# Primer on Seismic Isolation

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# PRIMER ON SEISMIC ISOLATION

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Task Committee on Seismic Isolation
Seismic Effects Committee
Dynamic Effects TA Committee
Structural Engineering Institute (SEI) of
the American Society of Civil Engineers

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# Contents

# Preface

The purpose of this primer is to provide structural engineers and other design professionals with an understanding of the basic concepts of seismic isolation, to describe the current state of seismic isolation technology and to highlight issues and concerns that are unique to the design of isolated structures. The primer was prepared by the members of the ASCE Task Committee on Seismic Isolation, who represent a wide range of structural engineering firms and academic institutions and who were committed during the editing process to presenting a balanced view of seismic isolation.

By definition, a primer is an overview document. As such, this document is arranged in short chapters, each chapter providing a review of a topic related to seismic isolation. Extensive references are provided for readers who wish to find out more about a particular topic.

Chapter 1 includes a brief discussion of the principles of isolation, a brief history of isolation, representative case histories of isolated structures, descriptions of common isolation systems currently in use, and a review of the performance of isolation systems in past earthquakes. Chapter 2 covers the fundamentals of designing isolation systems, including an overview of governing design codes, the design of isolated buildings, bridges and other civil structures, and a review of issues related to selection of earthquake ground motions for design. Chapter 3 focuses on methods of analysis, including mechanical characteristics of various isolation systems, and methods of modeling the isolation system and superstructure. Chapter 4 covers testing of seismic isolation systems, including testing procedures, practical limitations on testing, and discussion of shake table testing and long-term performance of isolators.

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# 1. Fundamentals of Seismic Isolation

## 1.1 PRINCIPLES OF SEISMIC ISOLATION

A seismic isolation system may be defined as a flexible or sliding interface positioned between a structure and its foundation, for the purpose of decoupling the horizontal motions of the ground from the horizontal motions of the structure, thereby reducing earthquake damage to the structure and its contents.

Structures are not normally isolated from vertical earthquake motions. Generally speaking, vertical ground motions are of a smaller magnitude than horizontal motions. In addition, because structures must be designed to resist static gravity loads they are inherently strong and stiff in the vertical direction, making isolation in the vertical direction of secondary importance.

Seismic isolation can have two advantageous effects on the seismic response of a structure: reduction of lateral forces in the superstructure, and concentration of lateral displacements at the isolation interface.

The first effect is illustrated in Figure 1-1 which shows two smoothed acceleration response spectra: the upper spectrum is for damping in the fixed base structure and the lower spectrum is for damping in the isolated structure; note that the spectrum corresponding to damping in isolated structure is lower because of higher damping provided by the isolation system compared to the fixed base system. The first mode period $T_f$ of a fixed base structure is shown by a vertical line on the left; the first mode period $T_i$ of the isolated structure is shown by a vertical line on the right. The isolation system lengthens the fundamental period of the structure, and adds damping. Both of these effects reduce the acceleration response of the structure, and consequently the lateral forces in the structure.

The second effect of seismic isolation is illustrated in Figure 1-2. Figure 1-2(a) shows that for a fixed base structure, lateral seismic displacements are distributed over the height of the structure. The relative displacement between adjacent floors, or interstory displacement, can result in both structural and non-structural damage. The magnitude of interstory displacements depends on the combination of the first several predominant modes of vibration of the structure. Figure 1-2(b) shows that for an isolated structure, the predominant mode of vibration is that related to displacement of the isolation system. The structure above the isolation system tends to move as a rigid body, and interstory displacements within the superstructure are greatly reduced. In other words, the presence of the isolation system concentrates lateral displacements at the isolation interface, and minimizes lateral displacements in the superstructure.

While the above explanations of the principles of seismic isolation are simplified, they do describe in basic terms how isolation can improve the seismic performance of structures. There are a number of additional aspects of behavior that must be addressed in the detailed design of an isolated structure. These include the potential for torsional response of the isolated structure; the effects of variations in ground motion characteristics on the response of the structure, including near-fault phenomena and long-period motions associated with sites with deep soil deposits; the influence of wear, aging and temperature effects on the behavior of the isolation system; the potential for uplift of isolators caused by overturning of the superstructure; the potential response amplification for structures with low fixed-base natural periods at the rising branch of the acceleration response spectrum, and the need to provide flexible utility connections.

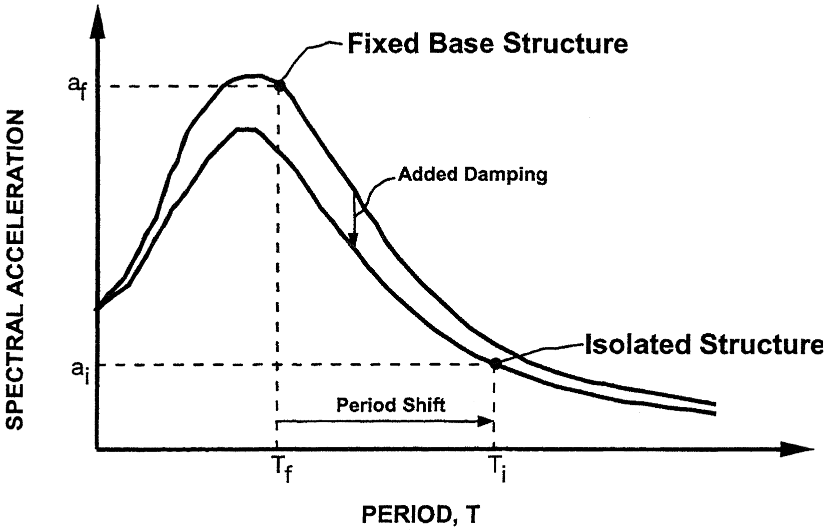


**Figure 1-1.** Response of a fixed base and isolated structure

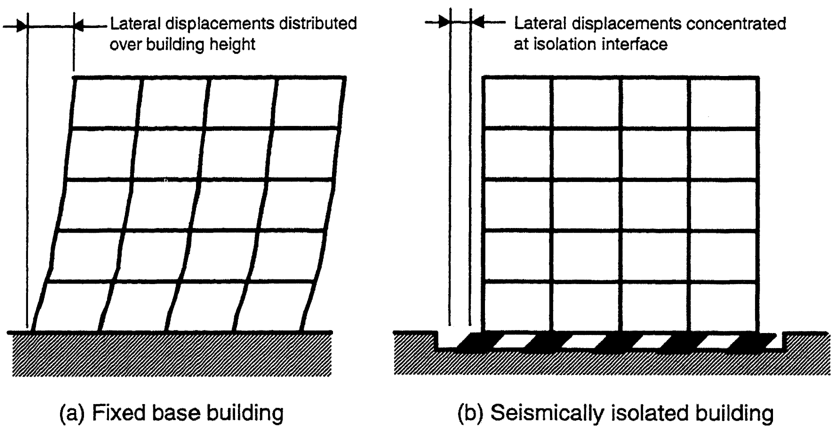


**Figure 1-2.** Lateral displacements in a fixed base and isolated structure

**Figure 1-3.** Foothill Communities Law and Justice Center (photo courtesy DIS, Inc.)

## 1.2 HISTORY OF SEISMIC ISOLATION

Although the fundamental principle of seismic isolation has been understood for at least a century, it is only in the last four decades that practical systems for achieving seismic isolation have become available, and only during the last decade has seismic isolation has been widely adopted. In this section an overview of the history of seismic isolation is presented, providing background on the development of current isolation technology.

Kelly (1996) notes several early examples of buildings with seismic isolation. These include two buildings constructed on rollers, one in Savastopol, Ukraine and the other in Mexico, and a building in China with a layer of sand between the foundation and the building, intended to permit sliding during an earthquake. Eisenberg (1992) describes a building constructed in 1959 in Ashkhabad, Turkmenistan that is suspended by cables so that it acts as a pendulum. The first building to utilize a rubber isolation system was a three-story school building constructed in 1969 in Skopje, Yugoslavia. The building rests on solid blocks of rubber, which do not contain internal horizontal steel reinforcing plates, as would be the practice today. In 1978 the first structure to utilize an isolation system with added damping was the Toetoe Viaduct in the North Island of New Zealand (Skinner, et al. 1993). The isolation system consists of laminated steel and rubber bearings incorporating a specially formulated high damping natural rubber; it also contains a central lead core for energy dissipation. This type of isolation system is now of wide use, and is commonly referred to as the Lead Rubber Bearing (LRB) isolation system.

The first building to use the LRB isolation system was the William Clayton Building in Wellington, New Zealand, completed in 1981.

The first seismically isolated building in the United States was the Foothill Communities Law and Justice Center (FCLJC) (Figure 1-3) in Rancho Cucamonga, California, constructed in 1984-85. The building is located about 20 km (12 miles) west of the San Andreas Fault.

Although the FCLJC building represented the first use of seismic isolation in the United States, and demonstrated the practicality and economy of seismic isolation as a means of protecting structures from earthquake damage, the FCLJC project was not quickly followed by other isolation projects. This was mainly due to a lack of official building code provisions governing the design of isolated structures. Without such provisions designers, owners and building officials were reluctant to proceed with isolation projects. In 1986 a committee of the Structural Engineers Association of Northern California (SEAONC) issued *Tentative Seismic Isolation Design Requirements* (SEAOC 1986). These provisions, along with subsequent revised and expanded provisions in the *SEAOC Blue Book* (SEAOC 1990, 1996), the *Uniform Building Code* (ICBO 1991, 1994, 1997) and *NEHRP Provisions* (NEHRP 1995, 1997) paved the way for implementation of seismic isolation in the United States. In the following section a number of examples of seismically isolated structures in the United States are presented. These illustrate the range of application of seismic isolation - both in new structures and existing structures - which has been achieved since the 1984-85 FCLJC project.

## 1.3 EXAMPLES OF SEISMICALLY ISOLATED STRUCTURES

No attempt is made here to provide a comprehensive listing of seismically isolated structures in the United States, simply because the list is growing rapidly and would be obsolete shortly after publication. (For a current listing of seismically isolated structures worldwide, see http://www.eerc.berkeley.edu/prosys/applications.html). However, it can be noted that to date at least 1,000 structures (buildings, bridges and tanks) have been seismically isolated around the world, and approximately 150 of these are in the United States. Following are descriptions of several structures representative of typical isolation projects undertaken in the United States and abroad. The projects listed are intended to illustrate a range of isolation system and superstructure types.

