force in walls), a minimum of five sets of motions should be propagated through each soil mesh to generate a minimum of fifteen values of each response parameter. If the goal is to establish distributions of response (e.g., 99th percentile displacement for design basis shaking), 10 or more sets of motions should be propagated through each soil mesh to generate 30 or more values of each response parameter.

Floor spectra for analysis and design of equipment supported above the isolation interface (see Figure 3-1) should be generated from the results of the nonlinear response-history analysis. Such spectra may then have to be modified per NRC practice as outlined in RG 1.122 (NRC, 1978).

7.1.3 Coupled Frequency-Domain Analysis

Equivalent linear, frequency-domain analysis is standard practice for design of non-isolated nuclear facilities. Guidance on such analysis is provided in ASCE 4-16 and NRC documents, and is not repeated here. A fully coupled frequency-domain analysis is suitable for design of nuclear facilities equipped with LDR bearings, which can generally be modeled with linear viscoelastic elements.

Isolators will be modeled as beam elements with equivalent linear properties (i.e., equivalent stiffness, damping) as described in Section 7.2.2 below. The beam elements will connect the foundation (or pedestal) to the basemat (see Figure 3-1). If any of the following conditions apply, equivalent linear models of LDR bearings cannot be used to calculate responses for design and either the multi-step method of Section 7.1.4 or the coupled time-domain method of Section 7.1.2 should be used to compute responses:

- The shear strain expected for the chosen intensity of shaking exceeds the shear strain at the onset of stiffening[1]
- Coupling of the vertical and horizontal responses is likely at the shear strain expected for the chosen intensity of shaking[2]
- Cavitation (e.g., Constantinou *et al.* (2007)) is expected in the LDR bearings for the chosen intensity of shaking.

The equivalent linear properties chosen for analysis of a nuclear facility isolated with LDR bearings a) should be shown to be appropriate following the analysis, and b) form a basis of the prototype- and production-testing program for the LDR isolators.

Floor spectra for analysis and design of equipment supported above the isolation interface could be generated from the results of the frequency-domain analysis per standard practice.

7.1.4 Multi-step Analysis

Multi-step analysis of a nuclear facility equipped with seismic isolators will generally involve two steps: 1) development of SIDRS at the foundation level of the isolated structure, and 2) nonlinear analysis of the isolation system and isolated structure using acceleration time series that are consistent with the SIDRS.

Step 1: Development of SIDRS

The SIDRS at the foundation level of an isolated nuclear facility will typically be generated by frequency-domain analysis, although time-domain procedures are equally viable. The generation of SIDRS using frequency-domain procedures will require the development of equivalent linear properties for the isolators. Section 7.2.2 provides guidance on these calculations. Aside from the isolators, the mathematical models (e.g., soils, structure) and analysis procedures used to generate the SIDRS should be identical to those models and procedures used for conventional (non-isolated) nuclear structures. The equivalent linear properties chosen for the isolators a) should be shown to be appropriate following the second step of the multi-step analysis.

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[1] As noted in Section 4.2.1.2.2, the horizontal stiffness of LDR bearings can be sensitive to axial load. The potential for vertical load to change an LDR isolator's dynamic properties should be considered.

[2] The analyst should consider the need for the shear strength to accommodate the clearance to the stop as part of the analysis and design process.

Step 2: Nonlinear response-history analysis of the isolated superstructure

Coupled nonlinear time domain analysis of the isolation system and isolated structure is the second step in the procedure. Acceleration time series should be generated to be consistent with the SIDRS. Three-component translational time series must be generated and rotational (rocking) time series should be generated if such motions are shown to be of significance by the SSI analysis in step one.

Isolators will generally be modeled as beam elements connecting the foundation (or pedestal) to the basemat. Each isolator in the isolation system should be modeled explicitly.

The number of sets of acceleration time series to be input to the model of the isolated superstructure is dependent on the goal of the analysis. If the goal is to establish mean estimates of response (e.g., horizontal displacement of the isolation system, floor acceleration response spectra, shearing force in walls), a minimum of five sets of motions should be used. If the goal is to establish distributions of response (e.g., 99th percentile displacement for design basis shaking), a minimum of 30 sets of motions should be used.

Floor spectra for analysis and design of equipment supported above the isolation interface (see Figure 3-1) should be generated from the results of the nonlinear response-history analysis. Such spectra may then have to be modified per NRC practice as outlined in RG 1.122.

7.2 Modeling of Isolator Units

7.2.1 Mechanical Properties of Isolator Units

The mechanical characteristics of the seismic isolation system will govern the seismic response of the isolated nuclear facility. The behavior of the seismic isolation system will be highly dependent on the mechanical properties of the seismic isolator units, which may evolve over the course of an earthquake, as described below.

Two types of mathematical models of isolators can be used for analysis, subject to certain limitations: equivalent linear, and nonlinear. Each type of model is described below.

The equivalent linear models can be used in the following cases: a) modeling LDR bearings in a coupled frequency-domain analysis (Section 7.1.3), and b) modeling isolators in the first step of a multi-step analysis (Section 7.1.4).

Nonlinear models can be used for all types of isolators considered appropriate at this time for possible use in nuclear facilities in the US: low damping rubber (LDR), lead-rubber (LR), and spherical sliding (Friction Pendulum™, FP) bearings. The nonlinear models can be used for a coupled time-domain analysis (Section 7.1.2) or the second step in a multi-step analysis (Section 7.1.4).

7.2.2 Equivalent Linear Models of Isolators

An equivalent linear model of a seismic isolator should include a) axial stiffness, and b) effective horizontal stiffness along each of two orthogonal horizontal directions.