An example of an isolated hospital building is the University of Southern California Hospital in Los Angeles (Figure 1-4). This facility is a 275-bed, 32,500 m$^2$ (350,000 ft$^2$) teaching hospital at the USC School of Medicine. The structure is seven stories tall, plus a basement, and consists of a steel braced frame situated on 68 lead rubber bearings and 81 rubber bearings. This building is notable because it was subjected to the January 17, 1994 Northridge earthquake. The hospital survived without damage to the structure or contents, and it was fully operational following the earthquake. The performance of the hospital is described in more detail in Section 1.5

An example of an isolated emergency center is the Washington State Emergency Operations Center (EOC) located at Camp Murray, near Tacoma, Washington (Figure 1-5). The EOC serves as the communications and logistical hub for disaster response activities in the State of Washington. The two-story, 2040 $m^2$ (22,000 $ft^2$) building was constructed in 1998, and consists of a steel braced frame supported on 22 friction pendulum system (FPS) isolators. The state government

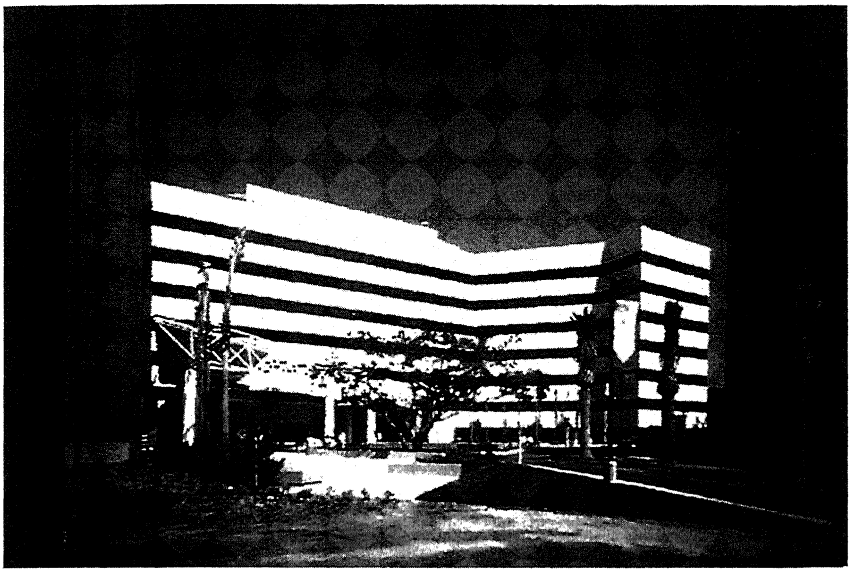

**Figure 1-4.** University of Southern California Hospital (photo courtesy DIS, Inc.)

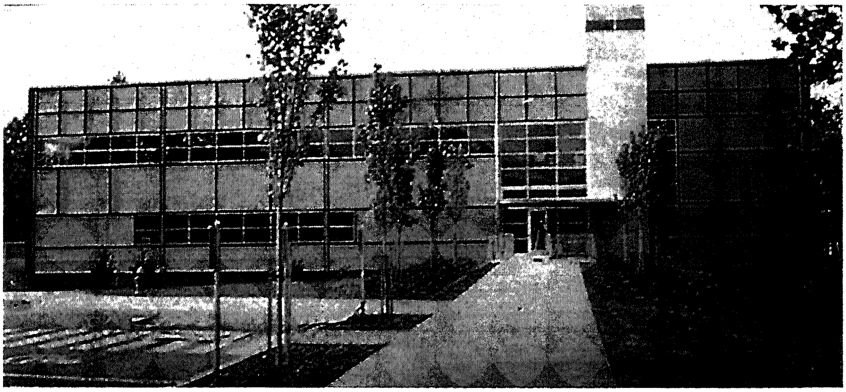

**Figure 1-5.** Washington State Emergency Operations Center (photo courtesy Earthquake Protection Systems, Inc.)

required that the EOC be designed to be fully operational within minutes after a major natural disaster, including an earthquake. Hence, seismic isolation was selected to achieve full functionality after a one-in-975-year event.

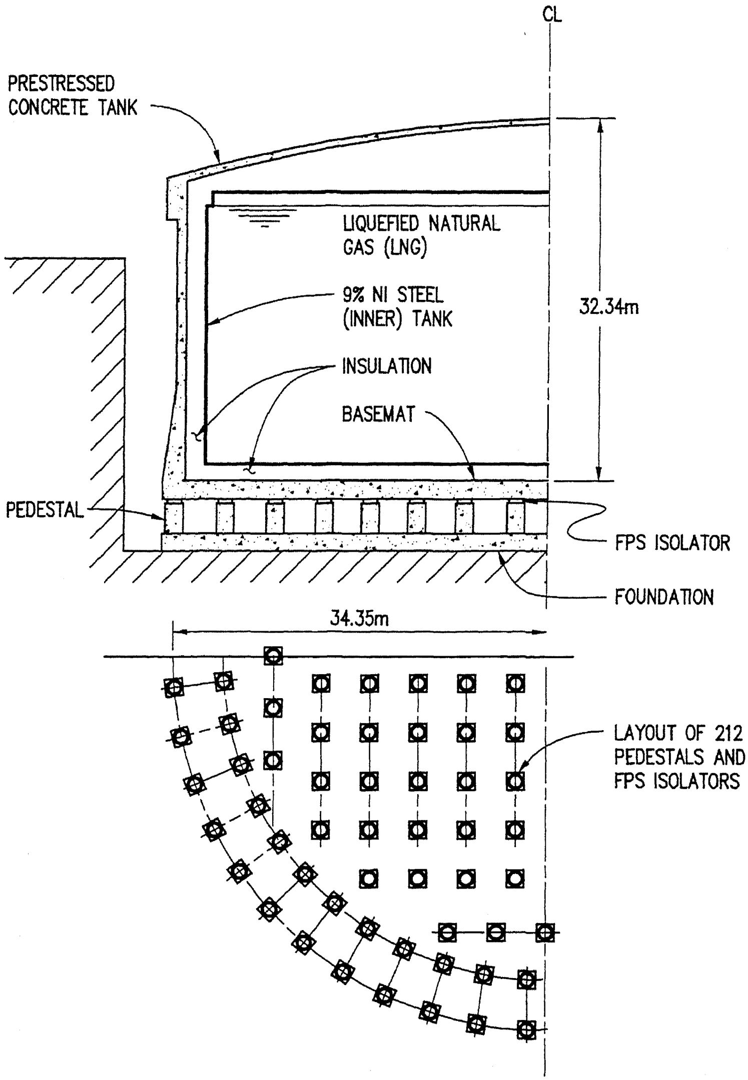


**Figure 1-6.** Cross section of liquefied natural gas (LNG) tank, Revithoussa, Greece (figure courtesy M.C. Constantinou)

Tanks containing hazardous or critical materials have also been constructed on seismic isolation bearings. The largest seismically isolated tanks in the world are two liquefied natural gas (LNG) tanks in Revithoussa, Greece, owned by the Public Gas Corporation of Greece. The tanks are of the full containment type with a 9% nickel steel unanchored inner tank of 65.7 m diameter and 22.5 m height and an outer prestressed concrete tank. The tanks are partially buried for reasons of aesthetics. Figure 1-6 presents the principal features of one of the tanks.The isolation system of each tank consists of 212 isolators that are supported on pedestals resting on the foundation slab. The isolation pit with a free height of 2.0 m allows for access to the isolation system for inspection and replacement of isolators if necessary. A pit ventilation system ensures a vapor free atmosphere and controls the temperature within the pit and beneath the base slab.

The isolation bearings are friction pendulum bearings with a radius of curvature of 1,880 mm and displacement capacity of 300 mm. The design of isolated tanks was based on analysis in which the likely upper and lower bound properties of the isolators were considered. For the determination of the bounding properties a procedure similar to the λ-factor approach of the 1999 AASHTO Guide Specifications for Seismic Isolation Design was utilized. The effects of temperature and aging on properties, the variability of properties (around the mean tested values) and the differences in properties between the first and subsequent cycles of movement were considered in establishing the bounding values. Dynamic analysis of the isolated tanks was performed with computer code 3D-BASIS-ME (Tsopelas, et al.

**Figure 1-7.** Overhead view of isolators, liquefied natural gas (LNG) tank, Revithoussa, Greece (photo courtesy M.C. Constantinou)

1994), which was modified for this project to include the effects of the vertical ground acceleration. The earthquake was defined in terms of elastic 5-percent damped spectra for the conditions of operating bases (OBE) and safe shutdown (SSE). The latter has a peak ground acceleration of 0.48g and spectral values of 0.61g and 0.29g at periods of 1.0 and 3.0 sec., respectively. The design called for totally elastic behavior in the inner tank under the conditions of the Safe Shutdown Earthquake.

Of interest in this project is the existence of two alternative preliminary designs, one non-isolated and the other isolated. The non-isolated alternative was of the same inner tank geometry and with massive anchors attached to both the outer and the inner tanks. The inner tank had a thicker shell and special detailing was provided to minimize thermal effects. While it was determined to be the alternative with the least initial cost, the owner opted for the isolated alternative because it was perceived to be a safer design with potential for lesser life-cycle cost.

Installation of the isolation bearings took place in 1995. In 1997, the outer tanks were completed and the inner tanks were under construction. The tanks became operational in late 1999. Figure 1-7 shows a view of the construction site following installation of the isolators.

Numerous bridges in the United States and abroad have been protected with seismic isolation systems. Seismic isolation has been used both for new construction and to retrofit existing bridges. A typical example of a retrofitted bridge is the Eel River Bridge, which carries U.S. Route 101 over the Eel River near Rio Dell, California (Figure 1-8). The original superstructure consisted of two 92 m (303 ft) long simple span steel through trusses supported on steel bearings. To protect the seismically vulnerable non-ductile concrete wall piers, the steel bearings were replaced with lead rubber isolation bearings in 1987. In April 1992 the bridge was

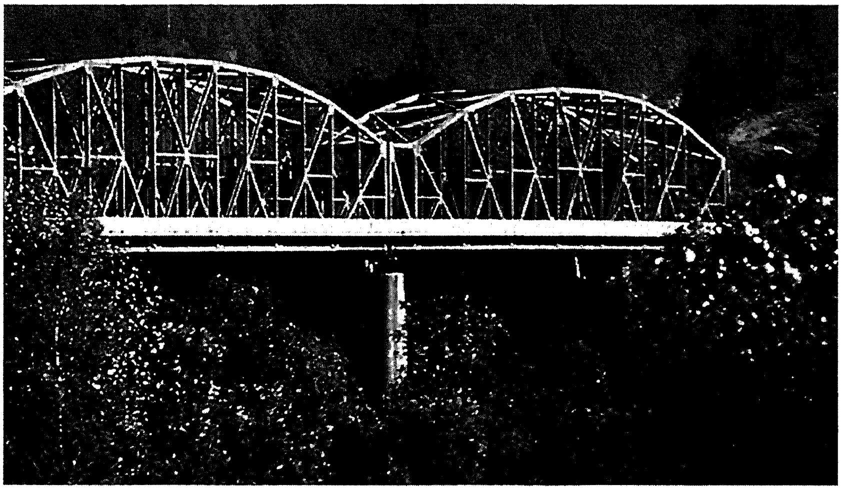

**Figure 1-8.** Eel River Bridge (photo courtesy DIS, Inc.)

subjected to the Cape Mendocino (Petrolia) earthquakes. The bridge remained in service after the earthquake. The seismic performance of the bridge is described further in Section 1.5.

Historic buildings with elaborate architectural details and finishes are often candidates for seismic isolation, since traditional seismic strengthening methods would require alterations of the superstructure that would damage historic features. The 3-story, 32,500 $m^2$ (350,000 $ft^2$) U.S. Court of Appeals building in San Francisco is an ornate Federal courthouse with granite exterior, marble interiors, decorative plaster and hardwood details (Figure 1-9). The courthouse was retrofitted in 1994 with 256 friction pendulum system (FPS) isolators. By retrofitting the building at the basement level, the historic superstructure was not disturbed.

Another common application of seismic isolation is to protect high technology manufacturing facilities, such as semiconductor manufacturing plants. These plants can house equipment worth billions of dollars, and the costs of disruption of the manufacturing process can be in millions of dollars a day. One example of such a plant is the Rockwell Semiconductor Systems Wafer Fabrication Facility in Newport Beach, California, shown in Figure 1-10. The existing 2-story 240,000 square foot building was retrofitted with 192 lead-core elastomeric isolators. The facility remained in operation during excavation beneath the building and installation of the isolators. A system of electronic instruments was put in place to monitor vibrations during the retrofit process. If vibrations exceeded pre-established limits, construction was stopped to avoid interference with the wafer manufacturing process.

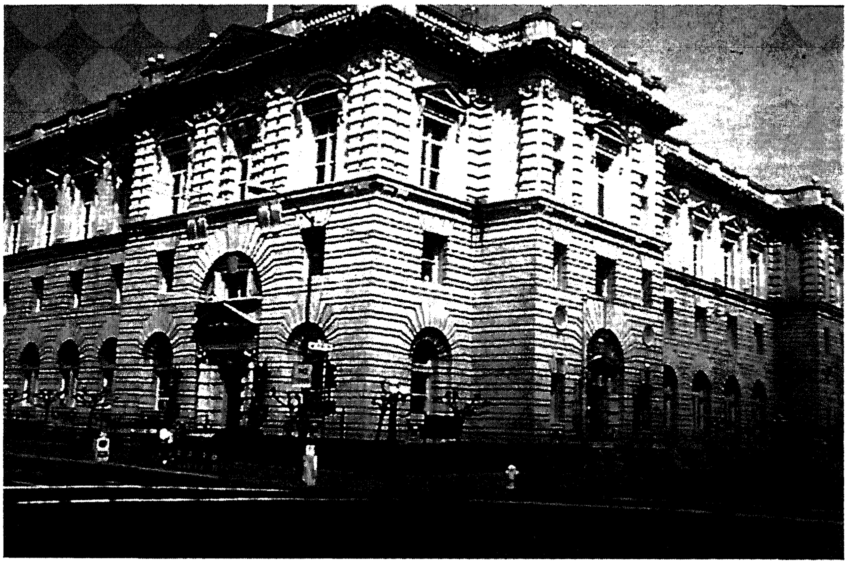

**Figure 1-9.** U.S. Court of Appeals building, San Francisco (photo courtesy Earthquake Protection Systems, Inc.)

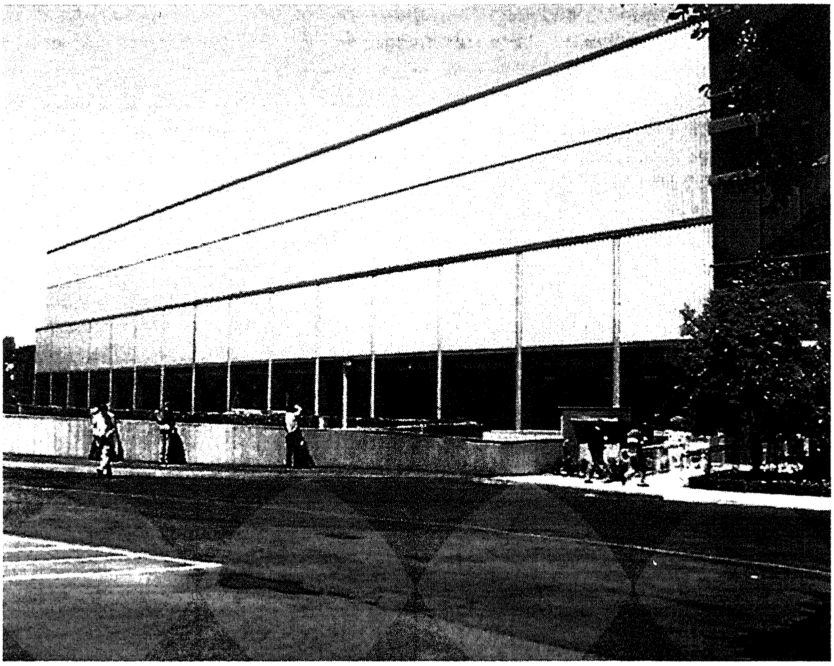

**Figure 1-10.** Conexant Systems Building 503 Manufacturing Plant Seismic Retrofit, Newport Beach, California (photo courtesy KPFF Consulting Engineers).

Several city government buildings in California have recently been retrofitted with seismic isolation devices, including the Los Angeles City Hall, the Hayward City Hall, the San Francisco City Hall and the Oakland City Hall. As an example, the Oakland City hall retrofit is described here, Figure 1-11. The Oakland City hall was constructed in 1914, and is currently registered as a National Historic Landmark. The building is 18 stories tall (the second tallest isolated building in the world, after the Los Angeles City Hall), consisting of a three-story podium, a ten-story office tower and a two-story clock tower base that supports a 28 m (91 foot) high ornamental clock. The floor area of the building is 14,200 m$^2$ (153,000 ft$^2$). The structure is a riveted steel frame with perimeter unreinforced masonry infill panels. The Oakland City Hall was damaged during the October 1989 Loma Prieta earthquake, and the city was forced to vacate the building until it could be repaired and upgraded. The retrofit isolation system was completed in 1995, and consists of 42 lead rubber bearings and 69 rubber bearings. A moat was constructed around the building to provide a seismic gap of 510 mm (20 in.). Installation of the isolators proved to be very complicated and required shoring of the columns, cutting of the columns, and transferring of the column loads to temporary supports. To protect the interior architectural finishes, the columns were raised by not more that 2.5 mm (0.1 in.) during the jacking process.

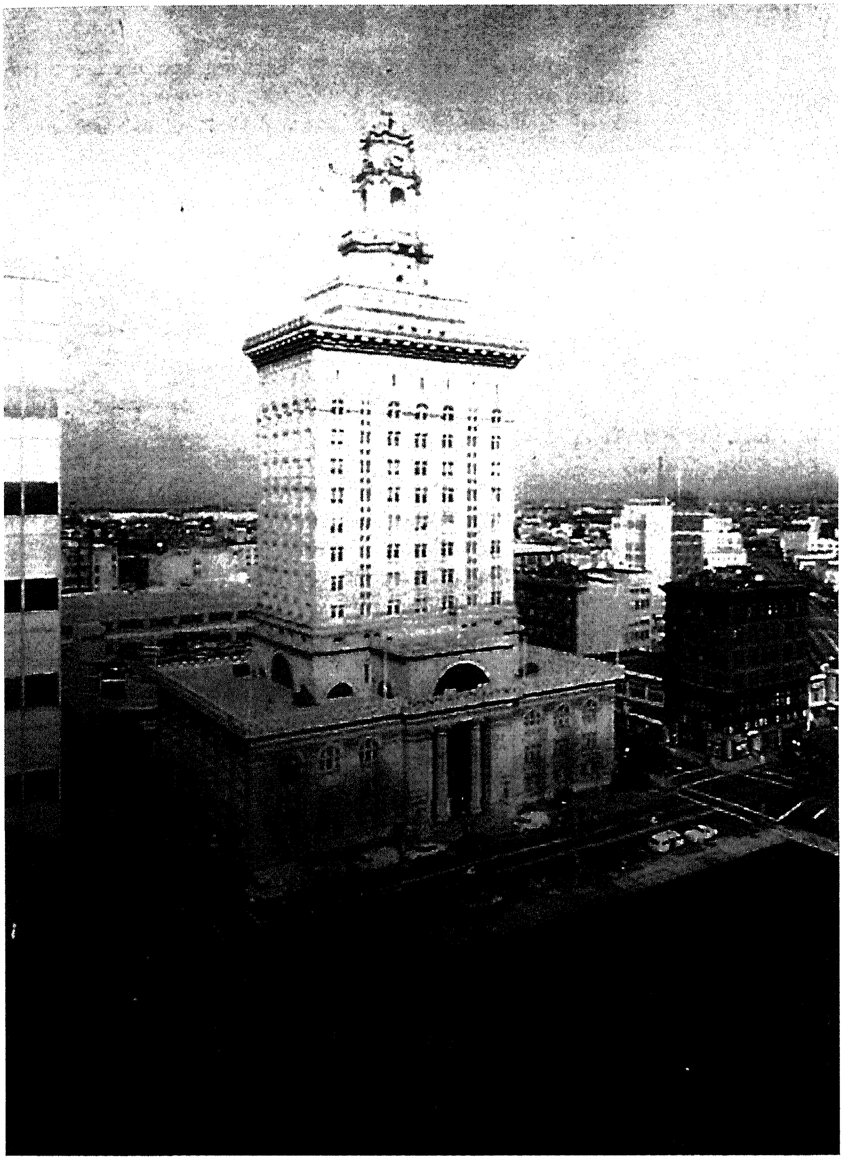

**Figure 1-11.** Oakland City Hall (Photo courtesy DIS, Inc.)

## 1.4 DESCRIPTIONS OF COMMON ISOLATION SYSTEMS

In this section the most common types of seismic isolation systems are briefly described. The emphasis is placed on isolation systems that are currently in common usage. Broadly speaking, isolation systems can be placed in two categories: elastomeric (or rubber) systems and sliding systems. Within the category of elastomeric systems, the two most common types are high damping rubber isolators, and lead core isolators. Within the category of sliding systems the two most common categories are the friction pendulum system (FPS), and flat sliding systems without and with restoring force capability. A third category of isolator could be considered the hybrid system, in which elastomeric isolators are combined with flat sliding isolators. Recently, isolators have been used in conjunction with supplemental damping systems, such as fluid viscous (hydraulic piston) dampers. Each of the above mentioned isolation systems is described briefly in this section

### *1.4.1 Elastomeric isolators*

Figure 1-12 illustrates a typical elastomeric isolator. The isolator consists of a sandwich of rubber layers alternating with steel reinforcing layers, commonly called "shims". The rubber layers and shims are bonded together in a vulcanization process. The purpose of the shims is to restrict outward bulging of the rubber layers when the isolator is subject to axial load, thereby increasing the axial stiffness and load carrying capacity of the isolator, while having essentially no effect on the lateral stiffness of the isolator. This results in an isolator with a vertical stiffness that is orders of magnitude greater than the lateral stiffness.