The axial stiffness of the FP bearing in compression is very high, and primarily a function of the stiffness of the slider (or sliders) between the sliding surfaces.

The axial stiffness of an elastomeric (LDR or LR) bearing in compression should be calculated from first principles; guidance is provided in Kelly (1993), Naeim and Kelly (1999) and

Constantinou *et al.* (2007). The axial stiffness in tension, prior to cavitation, can be set equal to the value in compression for the purpose of equivalent linear analysis.

The effective horizontal stiffness and effective damping ratio of a nonlinear isolator (i.e., LR bearing or FP bearing) is calculated using Figure 7-1 (which is similar to Figure 3-3). The equations have been used for more than 20 years and are those provided in ASCE 7-10. Derivations are available in a number of texts, including KKelly (1993) and Naeim and Kelly (1999).

The effective or secant horizontal stiffness (K_{eff}) of a bearing can be computed as:

$$K_{eff} = \frac{|F_{max}| + |F_{min}|}{|\Delta_{max}| + |\Delta_{min}|} \tag{7.1}$$

where Δ_{max} is the maximum positive horizontal displacement of a bearing, Δ_{min} is the minimum negative horizontal displacement of a bearing, and F_{max} and F_{min} are the horizontal forces corresponding to Δ_{max} and Δ_{min}, respectively. The effective damping ratio (β_{eff}) for a bearing can be computed as:

$$\beta_{eff} = \frac{2}{\pi} \frac{\mathrm{EDC}}{K_{eff}\left(|\Delta_{max}| + |\Delta_{min}|\right)^2} \tag{7.2}$$

where EDC is the energy dissipated per cycle of loading and all other terms were defined previously.

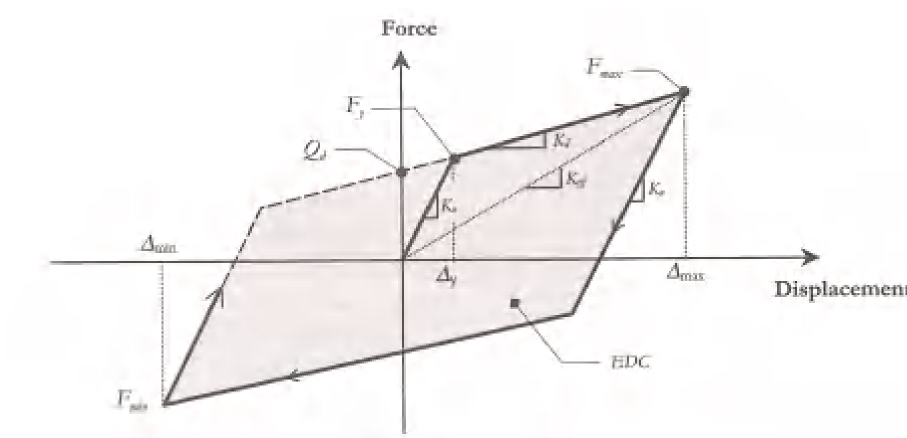

Figure 7.1 Linearization of a Nonlinear Isolator Unit (HITEC, 1999)

7.2.3 Nonlinear Models of Isolators

Nonlinear isolator models are required for time-domain analysis of isolation systems incorporating LR and FP bearings. A robust nonlinear model of an isolator should permit calculations of response under compressive and tensile axial loadings for random bidirectional horizontal motions with amplitude ranging from zero to the clearance to the stop (see Chapter 8). The models used for the isolators should capture:

- Expected axial (vertical) force-deformation relationships in tension and compression

- Expected horizontal force-deformation relationships
- Coupling of vertical and horizontal force-deformation relationships
- Effects of energy dissipation on horizontal force-deformation relationships

The required completeness of the model will be a function of the intensity of shaking (i.e., design basis or beyond design basis) and the expected response. For examples: 1) if axial tension is not expected in a LDR or LR bearing for the chosen intensity of shaking, there is no need to model axial tensile stiffness and the effects of cavitation, and 2) if stiffening is not expected in a triple FP bearing, there is no need to include this regime in the model. Assumptions made in model development should be verified after the analysis. Kumar *et al*. (2015e) provide guidance on the impact of different assumptions on the seismic response of a base-isolated NPP.

As a minimum, a nonlinear model should include axial force-displacement relationships in compression and tension, and coupled bilinear horizontal force-displacement relationships.

Axial stiffness should be modeled carefully because the chosen force-displacement relationship will influence a) vertical motions transmitted to the isolated superstructure and thus vertical floor spectra, and b) rocking response of the isolated superstructure. Coupling of vertical and horizontal responses can also affect the response of the isolation system. The axial stiffness of an elastomeric isolator reduces with an increase in horizontal displacement and this coupling of responses should be addressed explicitly. Fully coupled three-dimensional models of isolators (e.g., Koh and Kelly (1987); Ryan *et al.* (2005); Warn and Whittaker (2006a); Kumar *et al.* (2014, 2015a, 2015c)) can be used for this purpose.

The elastic stiffness (K_u in Figure 7-1) and the transition from elastic stiffness to second-slope stiffness (K_d in Figure 7-1) should be modeled carefully so as not to generate spurious higher mode effects in the isolated superstructure. Best estimate values of these relationships should be used for modeling.

The mechanical properties of nonlinear seismic isolation bearings will evolve over the duration of strong shaking as energy is dissipated: the yield strength of a LR bearing will drop as the lead core heats (and dissipates energy), and the coefficient of sliding friction will drop as the stainless steel overlay heats (due to frictional heating). Chapter 4 and Constantinou *et al.* (2007) provide information on this important subject.

Research funded by the NRC has advanced numerical models of elastomeric and sliding isolators to address extreme earthquake loadings. The advanced models for LDR and LR are described in Kumar *et al.* (2014) and Kumar *et al.* (2015a, 2015e). These models address a) change in hysteresis due to energy dissipation (LR bearings), b) tension force-displacement relationships, and c) coupling of tensile/compressive axial forces and horizontal displacements. The LDR and LR bearing models have been validated (e.g., Kumar *et al.* (2013, 2015a)). The advanced model for the high damping rubber bearing (HDR), which incorporated attributes from the advanced LDR model and the shear force-shear displacement relationship of Grant *et al.* (2004), is described in Kumar *et al.* (2015a). The LDR, LR and HDR models have been implemented in the commercial finite element codes LS-DYNA and ABAQUS, and the open-source code OpenSees (McKenna *et al.*, 2006):

- LDR: http://opensees.berkeley.edu/wiki/index.php/ElastomericX
- LR: http://opensees.berkeley.edu/wiki/index.php/LeadRubberX
- HDR: http://opensees.berkeley.edu/wiki/index.php/HDR

The advanced model for the FP sliding bearing is described in Kumar *et al.* (2015b, 2015c). This model captures the dependence of the coefficient of sliding friction on a) temperature on the sliding surface, b) pressure beneath the articulated slider, and c) relative velocity of the slider over the hemispherical surface, and is suitable for nonlinear response-history analysis.

8 RECOMMENDED PERFORMANCE CRITERIA FOR SEISMICALLY ISOLATED NUCLEAR FACILITIES

8.1 Philosophy in Developing Performance Criteria

In developing the performance criteria discussed in this chapter, significant effort was placed on defining criteria that are directly based on the NRC's risk-informed goals and a set of clear design philosophies. Because this report addresses a structural element that has not been applied previously to nuclear plants in the US, explanation of the technical basis for the approaches and criteria documented in this chapter is provided. In addition to the performance criteria described below, Chapter 9 provides additional recommendations related to the design, construction, operation and maintenance of the isolators and isolation system.

Non-redundancy of the critically important seismic isolation system poses a challenge for the development of appropriate performance criteria. The seismic isolation (SI) system uniquely connects the superstructure to the foundation. As such, the SI system is not redundant, even though individual isolator units can be considered to have a significant level of redundancy. The isolation system is composed of a large number of essentially identical isolators that act in parallel and experience similar but not fully correlated loadings. Therefore, it is important to demonstrate a High Confidence of Low Probability of Failure (HCLPF) of the isolation system for shaking more intense than the design basis and to avoid cliff-edge effects by introducing a physical barrier (i.e., stop) that limits the relative motion of the foundation and the basemat to preclude failure of isolators. To build further confidence, it is also important to demonstrate that the performance of the SI system will not be significantly compromised by failure of one or even several isolators.

The need to analyze performance under beyond design basis loading levels has been a consideration in the US nuclear regulatory environment for several decades. Design basis accidents or events are the postulated accidents or events that a nuclear facility must be designed and built to withstand without loss to the structures, systems and components necessary to ensure public health and safety. Beyond design basis accidents or events are analyzed and assessed to fully understand the capability of a design. In that sense, they are reviewed in relation to NRC's risk-informed performance goals (NRC, 1993a, 1993b, 2010). More recently, the NRC's Post-Fukushima Near Term Task Force Report (NRC, 2011) described a concept that includes specific performance and risk criteria for more intense ground motions than those used in design. In this case, the ground motions and associated performance or risk criteria become part of the licensing basis of the NPP and are a point of review for license applications.

The elements of the design basis and beyond design basis developed here for seismically isolated NPPs are discussed in the following sections and can be summarized as follows:

- The Ground Motion Response Spectra+ (GMRS+) are design basis ground motion response spectra (typically a geomean horizontal spectrum and a vertical spectrum) that are used to design the NPP and the isolators
- Beyond Design Basis (BDB) Ground Motion Response Spectra (GMRS) is a level of ground shaking used for review and assessment and is intended to assure that the isolation system, umbilical lines, and stop can perform their safety-related functions when subjected to more extreme ground motions.

Considerations used in creating the recommended performance criteria include:

- Singletons[1] that are safety related must have more stringent design criteria than non-isolated nuclear construction for which singletons do not exist.
- The concepts of First Onset of Significant Inelastic Deformation (FOSID) and HCLPF as used in non-isolated structures should be incorporated to the extent possible, while recognizing that most seismic isolators are inherently non-linear.
- The potential for cliff edge effects must be removed through use of a stop (physical constraint, sometimes referred to as a “fail-safe system” in international guidance).
- Assurance of performance must incorporate a combination of prototype and production testing to physically demonstrate quantifiable confidence levels and performance reliability in both the isolators and the umbilical lines.
- The performance criteria should consider and clarify how SI systems could fit within a certified design framework.
- Although this report focuses on seismically isolated light water reactor superstructures, the approach should be sufficiently technology neutral to be extended to other designs (e.g., small modular reactors and non-light water reactors) and other uses (e.g., equipment isolation).
- Realistic approaches for achieving clear and technically based performance targets should be described.

8.2 Performance Matrix

Table 8-1 provides a recommended matrix of performance statements for the analysis and design of isolated nuclear power plants, where the overall goal is to achieve a mean annual frequency of exceedance of failure of the isolation system of less than 1×10^{-6}. The cells in the matrix provide a risk-informed, performance-based framework for analysis and design, including calculations for design basis shaking, and beyond design basis shaking. Performance statements are made at each level of hazard for isolators and the isolation system, superstructure, umbilical lines, and the hard stop.