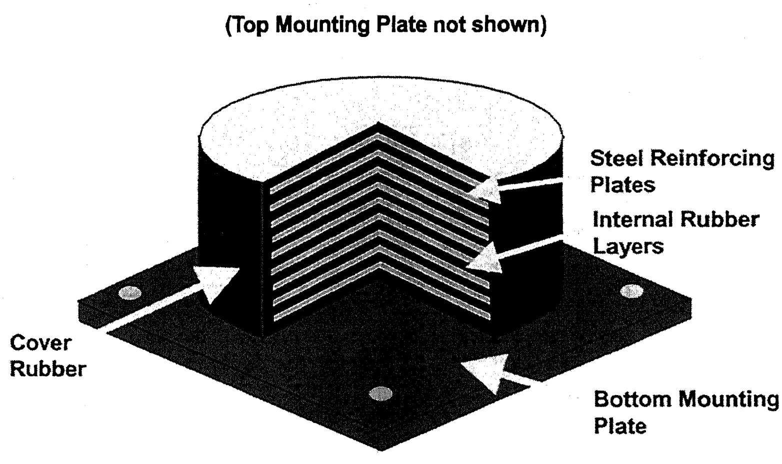


**Figure 1-12.** Typical elastomeric isolator (figure courtesy DIS, Inc.)

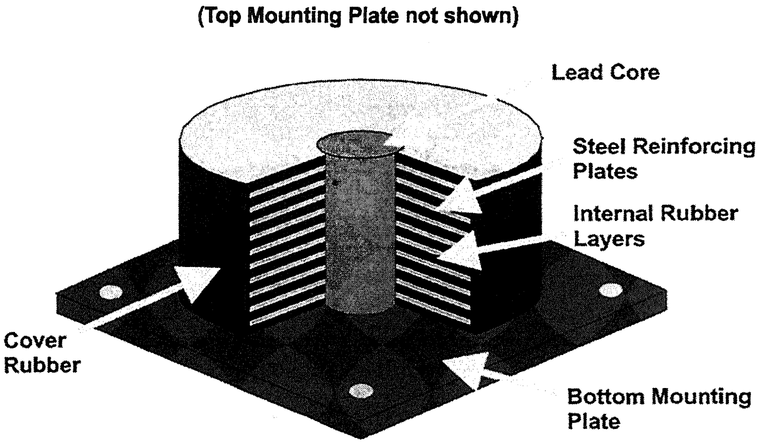


**Figure 1-13.** Lead core elastomeric isolator (figure courtesy DIS, Inc.)

During manufacture, the assembly is laid up in a mold using uncured rubber sheets and steel plates that have been treated to optimize chemical bonding of the steel and rubber. The assembly is then cured under pressure and heat to create the completed isolator unit. End mounting plates are either included in the original assembly process, or are added in a second stage of assembly.

The rubber used in elastomeric bearings is either a natural or synthetic rubber compound. The exact composition of the rubber compounds used by a particular manufacturer is normally proprietary, although the physical properties of the cured rubber and the mechanical properties of the completed isolator are determined by tests and supplied to the project engineer and owner.

Damping in elastomeric isolators is provided either through the use of a high damping rubber (HDR) compound, or by a lead core, as shown in Figure 1-13. The composition of high damping rubber compounds can be varied to achieve a range of damping values between 8 and 16 percent of critical. The level of damping in lead core isolators is controlled by varying the diameter of the lead core. Lead is used because of its elastic-plastic post-yield behavior, its capability to maintain strength during plastic deformation cycles, and the fact that damping values up to 30 percent can be achieved. It should be noted, however, that damping above a certain level is not necessarily beneficial. The threshold, above which there is no clear benefit to be gained from additional damping, is not sharply defined, and varies from structure to structure.

### *1.4.2 Sliding systems*

The simplest sliding isolator is the flat sliding system. Although many system details are possible, all flat sliding systems consist fundamentally of a stainless steel plate bearing against a low friction interface material such as polytetrafuoroethylene

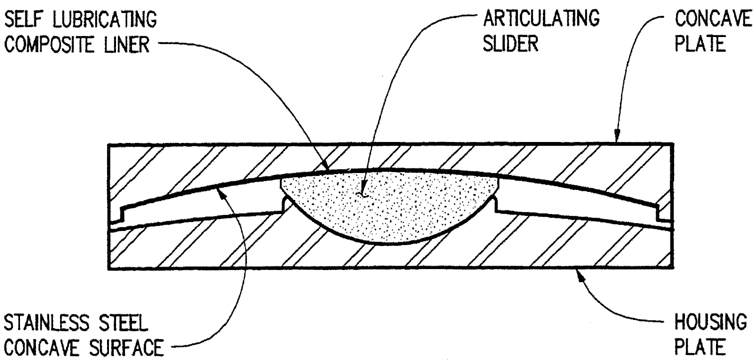


**Figure 1-14.** FPS isolator cross section (figure courtesy Earthquake Protection Systems, Inc.)

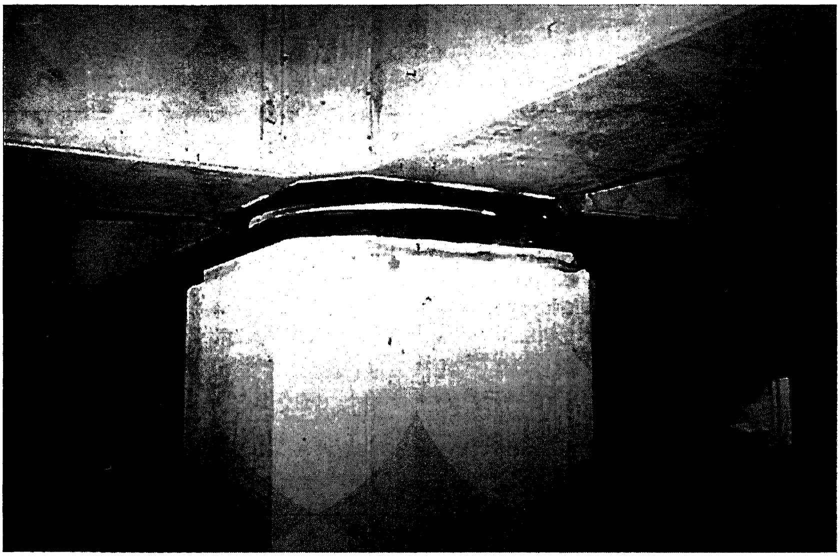

**Figure 1-15.** FPS isolator installed in Hayward City Hall (photo courtesy earthquake Protection Systems, Inc.)

(PTFE). Because flat sliding systems have no self-centering feature, supplemental centering devices or motion limiting stops must be employed.

In the United States a flat sliding isolation system has been employed that incorporates a motion-limiting ring surrounding the sliding surface. Several springs, made of a proprietary polymer, are positioned around the ring to cushion the impact of the slider against the ring, and to provide a re-centering force.

In Italy so-called "oleodynamic devices" have been used in conjunction with flat sliding isolators in bridges. These devices allow essentially unrestricted slow thermal movements of the bridge superstructure. Under high velocity seismic movement, however, the oleodynamic devices "lock up", providing energy dissipation through metal yielding, while still permitting the bridge superstructure to slide laterally on the sliding bearings. The force transmitted to the bridge substructure is limited by the yield capacity of the oleodynamic devices. A more detailed description of the application of oleodynamic devices may be found in Constantinou (1998).

The friction pendulum system (FPS) is illustrated in Figures 1-14 and 1-15. The assembly consists of a polished stainless steel spherical concave surface, an articulated slider, and a low friction composite liner. During an earthquake the articulated slider moves along the concave surface, causing the superstructure to move in a pendulum motion. The period of oscillation is a function of the radius of the concave surface and is independent of the mass of the superstructure. Damping is provided by the dynamic friction force, which can be varied by adjusting the properties of the low friction composite liner material. Typical damping values are between 10% to 30% of critical.

### *1.4.3 Hybrid systems*

A hybrid isolation system is one that consists of a mixture of different isolator types. The most common hybrid system is a mixture of elastomeric isolators and flat sliding isolators. The sliding isolators are typically used at a few relatively lightly loaded locations in the structure where earthquake-overturning loads are negligible. Used in this manner, they have little impact on any characteristic of the isolation system other than the total cost.

More recently, sliding isolators have been used more frequently for lower load applications. By reducing the overall stiffness of the isolation system, buildings with a relatively low dead load per column can be isolated to the same performance level as heavier structures.

Flat elastomeric-backed sliding isolators have been successfully used as one component of a hybrid isolation system in conjunction with lead-rubber isolators. The lead-rubber isolators are positioned around the perimeter of the system to help minimize torsional movement of the isolation system during an earthquake. Conventional sliding systems, which exhibit nonlinearities associated with high initial stiffness, can pass high frequency floor accelerations (on the order of 10 Hz or higher) through to superstructure. This effect is counteracted in sliding systems by providing an elastomeric backing to support the sliding interface, resulting in a lower isolator initial stiffness. Nonlinearities can also be minimized by using low coefficient of friction sliding materials (on the order of 2 to 4 percent).

One issue for mixed elastomeric and sliding isolation systems is the potential for load transfer during horizontal displacement. Elastomeric isolators typically shorten slightly at the maximum shear displacement, even under zero applied vertical load. This can result in significant load transfer to adjacent sliding isolators that do not exhibit this behavior.

### *1.4.4 Isolators in conjunction with dampers*

In situations where extreme isolator displacements are anticipated it may be advantageous to incorporate supplemental damping devices with the isolation system. There are no generic rules that can be used for the application of supplemental damping with isolators; the need for and design of supplemental damping must be assessed on a project-by-project basis.

An example of a project incorporating both isolators and dampers is the Arrowhead Regional Medical Center in Colton, California, shown in Figure 1-16. The medical center consists of five seismically isolated buildings, with a total floor area of 85,500 $m^2$ (920,000 $ft^2$). The site is located within 3 km (2 miles) of the San Jacinto Fault, and 14 km (9 miles) of the San Andreas Fault, potentially two of the most active faults in California. As a result the design ground motions for this project were severe. However, it was found that by incorporating hydraulic piston dampers in tandem with high damping rubber isolators, as shown in Figure 1-17, the lateral displacements of the structure in the maximum considered earthquake (MCE) could be limited to plus or minus 560 mm (22 inches). A total of 400 high damping rubber (HDR) isolators were used in this project in conjunction with 184 fluid viscous (hydraulic piston) dampers.

## 1.5 PERFORMANCE OF ISOLATED STRUCTURES IN PAST EARTHQUAKES

Described below are several seismically isolated structures that have been subjected to earthquake shaking. The examples are listed in chronological order of the earthquake events.