Table 3-1 provides a list of basic definitions (see also Figure 3-1) that are used here. For the purpose of this discussion, structural elements of the isolated power plant are further grouped and defined as follows:

***Isolators and isolation system*:** The isolation system is composed of seismic isolators or bearings (and possibly supplemental passive damping devices) that serve to isolate the supported structure from the effects of high frequency, horizontal earthquake shaking. The isolation system also includes all structural elements that transfer force between elements of the isolation system and all connections to other structural elements. The isolation system is installed in a horizontal plane without significant vertical offsets across the plan dimensions of the isolated structure. The isolators are installed atop pedestals that enable inspection and replacement as needed.

***Superstructure*:** The superstructure is composed of the structural framing (e.g., slabs, beams, columns, walls) that provides resistance to loads (e.g., gravity, wind and earthquake

[1] Singletons are SSCs of which there is only one and no redundancy exists. An example is the containment vessel.

loadings, impact loadings, pressure loadings, among others) and the nonstructural components of the building. For a non-isolated LWR, the structural framing would include primary and secondary containment, internal structure to support the power generation and safety-related components and systems, and the basemat (or diaphragm) immediately above the isolation system. The superstructure supports the systems and components above the isolator system.

Foundation**:** The foundation is composed of the mat beneath the isolation system, the pedestals, the moat walls and/or stop, and any other foundation elements.

Umbilical lines**:** Umbilical lines are nonstructural components and systems, mainly distribution systems that cross the isolation interface and must sustain the large isolator displacements (or deformations) associated with design basis and beyond design basis earthquake shaking. Examples of umbilical lines could include high-pressure steam lines from the power reactor to the turbines and cables located on trays or in ducts from emergency power systems located off the nuclear island to the power reactor (Umbilical lines are also sometimes called crossover structures in international guidance).

Stop**:** The stop is a series of rugged structural elements (e.g., the moat wall) that are designed to indirectly protect the isolators from earthquake damage (by preventing additional horizontal displacement) in shaking more severe than design basis. The stop could be an integrated structural system such as a moat surrounding the isolated superstructure or a series of isolated but integrated structural elements. The horizontal gap between the supported superstructure and the hard stop should be equal to or greater than the 90^{th} percentile displacement associated with beyond design basis shaking (see Table 8-1), as calculated for each horizontal axis of the isolated structure. The stop should be designed to assure that it acts as a physical restraint on the displacement of the isolated superstructure.

The performance matrix in Table 8-1 does not address the analysis and design of all structural elements in a NPP. It is expected that the structural elements themselves would be designed using best practices and the applicable codes and standards. The structural elements should be designed to assure acceptable performance under the loads that result from the analyses described in Chapter 7. For example, the pedestals supporting the isolators, and the foundation slab supporting the pedestals, should be designed for the maximum axial and shearing forces and bending moments the isolators could deliver at a lateral displacement equal to the clearance (gap) to the hard stop, along each axis of the structure. The pedestals and foundation slab would be designed for these forces without reduction for inelastic action and would have to be detailed for limited ductile response using the prescriptive rules set forth in RG 1.142 and the related ACI 349 code, such that the NRC's existing design and performance criteria are met. Other forces and their load combinations per ASCE 4-16 (ASCE, 2017) may have to be considered for these structural elements.

Below, one pathway is provided for achieving NRC's performance goals. Other strategies could be followed but they are not described here.

8.3 Hazard Definitions for Analysis of Seismic Isolation Systems

Two levels of seismic hazard should be considered in the design of an isolated safety-related nuclear structure, as discussed below. The ground motions are based on probabilistic hazard definitions. The deterministic term “design basis earthquake” found previously in this report is not used and is replaced with the more appropriate and descriptive “design basis ground motion response spectrum”.

Horizontal spectra are defined as 5% damped geometric mean unless noted otherwise. Pairs of horizontal ground motion records should be selected and scaled in a consistent manner with the geometric mean spectrum (i.e., Max-Min per Huang *et al*. (2009b)), with explicit consideration of correlation, and for sites outside the CEUS, near-fault shaking effects. Vertical motions may also require scaling. A discussion of state-of-the-art approaches to ground motion selection and modification techniques can be found in NIST GCR 11-917-15 (NIST, 2011). Simultaneous horizontal and vertical loading should be applied as discussed in Chapter 7.

> **Ground Motion Response Spectra+ (GMRS+):** Ground motion response spectra (GMRS) are defined per NRC RG 1.208 (NRC, 2007a) and ISG-DC/COL-20 (NRC, 2010a). These spectra are typically associated with earthquake shaking that has a mean annual frequency of exceedance (MAFE) of 1×10^{-4} (ground motion shaking return period of 10,000 years). The ordinates of the vertical spectrum in the GMRS may be further increased by a design factor, as described in RG 1.208 and ASCE 4-16. The horizontal GMRS+ is a composite spectrum that envelops the GMRS and the free field ground motion (at the GMRS control point) consistent with a mini`mum foundation input motion required by Appendix S to 10CFR50 (i.e., an appropriate spectral shape, such as the RG1.60 spectral shape, anchored to a PGA of 0.1g). The GMRS+ ground motion is developed in this report to incorporate both the GMRS described in RG 1.208 and the minimum requirement described in 10CFR50 in a single transparent input response spectrum.

> **Beyond Design Basis (BDB) Ground Motion Response Spectra (GMRS):** The uniform hazard response spectra associated with a MAFE of 1×10^{-5} (ground motion shaking return period of 100,000 years) but with ordinates no less than 167% of the GMRS+ spectrum (ISG-DC/COL-20) is the proposed seismic margin review ground motion. This MAFE is associated with First Onset of Significant Inelastic Deformation (FOSID) in non-isolated NPPs. The Beyond Design Basis (BDB) ground motion represents shaking that are more intense than the GMRS+ and are used to assess the ability of the isolation system to perform their safety-related function in more extreme conditions than design basis. The provision of a stop, the low probability of the superstructure hitting the stop at this BDB ground motion level and the prototype testing of isolators for BDB displacements and axial forces assures that the MAFE of unacceptable performance of the isolation system is less than 1×10^{-6}.

Calculations that support the choice of return period for the BDB GMRS to achieve the MAFE of unacceptable performance of smaller than 1×10^{-6} are provided in Kumar *et al*. (2015c) and Kumar *et al*. (2017). These calculations assume that demands and capacities of all isolators in a system are fully correlated, which is incorrect but both very conservative and necessary; necessary because the number of isolators in an isolation system and the distribution of isolators across the plan footprint of an isolated structure will be project specific, whereas these recommendations are generic.

Analysis is performed using the GMRS+ ground motion level to a) provide displacements and axial forces for production testing of isolators, b) provide design basis motions for the isolated superstructure, and c) ensure the probability of the isolated superstructure striking the stop (or moat) is less than 1%.

Analysis is performed using the BDB GMRS ground motion level to a) estimate the minimum clearance to the stop (CS) or moat along each axis of the structure, b) provide axial forces and lateral displacements for prototype testing of isolators, c) establish high confidence that the umbilical lines that cross the isolation interface will perform as intended, and c) provide design forces for the stop or moat.

The criteria described above differ from those in ASCE 43-05, although both are intended to meet plant-level performance and risk targets, due to differences in the behavior of isolated and non-isolated NPPs and the goal of this report to provide criteria that can be broadly applied beyond light water reactors. As described in Section C1.3 of ASCE 43-05, the criteria provided in ASCE 43-05 were developed through analysis of non-isolated light water reactors and were intended to provide engineering-based targets that are practical for the design engineer yet still meet performance goals. The criteria in ASCE 43-05 have not been demonstrated to meet NRC objectives for seismically isolated NPPs or for designs that differ from the current generation of light water reactors.

8.4 Performance Expectations for GMRS+ Shaking

For GMRS+ ground motion levels, the isolators (and thus the isolation system) are expected to suffer no damage and be capable of sustaining large aftershocks within tens of minutes of the main shock, also with no damage. The performance of each isolator should be confirmed by production testing at the mean lateral displacement of the center of mass of the isolated superstructure associated with GMRS+ ground motions. For each size and type of isolator in the isolation system, the 16th percentile and 84th percentile earthquake-induced axial force should be calculated at this lateral displacement. A range of axial force should be considered during production testing of each isolator to account for gravity load effects and the 16th percentile and 84th percentile earthquake-induced axial loads.

The superstructure is designed for the forces, displacements and deformations, and floor accelerations associated with mean demands from computed by response-history analysis for GMRS+ shaking levels. Proportioning of elements in structural framing should use load combinations and design capacities per relevant NRC guidance and related codes and standards for nuclear safety-related structures.

There is no performance statement for the safety-related umbilical lines at GMRS+ shaking levels because more onerous demands are imposed at BDB shaking levels.

The analyst should demonstrate that the probability of the isolated superstructure striking the stop or moat wall is less than or equal to 1% for the GMRS+ shaking. Response-history analysis is generally required to accomplish this goal. Examples of calculation procedures are presented in Huang *et al*. (2009a) for conditions and ground motion levels consistent with hazard estimates (at the time the analyses were performed) at the sites of three NPPs in the US: the North Anna Power Station, Vogtle Electric Generating Plant, and the Diablo Canyon Power Plant. Pairs of ground motions should be scaled to the geometric mean GMRS+ spectrum for use in the response-history analysis, with appropriate considerations of variability in the two components and directionality (see Huang *et al*. (2009a) for details). Spectrally matched ground motions, which are often used to estimate mean (central tendency) responses and not distributions of response, cannot be used for computing distributions of displacement response. Alternatively, the mean maximum GMRS+ displacements from response-history analysis can be multiplied by factors that can be computed from information in Huang *et al*. (2009a) to establish the 99th percentile GMRS+ displacement. The appropriate factor varies based on site conditions, characteristics of the ground motion, and the type and properties of the isolator. A conservative value of the (mean to 99th percentile) scale factor is 2.2.

8.5 Performance Expectations for BDB GMRS Shaking

8.5.1 Clearance to the Stop

A physical stop is likely necessary for seismically isolated, safety-related nuclear structures to ensure the mean annual frequency of failure of the isolation system is very small. Here, the return period on BDB GMRS shaking is set equal to 100,000 years so as to achieve a mean annual frequency of failure of the isolation system of smaller than 1×10^{-6}. A moat wall can serve as a stop.

An isolated nuclear structure will displace in the horizontal plane into a space defined by the stop provided in two orthogonal horizontal directions of the structure. The clearance between the isolated building and the stop along each horizontal direction would be greater than or equal to the 90th percentile BDB GMRS displacement (i.e., the 90th percentile displacement that results from the BDB GMRS ground motions) in that direction. The clearances in the two directions will likely be the same. The 90th percentile BDB GMRS lateral displacements in each horizontal direction can be calculated by response-history analysis of the isolated structure using pairs of ground motions scaled to the geometric mean BDB GMRS, with appropriate considerations of variability in the two components and directionality (see Huang *et al*. (2009a) for details). As noted above, spectrally matched ground motions cannot be used for computing a distribution of displacement response.

Alternatively, the mean maximum BDB GMRS displacements from response-history analysis can be multiplied by factors that can be computed from information in Huang *et al*. (2009a) to establish the 90th percentile BDB GMRS displacement. The appropriate factor varies based on site conditions, characteristics of the ground motion, and the type and properties of the isolator. A conservative value of the (mean maximum to 90th percentile) scale factor is 1.6.