An isolated bridge that was subjected to strong ground shaking was the Te Teko Bridge in New Zealand (Dowrick 1988). This bridge is 103 m long, with 5-spans of prestressed concrete U-beams. Lead-rubber bearings are at each pier and plain rubber bearings are at each abutment. The bridge is located 9.6 km (6 miles) from the epicenter of the 1987 Edgecumbe earthquake. This earthquake devastated a small farming community, and closed a major pulp and paper mill for 18 months. The bridge itself was not instrumented, but the peak ground acceleration at the bridge site was estimated to be between 0.35g and 0.40g, and the bridge displacements were estimated to be on the order of 100 mm (4 inches). Although one isolator displaced out of its retaining ring, the bridge was operational after the earthquake.

The Mark II Detector, at the Stanford Linear Acceleration Center, was 51.5 km (32 miles) from the epicenter of the 1989 Loma Prieta earthquake (DIS 1996). A peak ground acceleration of 0.29g was recorded on the nearby Stanford University campus. The Detector itself was not instrumented, but movement of the Detector was estimated to be 100 mm (4 inches), and there was no damage. The Detector, which consists of fragile equipment and weighs about 1500 tons, is about 9 m high with an 8 m by 11 m footprint. It was isolated using four lead-rubber bearings, each 740 mm square and 330 mm high, with 240 mm diameter lead plugs.

The most significant case of an isolated bridge undergoing strong ground

**Figure 1-16.** Arrowhead Regional Medical Center (photo courtesy KPFF Consulting Engineers)

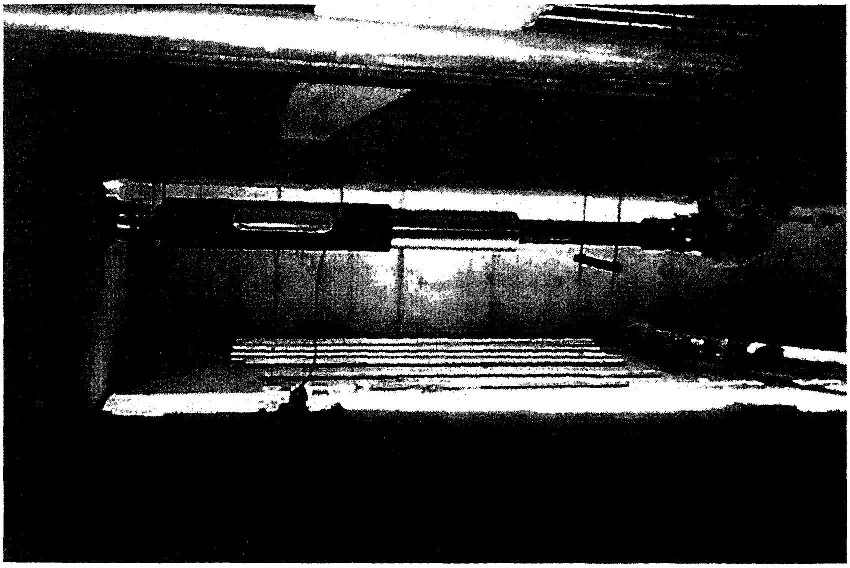

**Figure 1-17.** One of the hydraulic piston dampers used in the Arrowhead Regional Medical Center (photo courtesy Taylor Devices, Inc.)

shaking is the Eel River Bridge, described previously. Although the bridge was not instrumented, the peak ground acceleration measured at another nearby site in the April 25-26, 1992 Petrolia earthquakes was 0.55g. Displacements at the bridge were estimated to be 200 mm (8 inches) longitudinally and 100 mm (4 inches) transversely. The bridge performed well and remained in service.

The most significant case of an isolated building undergoing strong ground shaking is the USC Hospital, described above, which was subjected to the January 17, 1994 Northridge earthquake. The performance of this building was reviewed by Asher et al. (1997). It was found that an analytical model of the structure matched well with the recorded behavior of the structure. The measured free field peak ground acceleration in the January 1994 Northridge Earthquake was 0.49g, while the peak ground acceleration throughout most of the structure was less than 0.13g, and the peak acceleration at the roof was 0.21g due to structural amplification in the upper two stories. By analyzing a model of the hospital without isolation, Asher et al. concluded that accelerations throughout the fixed base structure would have ranged between 0.37g and 1.03g, and that damage to building contents and disruption of service would have been almost certain.

The West Japan Postal Savings Computer Center experienced ground motions with a peak site acceleration of 0.40g in the January 17, 1995 Kobe earthquake (DIS 1996). The Computer Center exhibited no damage, and the maximum recorded acceleration in the building was 0.12g. A nearby fixed base building of approximately the same height was also instrumented, and the maximum recorded acceleration at the roof was 1.18g.

Shimizu Corporation's office building, in Tsuchira, Japan is fully instrumented and has been subjected to 63 different earthquakes with magnitudes varying between 4.2 and 6.8 (DIS 1996). The two largest events produced peak ground accelerations of 0.27g and 0.22g at the site. In these two events floor accelerations in the building were reduced by a factor of 2.5, compared to the input ground motions.

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# 2. Design of Isolated Structures

## 2.1 GOVERNING DESIGN CODES

### *2.1.1 Brief history of code development*

The development of building code provisions for seismic isolation of new buildings in the United States is presented in Chapter 9 of Federal Emergency Management Agency report FEMA 274 (FEMA 1997b). This history is summarized here.

The first attempt to codify the design of seismic isolation systems in the United States was made in the mid 1980's by the Northern Section of the Structural Engineers Association of California (SEAOC). This effort resulted in the Northern Section publishing in 1986 *Tentative Seismic Isolation Design Requirements* (SEAOC 1986). Over the next few years these provisions were revised and expanded by the Seismology Committee of SEAOC. In 1990 SEAOC published "General Requirements for the Design and Construction of Seismic-Isolated Structures," as Appendix 1L of the *SEAOC Blue Book* (SEAOC 1990).

The 1990 SEAOC provisions were adopted, with minor changes, by the International Conference of Building Officials (ICBO) in the 1991 *Uniform Building Code* (ICBO 1991). The provisions were not mandatory, but appeared as an appendix to Chapter 23 of the *UBC*. The SEAOC Seismology Committee continued to revise its version of the seismic isolation provisions, and these have appeared in the subsequent edition of the *SEAOC Blue Book* (SEAOC 1996). Similarly, ICBO has published revised versions of seismic isolation provisions in subsequent editions of the *UBC*. These provisions are similar to, but not identical to the SEAOC provisions.

A third parallel program to develop guidelines for design of seismic isolation systems was started in 1992. The National Earthquake Hazard Reduction Program (NEHRP) published the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings* (NEHRP 1998a, 1998b). Technical Subcommittee TS-12 was formed by the Building Seismic Safety Council to develop provisions for base isolation and energy dissipation systems for inclusion in the 1994 edition of the *NEHRP Recommended Provisions*. The provisions developed by the committee were based largely on the 1994 *UBC* provisions, but the *UBC* Provisions were converted to the strength design approach used in the *NEHRP Recommended Provisions*. Over the following two years a concerted effort was made to resolve any differences in the seismic isolation provisions found in the *UBC* and the *NEHRP Recommended Provisions*. The seismic isolation provisions in the 1997 editions of these two documents are essentially identical, as both documents have been coordinated and are now based on a strength design approach.

### *2.1.2 Current codes and resource documents*

In this section current codes and resource documents for the design of isolated structures are summarized.

*International Building Code (IBC)*: As mentioned earlier, the seismic isolation provisions found in the 1997 edition of the *Uniform Building Code (UBC)* (ICBO 1997) are nearly identical to those published in the 1997 edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings*. The *UBC* ceased publication with the 1997 edition, and was replaced by the *IBC* in 2000. The latest edition of the *IBC* (2003) was published in 2003.

*NEHRP Recommended Provisions for Seismic Regulations for New Buildings*: The latest edition of the *NEHRP Recommended Provisions* is the 2003 edition (NEHRP 2003a, 2003b). This edition contains seismic isolation provisions essentially identical to those found in the 1997 *UBC*.

*SEAOC Blue Book*: The most recent edition of the *SEAOC Blue Book* (SEAOC 1999) contains provisions that are similar to those found in the 1997 edition of the *UBC* (ICBO 1997).

*AASHTO Code*: In 1995 Task Group T-3 was formed by the American Association of State Highway and Transportation Officials (AASHTO) to develop new provisions for the design of seismically isolated bridges. The work of this committee represents a departure in some respects from previous UBC/NEHRP/SEAOC provisions for the design of seismically isolated buildings. This is because some aspects of bridge design are fundamentally different from building design. It is also because Task Group T-3 developed innovations in some areas. In particular, the group developed provisions to include the effects of environmental and materials factors on the performance of isolators. These include temperature, aging, contamination, wear, frequency, velocity, and scragging (first excursion vs. subsequent excursions stiffness). The latest edition of the *AASHTO Code* (1999) was published in 1999.

*FEMA 356*: In 1997 the Federal Emergency Management Agency (FEMA) published two guideline documents that address the seismic rehabilitation of existing buildings: FEMA-273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 1997a), and FEMA-274 *NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings* (FEMA 1997b). One option for seismic rehabilitation presented in FEMA 273/274 is retrofitting a building with a seismic isolation system. The philosophy followed in FEMA 273/274 for design of retrofitted seismic isolation systems is similar to that found in the UBC/NEHRP/SEAOC provisions for isolation systems in new buildings. However, the FEMA 273/274 provisions are written from a performance-based perspective that allows a selection of performance objectives (e.g. immediate occupancy, preservation of life safety, collapse prevention) not addressed by the UBC/NEHRP/SEAOC provisions. The successor guideline to FEMA 273/274 is FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA 2000).

It should be noted that the codes and guidelines listed above contain requirements for third party review of isolation designs. The peer review process should be recognized early in the planning phases of an isolation project, as experience has shown that coordination of the review process adds complexity, and potentially time, to the design schedule.

*ASCE Standard for Testing Seismic Isolation Systems*: In 1996 a committee was formed by the American Society of Civil Engineers to develop code provisions for testing seismic isolation systems. The committee used as a resource document an earlier guideline on testing developed at the National Institute of Standards and Technology (NIST 1996). The standard will include procedures for basic property testing, prototype testing and quality control testing of both elastomeric and sliding isolation systems.

## 2.2 DESIGN OF ISOLATED BUILDINGS

### *2.2.1 Superstructure design*

The design of the superstructures of fixed-base and seismically isolated buildings is very similar in many respects. However, it is an oversimplification to state that the only difference between an isolated and a fixed-base superstructure is that the isolated superstructure is designed for lower force and displacement demands. There are certain unique aspects of isolated superstructures that the designer would not consider in designing a fixed base structure. Several superstructure design issues raised in a paper by Mokha (1998), as well as other issues, are summarized below.