8.5.2 Isolators

For ground motions consistent with the above definition of the BDB GMRS, the isolators (and thus the isolation system) are expected to suffer no loss of load carrying capacity and to be capable of sustaining large aftershocks within tens of minutes of the main shock. Minimal damage to the isolators (e.g., loss of cover rubber in an elastomeric bearing, substantial wear of the composite liner in a FP bearing) is acceptable but gravity load carrying capacity cannot be compromised. The integrity of the isolator units and seismic isolation system, measured here in terms of their ability to carry gravity loads, cannot be compromised (as measured by 90% confidence of the performance of the units) at a lateral displacement equal to the greatest displacement possible of an isolator within the space created by the stops, denoted here as CS. Prototype isolators, which are generally few in number for each type and size of isolator being installed as part of the isolation system, are to be tested dynamically at a lateral displacements equal to (or greater than) CS, under a combination of gravity and earthquake-induced axial forces that represent the 16th percentile and 84th percentile axial loads at a lateral displacement equal to CS. Isolators may have to be tested in tension for this extreme loading condition. The 90% confidence limit can be achieved by testing either many prototype isolators if variations in material properties are significant for a given type and size of isolator, or by testing a small number of prototype isolators if variations in material properties are insignificant. In either case, the reliability of the isolator design at the displacement CS should be demonstrated.

8.5.3 Umbilical Lines

To ensure a high confidence of a low probability of failure of the systems and components in the isolated nuclear structure that rely on umbilical connections, all umbilical lines and their connections must be tested (physically or numerically or by a combination of both) to demonstrate that the probability of their failure, conditioned on a displacement in the isolation system equal to CS, is 10% or less. Reducing the dynamic testing program to a small sample of each type of umbilical line and their typical connections may be feasible if either a) the margin against failure is great, or b) the variability in the response of the umbilical line and its connections is very small. Qualification of the reliability of umbilical lines and their connections by numerical analysis would require development of fragility functions that plot probability of failure against an appropriate seismic demand parameter (e.g., lateral displacement for displacement-sensitive umbilical lines), where failure is defined as the loss of ability to perform the intended function.

8.5.4 Stop

Impact between the stop and the isolated superstructure will occur in the event that BDB GMRS shaking generates displacement in the isolation system that exceeds CS. The effects of the impact on the stop should be assessed. The structural element that acts as the stop must be capable of sustaining the effects of the impact. Limited damage to the stop is acceptable provided deterioration of significant strength and stiffness is avoided through the use of ductile details per the prescriptive rules of ACI 349. The type and rigor of the impact analysis will depend on the speed of the isolated superstructure at impact speed, which is a function of the intensity of ground motion, the isolation system design, and the chosen CS. The isolated superstructure need not be analyzed for impact loadings because the mean annual frequency of impact is very low and the velocity at impact will likely be a small fraction of the peak relative velocity over the height of the isolation system.

8.6 Seismic Probabilistic Risk Assessment

Seismic probabilistic risk assessment (SPRA) of isolated nuclear structures will likely be undertaken using procedures similar to that described in Huang *et al*. (2009b) and referenced in Appendix A of ASCE 4-16, wherein seismic demands are calculated by nonlinear dynamic analysis and fragility functions are defined using component-level demands and not using surface free-field spectral demands anchored to peak ground acceleration. Although the recommendations presented above target a mean annual frequency of failure of the isolation system of less than 1×10^{-6}, accident sequences involving the seismic isolation system will likely be developed. Possible failure modes introduced by seismic isolation could include a) axial load failure of the isolators, b) failure of umbilical lines crossing the isolation interface, c) failure of the stop (and an inability to restrain the lateral displacement of the isolation system), and d) damage to the isolators due to internal hazards and external hazards other than earthquakes. Fragility functions for a) could be developed using the guidance provided in Kumar *et al*. (2015c).

Table 8-1 Performance and Design Recommendations for Seismically Isolated Nuclear Power Plants[1]

	Isolation system				
Ground motion levels	**Isolator unit and system design and performance criteria**	**Approach to demonstrating acceptable performance of an isolator unit**	**Superstructure design and performance**	**Umbilical line design and performance**	**Moat or stop design and performance**
GMRS+[2] Envelope of RG 1.208 GMRS and the minimum foundation input motion[3]	No long-term change in mechanical properties. Extremely high confidence of the isolation system surviving without damage when subjected to the mean displacement of the isolator system under the GMRS+ loading.	Perform production testing on each isolator for the mean system displacement under the GMRS+ loading and corresponding axial force.	Superstructure design and performance to conform to NUREG-0800 for GMRS+ loading.	Umbilical line design and performance to conform to NUREG-0800 for GMRS+ loading.	Moat gap sized such that there is less than 1% probability of the superstructure impacting the moat or stop for GMRS+ loading.
BDBE GMRS[4] Envelope of the UHRS at a MAFE of 1×10^{-5} and 167% of the GMRS+ per ISG 20	90% confidence of each isolator and the isolation system surviving without loss of gravity-load capacity at the mean displacement under BDBE GMRS loading.	Perform prototype testing must be performed on a sufficient number of isolators at the CS[5] displacement and the corresponding axial force to demonstrate acceptable performance with 90% confidence. Limited isolator unit damage is acceptable but load-carrying capacity must be maintained.	Less than a 10% probability of the superstructure contacting the moat or stop under BDBE GMRS loading.	Greater than 90% confidence that each type of safety-related umbilical line, together with its connections, shall remain functional for the CS displacement. Performance may be demonstrated by testing, analysis or a combination of both.[6]	Moat gap sized such that there is less than a 10% probability of the superstructure impacting the moat or stop for BDB GMRS loading. Stop designed to survive impact forces associated with isolation system displacement to 95th percentile BDBE isolation system displacement.[7] Limited damage to the moat or stop is acceptable but the moat/stop should perform its function.