***Lateral Force Resisting System.*** Because forces in the superstructure are lower in an isolated building than in an equivalent fixed base building, and because most lateral deformations in an isolated building are concentrated at the isolation interface, a wider range of options for lateral force resisting systems may be feasible in an isolated building. However, there is an interplay between the dynamic response of the superstructure and the dynamic response of the isolation system. Therefore, in selecting the lateral force resisting system for an isolated structure the designer must consider explicitly the dynamic response of the isolation system and the superstructure acting together. Fundamentally, for the isolation system to be effective, the natural period of the superstructure (if it were constructed on a fixed base) must be significantly shorter than the natural period of the isolation system supporting the mass of the superstructure. Torsional effects may also be a consideration: if the center of mass and center of stiffness of the superstructure are far apart then unacceptable torsional motions may be introduced that cannot be accommodated by the isolation system.

For simple, symmetric isolation systems and superstructures, it is usually possible to obtain a good approximation of the dynamic structural response using equivalent static linear analysis procedures. However, to more accurately model the dynamic interaction of the superstructure and isolation system, and the nonlinear aspects of the isolation system, a nonlinear dynamic analysis is usually conducted. It is sometimes necessary to try several variations of the superstructure and isolation system before an optimal configuration is determined. Mokha (1998) suggests the following guidelines for modeling the isolation system. The isolation system must be modeled in sufficient details to 1) account for spatial distribution of the isolators; 2) account for bi-directional loading effects in the isolators; 3) account for the effects of rate of loading, when important; 4) account for vertical motions, when important; and

5) assess overturning/uplift effects. For further guidance on analytical modeling, refer to Chapter 3.

**Diaphragm above the Plane of Isolation.** A feature of isolated superstructures that is not considered in fixed base structures is the diaphragm located above the plane of isolation. This diaphragm is necessary to redistribute lateral loads from the superstructure into the isolation system. Because this diaphragm represents additional complexity and cost, it should be considered when comparing fixed base and seismically isolated alternatives.

A related consideration is the placement of the plane of isolation. If the plane of isolation is located below grade, such as in a basement or crawl space, then a seismic isolation gap, or "moat" will need to be provided around the perimeter of the structure. If isolators are located within columns (at the base, midheight or top of basement or first story columns), then the columns will have to be evaluated for their ability to resist the loads imposed by the isolators, including P-Δ moments induced by displacement of the isolators. In some cases, due to restrictions imposed by the geometry or use of a structure, it may be necessary to place some isolators in one plane and the remaining isolators in another parallel plane within the structure.

***Uplift of Isolators.*** Although some isolation systems can accommodate uplift displacements and forces, uplift is generally considered an undesirable action in an isolated structure. To avoid or at least minimize uplift, changes may be required in the superstructure. These may include reconfiguration of mass to position more dead load over certain isolators (especially corner isolators, which are often the most susceptible to uplift); broadening of the base of moment frames, braced frames or shear walls to reduce the tendency for uplift at the end of the frames or walls; increasing bay widths; and reducing the overall height of the superstructure.

All of these actions have significant impacts on the superstructure configuration and architectural layout of the building. However, the tendency for uplift in certain isolators may not be recognized in the early schematic design phase of a project, and may not become apparent until a detailed computer analysis of a proposed structure has been carried out. Therefore, the design team should be cautioned about the possibility of uplift in isolators, and made aware that changes in configuration may be necessary following initial computer modeling analysis of the isolated building.

***Vertical Deformations.*** All seismic isolation systems, with the exception of flat sliding systems, undergo small vertical deformations in combination with lateral deformation of the isolator during an earthquake. While these deformations are generally not a concern in the design of the superstructure, care should be taken to ensure that vertical deformations act uniform and simultaneously over the extent of the isolation plane. Differential vertical deformations would induce undesirable stresses in the superstructure.

***Isolator Longevity.*** Longevity issues that could affect superstructure performance include stiffening or softening of the isolators over time and differential compaction of isolators under sustained gravity loads. Because seismic isolation is a relatively young technology, there is limited data on variations in isolator properties when isolators are subject to sustained gravity loads over long periods of time (in excess of 10 years). The available evidence suggests that well constructed and maintained isolations systems of the types most commonly in use - elastomeric systems, sliding systems - generally exhibit stable long-term performance. Nonetheless, the possibility of variations in isolator properties over time should be considered in design of the superstructure. Estimates of the possible range of property variations should be made, and the effects on the superstructure should be assessed using bracketed analyses.

### *2.2.2 Isolation system design*

Selection and detailing the isolation system is the most important and challenging step in the design of a base isolated structure. It is a complex problem that often involves balancing competing design objectives, and which may have more than one acceptable solution. In one viewpoint (Asher 1998) five key considerations are described, which are briefly summarized below:

***Seismic Performance Goals.*** The seismic performance goals for a seismically isolated building are determined by the owner/user of the building. However, the owner/user is usually does not have the technical background to provide a specification of seismic performance. Hence it is important to educate the owner/user about the possible range of performance that could be expected from an isolated building, and the practical aspects (costs, building configuration, construction schedule, etc.) related to achieving specific levels of performance. Two areas of discussion that may help clarify the performance objectives desired by the owner/user:

1) What is the nature of the business that is conducted in the facility, and what are the factors critical to minimizing potential business disruption caused by a strong ground motion event? For example, if the isolated building is a hospital the owner should identify the key operations that must be preserved to maintain the facility's function as a hospital after an earthquake.

2) What are the specific seismic performance goals associated with the range of strong ground motion events that may occur at the site? That is, what level of performance does the owner expect for earthquakes that vary from the most minor to the most severe at a given site? This topic could be approached by constructing a matrix of expected building performance (fully operational, operational, life-safe, near collapse) vs. earthquake frequency at the site (frequent, occasional, rare, vary rare). This performance-based approach to seismic design is discussed in depth in the Structural Engineers Association of California document "Vision 2000, Performance Based Seismic Engineering of Buildings" (SEAOC 1995).

***Site Specific Ground Motion Characteristics.*** Three factors to consider when assessing the effects of site conditions on design of a seismic isolation system are: site soil conditions, proximity to faults, and site specific spectra and time histories. See section 2.5 for further discussion.

***Building Configuration.*** As with fixed-base structures, there are certain attributes of seismically isolated superstructures that are conducive to good seismic performance. A regular distribution of mass and stiffness reduces torsional motions, and a direct and continuous load path for lateral forces avoids areas of the structure with high localized demands. These attributes in turn reduce displacement and force demands on the isolation system.

There is a practical lower limit on superstructure mass, below which it is difficult to design a base isolation system that will operate effectively. This lower limit on mass depends on the seismic isolation system employed, and must be determined on a project-by-project basis.

There are also limitations on superstructure height-to-width aspect ratio. The taller the superstructure, the higher the potential for uplift at the isolators. The designer must evaluate the extent of uplift that can be accommodated, if any, and whether it is necessary to implement measures to reduce uplift.

***Lateral Force Resisting System.*** Generally speaking, any lateral force resisting system can be used in conjunction with a seismic isolation system. However, care must be taken that the dynamic characteristics of the superstructure are appropriate for isolation. Specifically, the natural period of the superstructure must be sufficiently shorter than the natural period of the isolation system so that the benefits of using seismic isolation are realized. If yielding is permitted in the superstructure, consideration should be given to the yielding dynamic characteristics of the superstructure. Although design codes permit yielding of isolated superstructures, this is not an option that is exercised often.

The configuration of the lateral force resisting system also affects the isolator design. If the lateral force resisting system is configured with multiple load paths to the foundation, the maximum force demand on any single isolator will be reduced. In addition, it is necessary to consider how lateral forces are distributed throughout the diaphragm just above the plane of isolation, as this also affects the force demands on isolators.

***Design Responsibilities.*** Although the division of design responsibilities between the structural engineer and the seismic isolation device supplier might appear to be a logistical issue, rather than a technical one, it does impact the design of both the isolation system and the superstructure. In one scenario, the structural engineer may design the superstructure, and the isolation supplier receives performance specifications from the structural engineer and designs the isolation system separately. In another scenario, the structural engineer designs both the superstructure and the isolation system together. While the second alternative is a global approach to design which ideally will result in better overall performance of both the superstructure and isolation system, the fact is that most structural engineers lack the specialized expertise necessary to design an efficient and economical isolation system. Therefore

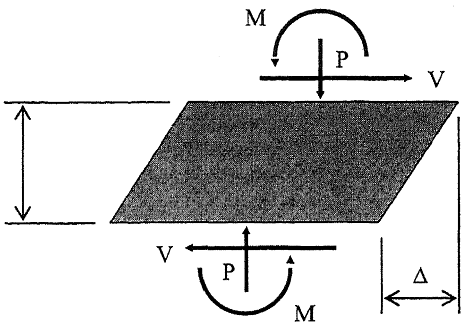


**Figure 2-1.** Forces on elastomeric isolators

the structural engineer and isolation supplier must work together, in an iterative process. The engineer must follow closely the design approach of the isolation supplier, and the supplier must understand the functional requirements imposed by the superstructure design, to insure that the overall isolation design is optimal.

### *2.2.3 Utilities, moats, foundations, etc.*

A unique facet of the design of seismically isolated buildings is the design of the sub-structure. This includes provisions for isolator pedestals and footings that resist not only axial gravity and seismic loads, but also moments induced by lateral displacements of the isolation system. Care should be taken to consider the forces imposed by isolators on columns at the foundation level, including P-Δ moments caused by isolator displacements. Other considerations include the isolation gap (or "moat") around the perimeter of the base; and flexible utility connections that can accommodate large lateral movements during an earthquake. These issues are discussed in a paper by McGruer and Bachman (1998), which is summarized below.

***Isolator Foundations.*** The usual forces on foundations considered in the seismic design of a fixed base structure include gravity forces and vertical and horizontal forces due to lateral earthquake loads. For a seismically isolated structure additional moments caused by lateral displacement of the isolation system must also be considered. These additional moments include a P-Δ component caused by vertical loads acting on the displaced isolator, and moments caused by shear loads acting on the displaced isolator. For elastomeric isolators, these additional top and bottom moments, which by symmetry are assumed to be equal, are illustrated in Figure 2-1 and are given by equation 2-1.

$$M = \frac{P * \Delta}{2} + \frac{V * H}{2} \tag{2.1}$$

It should be noted that for friction pendulum isolation systems (FPS) the P-Δ component of the moment is resisted solely by the superstructure or by the foundation, depending on the orientation of the isolator. This distinction is illustrated

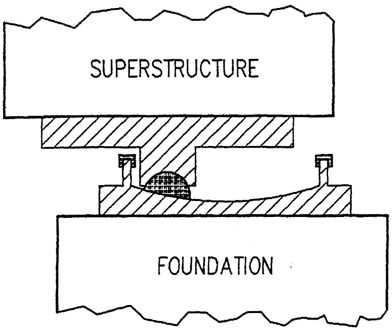


**Figure 2-2.** Two configurations of FPS isolators

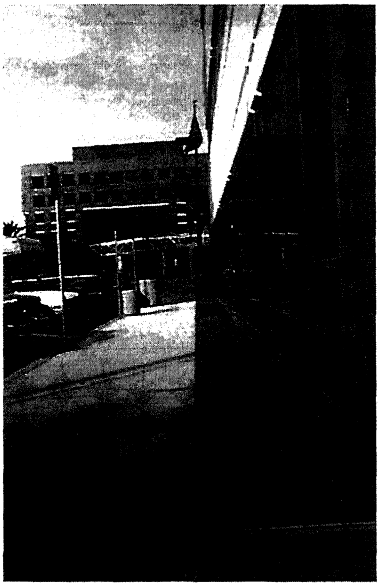

**Figure 2-3.** Typical seismic gap cover, USC Hospital (photo courtesy KPFF Consulting Engineers)

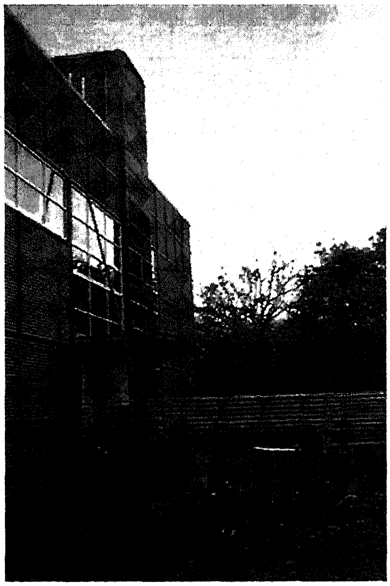

**Figure 2-4.** Access ramp crossing the seismic gap. Emergency Operations Center, Camp Murray, Washington (photo courtesy KPFF Consulting Engineers)

in Figure 2-2. If the spherical sliding surface is mounted in the upright position [Figure 2-2(a)], then the P-Δ moment is resisted by the pedestal below; if the spherical sliding surface is mounted in the inverted position [Figure 2-2(b)] then the P-Δ moment is resisted by the superstructure above.

**Seismic Gap.** To accommodate lateral movements of an isolated building, a clearance space, or "seismic gap" must be provided around the perimeter of the base. Often, where the isolation system is located below grade, the seismic gap takes the form of a moat. The width of the moat corresponds to the ultimate permitted lateral displacement of the isolation system. In some buildings the isolation system is located above the surrounding grade, so a moat is not formed. In those cases other lateral displacement limiting stops must be provided.

Special architectural features are associated with the seismic gap. If a moat is formed around the perimeter of the base a cover is usually provided over the seismic gap. This cover must support gravity loads associated with ingress and egress from the building (pedestrians, freight) but must not restrict lateral movement of the superstructure. The simplest and most common form of seismic gap cover consists of horizontal steel plates attached to the superstructure, as shown in Figure 2-3. The outer free edge of the plates rests on the top of the moat wall, with enough overlap to prevent the plates from falling into the gap under the maximum possible lateral

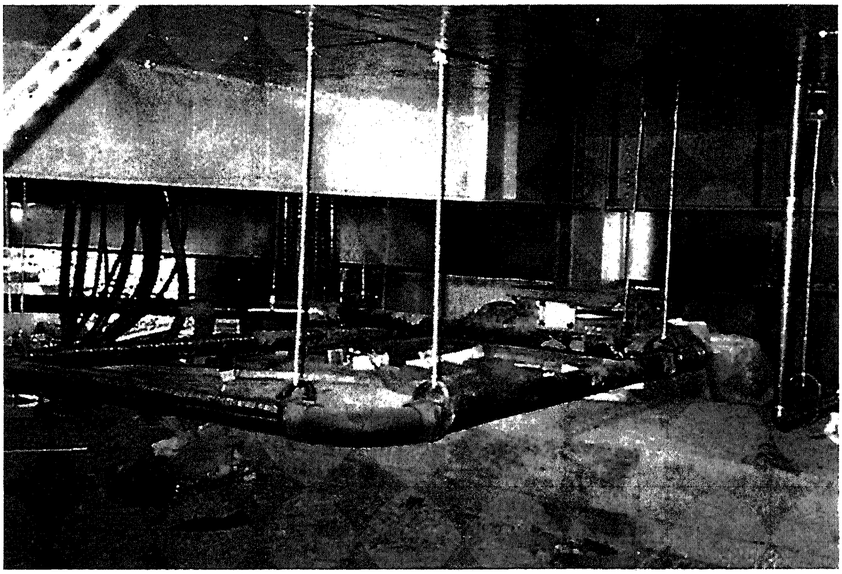

**Figure 2-5.** Flexible electrical conduit connections (left), and multi-jointed sanitary sewer connection (center) at the Washington State Emergency Operations Center (photo courtesy KPFF Consulting Engineers)

displacement of the structure. If a moat cover is not provided, access to the building is sometimes provided by access bridges that are attached to the building at one end and are free to slide on the ground surface at the other end. An example is shown in Figure 2-4. Another architectural consideration is the configuration of elevator pits. Typically in isolated buildings the elevator pits are suspended below the first floor of the structure, within the space provided for the seismic isolators. Sufficient clearance is provided around the elevator pits to avoid interference when the isolation system undergoes the maximum possible lateral displacement. Note that center piston hydraulic elevators are not normally considered for use in isolated structures because the piston would have to cross the plane of isolation.

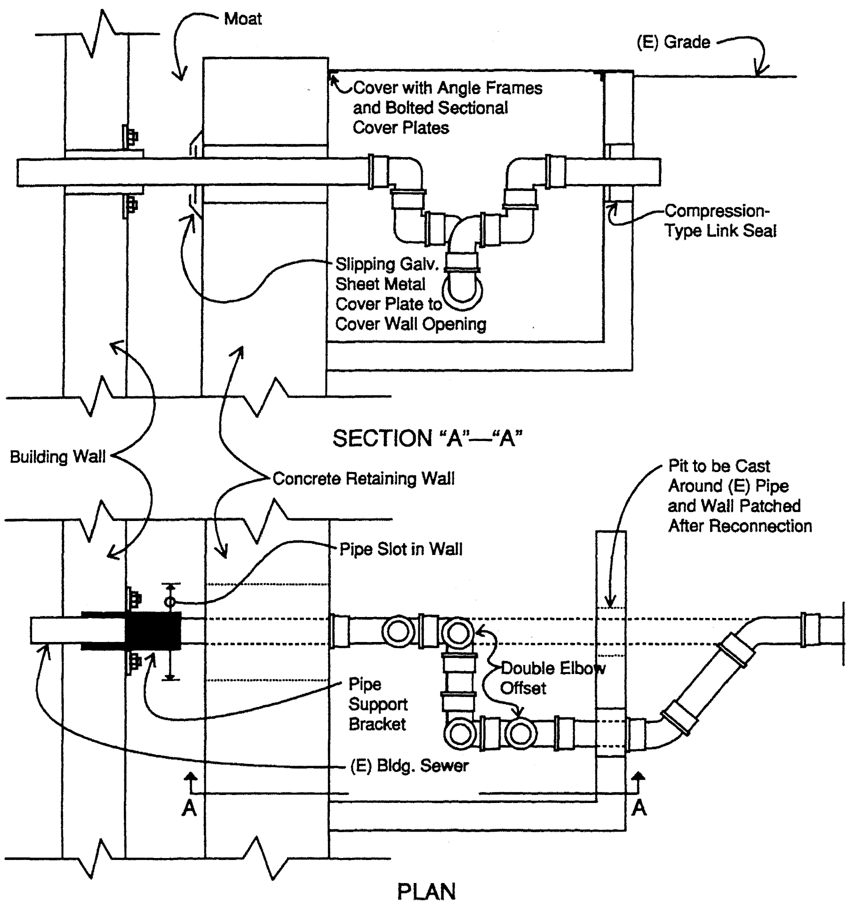


**Figure 2-6.** Example of a multi-jointed rigid pipe utility connection (figure courtesy DIS, Inc.)

Special provisions must be made for all utility lines crossing the seismic gap. Flexible connections must be provided for water supply, wastewater, natural gas, electrical supply, and telephone services. Flexible utility connections must be capable of undergoing displacements in multiple horizontal directions, and must

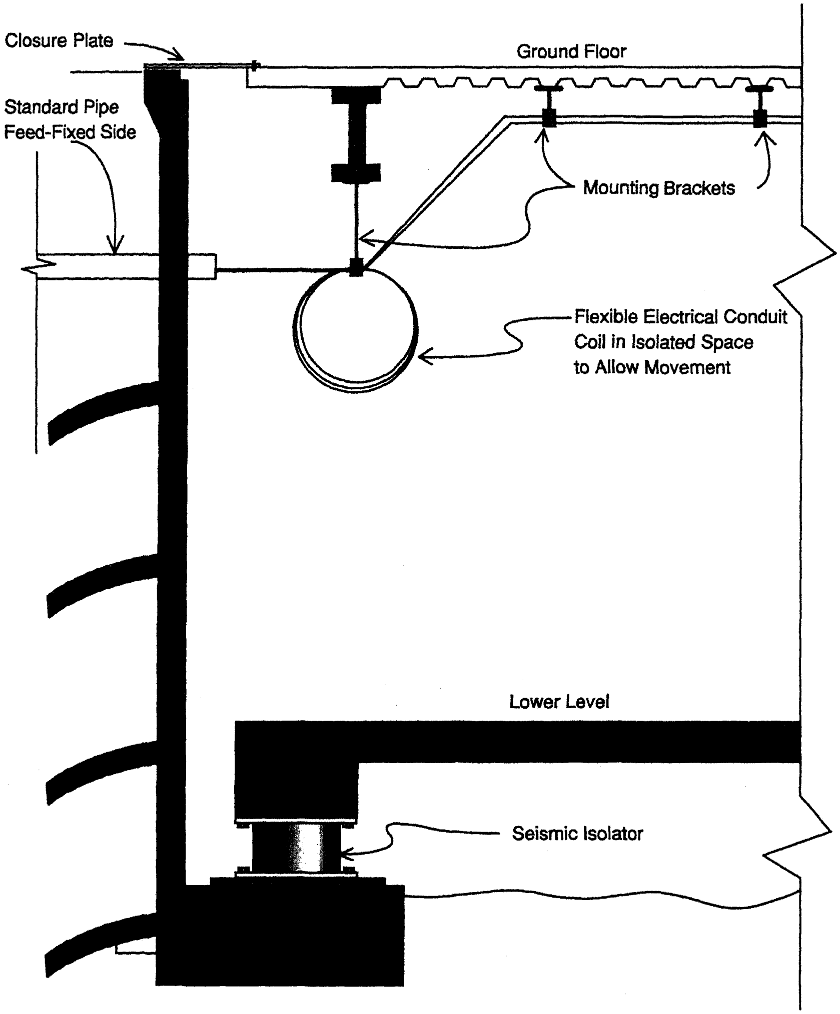


**Figure 2-7.** Example of a flexible electrical conduit connection (figure courtesy DIS, Inc.)

accommodate small vertical movements as well. Various methods for providing these connections have been developed. These include flexible armored hoses and cables, flexible pipes coiled into large loops, and assemblies of rigid pipes with flexible joints. Examples of flexible utility connections are shown in figures 2-5 to 2-8.