1. Analysis and design of safety-related components and systems shall conform to NUREG-0800.
2. 10CFR50 Appendix S requires the use of an appropriate free-field spectrum (often the RG 1.60 spectral shape) with a peak ground acceleration of no less than 0.10g at the foundation level.
3. The analysis can be performed once using a composite spectrum or twice using the GMRS and the minimum spectrum separately.
4. The analysis can be performed once using a composite spectrum or twice using the 1×10^{-5} MAFE UHRS and the 167%GMRS+ separately.
5. CS=Clearance to the Stop
6. Seismic Category 2 SSCs whose failure could impact the functionality of umbilical lines shall also remain functional for the CS displacement.
7. Impact velocity calculated at the displacement equal to the CS assuming cyclic response of the isolation system for motions associated with the 95th percentile (or greater) BDB GMRS displacement.

9 ADDITIONAL RECOMMENDATIONS ON DESIGN, CONSTRUCTION AND OPERATION

The recommended criteria in Chapter 8 address the design and performance of the isolation system, umbilical lines and the physical stop. The recommendations provided below are intended to help ensure risk and performance objectives are achieved.

These recommendations are written for the seismic isolation (SI) of structures associated with large light water reactors. This does not preclude other uses of SI technology but additional requirements may apply in such cases.

9.1 Additional Considerations

9.1.1 Long-Term Changes in Isolator Mechanical Properties

The analysis and design of the isolators and isolation system must account for effects such as aging, creep, operating temperature, exposure to moisture and other deleterious substances in the immediate vicinity of the isolators.

The mechanical properties of the isolation system (i.e., the force-displacement relationships) should not vary over the lifespan of the NPP by more than ±20% from the best-estimate values (with 95% confidence) from those assumed for analysis and design. The range of values used in design should be clearly specified and the in-service inspection program (see Section 9.3.1) must periodically verify that isolator properties remain in this range. If the change in isolator properties exceeds the range used in design, the system should be reassessed and isolators should be changed out if sufficient safety margin cannot be demonstrated.

9.1.2 Basemat and Foundation Design

The basemat and foundation must be designed to have adequate stiffness in both the horizontal and vertical directions to engage all of the seismic isolators in gravity and lateral-load resistance. The analysis and design of the individual isolators and the isolation system should address the short-term and long-term effects of differential settlement of the soil and foundation flexibility. Determination of loading on the isolators and comparison with design assumptions should also be considered if isolators need to be replaced. If the load distribution is not within design assumptions, additional analyses may be needed to demonstrate that the system will perform acceptably.

The basemat should be designed to resist gravity loads assuming the loss of one isolator due to local vertical settlement of the foundation below the isolation system. Multiple calculations, assuming the loss of a different isolator, should be performed.

9.1.3 Anchorage Design

Isolators will be generally installed atop pedestals as shown in Figure 3.1. The isolators will be anchored to the basemat above and the pedestal/foundation below. The anchorage forces used for design should be those developed in the isolators at displacements associated with their maximum possible movement, as limited by the stop (i.e., no less that the 90^{th} percentile beyond design basis displacement along each horizontal axis of the structure). The design of anchorages should follow RG 1.199 (NRC, 2003).

9.1.4 Other External Events

The SI system must be protected against, or designed for fire, high winds, flood, and other natural hazards so that it can perform its intended safety function. The potential for long-term flooding conditions should be addressed for sites in which this hazard exists. In these cases, additional requirements for post-flood inspection and testing of isolators should be developed. Consideration should be given to lightning strikes because rubber bearings can create a break in the energy path. Realistic combinations of naturally occurring events should be considered.

Consideration should also be given to other extreme loadings such as aircraft impact (Blandford *et al.*, 2009), including potential for related fire hazards, and air-blast and ground shock due to accidental and malevolent loadings (Huang *et al.*, 2008). Small or lightweight nuclear structures, isolated or not, may be susceptible to local and/or global failure if subjected to these extreme loadings.

9.1.5 Accident Conditions and Emergency Response

The isolation system must be analyzed and designed to deal with emergency conditions[1]. The protection of the SI system should be included in emergency and severe accident mitigation planning where appropriate.

9.1.6 Moat Cap Design

An appropriate moat cap should be constructed to keep the isolation system clean and clear of debris. It should also be designed to avoid excessive water infiltration from precipitation.

9.1.7 Near-fault Ground Shaking

Near-fault earthquakes can generate high seismic acceleration and displacement demands in fixed-base and isolated buildings, bridges and nuclear structures. Probabilistic seismic hazard analysis for NPPs located close to active faults capable of generating large magnitude earthquakes must consider near-field effects and directivity effects, regardless of whether the plant is isolated. Ground motion response spectra (GMRS) must address near-fault earthquake shaking effects. Sets of ground motions used for design basis analysis and beyond design basis performance assessment should be appropriately selected and scaled to the GMRS per best practice, recognizing the long period response of seismically isolated structures. NIST (2011) provides state-of-the-art information on selecting and scaling ground motions in the near field.

9.1.8 Peer Review

A peer review program should be implemented to review the design of the isolation system, related test programs, and the isolated structure. The review team should consist of one or more members experienced in the application of SI and large-scale testing of isolator units. In addition to other reviews required by the NRC, the applicant's peer review of the SI elements should include, but is not limited to, the following:

- Review of numerical models of isolators
- Review of the SSI analysis and the resulting in-structure response spectra
- Review of displacement and force calculations for the isolator units and all associated structures, systems, and components
- Review of the analysis and design of the umbilical lines

[1] One example of an emergency condition is the loss of offsite power (LOOP). The potential for a LOOP condition led to the recommendation that all components of the SI system are passive and do not require electricity.

- Review of the analysis and design of the hard stop
- Review of the seismic monitoring program
- Review of the prototype test program
- Review of the production (quality control) test program
- Review of the isolator inspection and post-installation testing program
- Review of post-earthquake inspection protocols
- Review of design or protection measures against other external events

9.2 Additional Manufacturing and Construction Considerations

9.2.1 Quality Control and Quality Assurance

Quality control and quality assurance procedures for the testing and construction of the isolator units and seismic isolation system should follow ANSI/ASME NQA-1-2015 *Quality assurance requirements for nuclear facility applications* (ASME, 2015) or approved equivalent. Although the isolation system as a whole acts as a structural element, individual isolators can be considered to be mechanical components that are subject to the requirements of 10 CFR 50, Appendix B and RG 1.28 (NRC, 2010b).

9.2.2 Testing of Prototype and Production Isolators

The testing program must be sufficient to demonstrate adequate performance of the isolator units and isolation system as discussed in Chapter 8. At least two different series of tests should be performed on isolator units: prototype tests and production (quality control) tests.

Prototype tests should be performed individually on full size specimens of each predominant type and size of isolator unit in the isolation system. Test results of full size specimens should be included in applications. Prototype tests should be dynamic and to displacements equal to or greater than the clearance to the stop. Prototype test specimens may not be used in construction. A sufficient number of prototype isolators should be tested to provide a minimum 90% confidence in the performance of the isolators.

Production (construction quality control) tests should be performed on each isolator unit to be installed in the NPP as part of the isolation system. Each isolator should be tested to the mean GMRS+ displacements. Damaged isolators should be rejected and not used for construction.

Appropriate quality control and quality assurance programs must be employed for all testing programs.

9.2.3 Construction Assurance

Consistency with the design assumptions should be verified during the construction phase, as appropriate. Areas of special inspection include but are not limited to:

- Isolator bolt tightness
- Levelness of pedestals and isolator units
- Levelness of upper and lower basemats
- Settlement of isolation system/distance between basemats at construction

9.3 Operation Considerations

9.3.1 In-service Inspection, Replacement and Maintenance

Nuclear facility structures should be designed to enable in-service inspection of isolator units. In consultation with isolator vendors, applicants should develop an appropriate inspection plan that ensures that performance criteria consistent with the NPP design and operational assumptions are met. This plan should include both criteria and methods to be used for the in-service inspection program. This plan should include periodic verification of mechanical properties of isolator devices to assess the effect of ageing on their performance. To this end, nuclear facility owners should maintain an adequate number of non-load-bearing isolators in the environment that is essentially identical to that of the in-service units. A representative compressive load should be maintained on the non-load-bearing isolators. Elements that meet operational needs, such as access to the isolation layer, strong points for jacking, and ability of the superstructure mat to span over one (or more) isolator unit should be incorporated into the design.

Proper maintenance being essential for plant safety, the isolation system and its individual isolators would be subject to the NRC requirements for monitoring the effectiveness of maintenance at nuclear power plants (10CFR50.65).

9.3.2 Additional Seismic Monitoring Equipment

Regulatory Guide 1.12 (NRC, 1997) provides guidance on nuclear power plant instrumentation for earthquakes. A proposed update to RG 1.12 has been made available for public comment (NRC, 2016) as DG-1332. For seismically isolated facilities, modern 3-component digital seismic monitoring equipment should be placed at a minimum of 3 locations around the perimeter of the basemat to capture the acceleration response of the isolated superstructure. Instruments should be located on the foundation and in the free field to enable characterization of soil-foundation structure interaction and the effect of seismic isolation in terms of the transmission of earthquake demands from the foundation to the superstructure. Additional instruments should be deployed across the footprint and height of the isolated NPP to characterize the response of the NPP and demands on SSCs.

9.3.3 Monitoring of Foundation Deformations

Long-term foundation deformations should be periodically monitored, including movement of the moat walls, relative displacements between the basemat and foundation, and vertical displacements of the basemat and foundations.

9.3.4 Requirements for Safety-related Equipment

Because SI equipment is a critical part of the foundation of a structure, every part of the SI layer itself should be treated as a safety-related system. The fire suppression system that protects the seismic isolators should be considered safety-related equipment.

9.3.5 Operating Temperature

Seismic isolators should generally be installed in a dry air-conditioned space that is maintained at a temperature of between 40ºF and 80ºF, unless an alternate temperature range was assumed and accounted for in the analysis, design and testing of the isolators and isolation system.

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Ketter Hall, Buffalo,

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11. ABSTRACT (200 words or less)

Seismic isolation (SI) is a technology that has proven useful in constructing structures capable of withstanding intense earthquake ground motions. Although various SI techniques have been known for centuries, the global use of modern SI devices greatly expanded in recent decades. Seismically isolated structures now number in the thousands around the world and the technology has been shown to meet the performance needs of important non-nuclear structures. Seismic isolation also has been used in the design and construction of nuclear power plants (NPPs) in France and South Africa and was recently used to isolate new emergency response centers at NPPs in Japan. As a result, SI has been considered for nuclear facilities in the United States.

This report was developed under the NRC's Seismic Research Program Plan with the goal of providing technical information for development of regulatory guidance on the use of SI technology. This report develops a performance-based and risk-informed design philosophy for SI informed by NRC safety goals and regulatory approaches. This design philosophy, in turn, leads to a set of recommended performance objectives and criteria that can serve as the foundation for development of future NRC guidance on the use of SI and related technology.

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