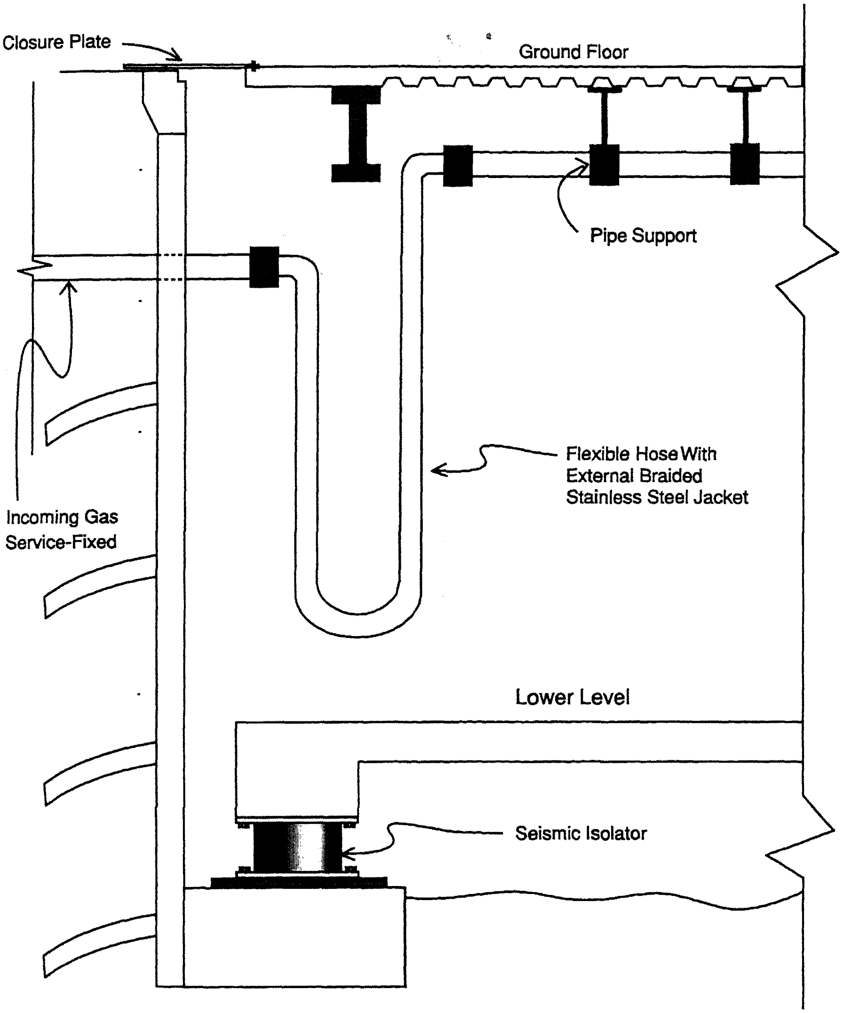


**Figure 2-8.** Example of a flexible natural gas connection (figure courtesy DIS, Inc.)

## 2.3 DESIGN OF ISOLATED BRIDGES AND VIADUCTS

The design of isolated bridges and viaducts differs from the design of isolated buildings in several respects. First, because bridges and viaducts are often long structures, the effects of spatial variance of ground motions should be investigated. For the design of buildings it is commonly assumed that at any given instant all points of the substructure are subjected to the same ground motion. However, for bridges and viaducts, spatial variations in the ground motions may become important. Second, the substructures of bridges and viaducts are commonly more flexible in the horizontal direction than the substructures of buildings. While buildings usually are supported on relatively rigid foundations, bridges and viaducts are often supported on flexible piers. Third, the flexibility of the substructure may vary over the length of a bridge or viaduct. This is because the heights of bridge piers change to accommodate variations in terrain. Fourth, vertical ground motions may become important in the design of isolated bridges. This is because the long spans typical of bridges and viaducts create structures that are flexible in the vertical direction, possibly making them sensitive to vertical ground motions. Finally, the philosophy behind the application of isolation in bridges and viaducts is sometimes different than for buildings. With bridges, the primary goal may not be to limit forces and displacements in the superstructure, but rather to limit forces in the substructure (piers). This is particularly true in retrofit applications, where the existing bridge superstructure may be satisfactory, but the existing substructures have insufficient lateral capacity.

A recent innovation in the design of isolated bridges is the introduction of system property modification factors, also known as "λ-factors", that appear in the AASHTO *Guide Specifications for Seismic Isolation Design* (AASHTO 1999). The development and application of λ-factors are described by Constantinou et al. (1999). The λ-factors are intended to correct for the effects of temperature, aging, velocity, wear, contamination and scragging. The factors are applied to obtain bounds on properties such as yielded stiffness, characteristic strength, effective period, base shear coefficient and displacement.

The upper bound on a design property is obtained by multiplying the nominal value of the property by the maximum value of the composite λ-factor, $\lambda_{max}$. For example, the maximum value of yielded isolator stiffness, $K_d$, would be obtained by

$$K_{d,\,max} = \lambda_{max}\, K_d$$

where

$$\lambda_{max} = (\lambda_t)\,(\lambda_a)\,(\lambda_v)\,(\lambda_{tr})\,(\lambda_c)\,(\lambda_{scrag})$$

and

$\lambda_t$ = factor to account for the effect of temperature

$\lambda_a$ = factor to account for the effect of aging (including corrosion)

$\lambda_v$ = factor to account for the effect of velocity (established by tests at different velocities)

$\lambda_{tr}$ = factor to account for the effect of travel and wear

$\lambda_c$ = factor to account for the effect of contamination (sliding systems)

$\lambda_{scrag}$ = factor to account for the effect of scragging (elastomeric systems)

Additional multipliers are applied to the λ-factors listed above (except $\lambda_v$) to account for the importance of a bridge. These adjustment factors are 1.0 for critical bridges, 0.75 for essential bridges, and 0.67 for all other bridges. The adjustment factors are multiplied only by the portion of the λ-factor that deviates from unity. For example, if $\lambda_a = 1.4$ for an essential bridge, then $\lambda_{a,\ adjusted} = 1 + 0.75 \times 0.4 = 1.3$.

Recommended maximum values for some λ-factors are contained in the AASHTO specifications, while other maximum λ-factors must be obtained through testing. At this time the minimum value of all λ-factors is specified as 1.0.

## 2.4 ISOLATION OF INDUSTRIAL FACILITIES AND EQUIPMENT

Although seismic isolation has most commonly been used to protect buildings and bridges from earthquakes, isolation is increasingly being used to protect other types of structures and equipment, including power plant vessels, computers, sensitive equipment, tanks, and marine facilities. Further information on these types of structures may be found in Tajirian (1998) and Constantinou (1998).

Generally speaking the same principles that apply to isolation of buildings and bridges apply to isolation of equipment and tanks. However, there are a few key differences:

1) Equipment and components usually have much lower mass than buildings and bridges, which makes the implementation of seismic isolation more difficult. The smallest possible number of isolators must be used, which is typically four. Since there is no redundancy of supports if four isolators are used, the reliability of individual isolators becomes particularly important.

2) When an existing piece of equipment or component is retrofitted with seismic isolators, there may be restrictions on the allowable lateral translation of the equipment or component.

3) When isolated equipment is located at the top of a building, isolator displacements may be greater than if the equipment was located on the ground, due to amplification of response in the upper stories of the building.

### *2.4.1 Tanks*

Tanks have not performed well in past earthquakes. Common types of observed damage include "elephants foot buckling", tearing of tank shells at hold down connections, sliding of tanks, damage of tank roofs, and spilling of tank contents due to sloshing. Isolation can reduce the seismic forces induced on tanks, thereby decreasing damage and lessening the danger of release of hazardous contents. However, care should be taken that interaction between the tank contents and the isolation system does not exacerbate sloshing.

Liquid storage tanks are subjected to hydrostatic and hydrodynamic forces that result in the following conditions in the tank:

1) *Shell hoop stresses*: Hoop tensile stresses develop as a result of the hydrostatic pressure and the hydrodynamic pressure due to horizontal and vertical ground motions.

2) *Shell compression:* The impulsive and convective liquid loadings induce overturning moments on the tank that cause vertical compressive stresses to develop in the tank shell. These stresses may cause buckling of the shell. Particularly, the combined vertical compression and hoop tensile stresses may cause elastoplastic buckling (known as "elephant's foot buckling") as the latter stresses approach the yield limit of the metal. This problem is further compounded in unanchored tanks that experience uplift. The problem can lead to requirements for (a) a wider and shallower tank, (b) thicker plates in the shell and/or annular area, (c) holding down bolts or straps.

3) *Sliding*: In unanchored tanks, the horizontal seismic forces may exceed the frictional resistance between the tank and its supporting base

4) *Liquid Sloshing:* Sloshing waves create the requirement for additional shell height to prevent either damage to the roof or spillage of liquid

Liquefied natural gas (LNG) storage tanks have unique design requirements that make the use of seismic isolation in areas of high seismicity an economically viable solution. LNG storage tanks are commonly tested prior to commissioning by filling to full height with water. Since water has a density twice that of LNG, this test requirement results in additional shell thickness and a natural ability of the tank to safely resist moderate seismic forces. However, in areas of high seismicity, the seismic forces dominate the design of the structure. Options available to the designer are as follows:

1) Maintain the selected tank geometry and accommodate the seismic forces by providing thicker tank shell and anchorage against lateral and vertical tank movement. While this is a viable option for water and oil storage tanks, it is often undesirable for tanks storing liquids at cryogenic conditions. The reason is that the anchorage (a) complicates the installation of insulation between the inner and outer tanks, (b) allows for heat leakage, (c) increases the requirements for heating conduits that prevent frost heave in the foundation, (d) anchors have to accommodate thermal movements of the inner tank and penetrate the outer tank bottom (for a steel outer tank) or the steel vapor

barrier (for a concrete outer tank). This makes the detailed design of the anchors difficult, (e) anchors result in stress concentrations and other uncertainties in the already highly stressed tank shell (where hoop forces and vertical bending are maximum). Accordingly, unanchored tanks are viewed as safer tanks.

2) Modify the tank geometry and allow the use of a low aspect ratio. This option is often a costly alternative since it typically results in an uneconomical tank shape and uneconomical use of site area due to tank spacing regulations. Further costs are incurred in the case of in-ground tanks due to the cost of the additional excavation. Moreover, the change of shape increases the problems associated with sloshing.

3) Utilize a seismic isolation system. Such a system may reduce the most impacting horizontal seismic loads and thus allow the use of an economical tank shape without the requirement for vertical and lateral restraints. Additional benefits are realized in the design of the outer tank. Effectively, the isolation system allows the utilization of a design that is appropriate for low and moderate seismic forces in an area of high seismicity. Added cost penalties are incurred due to the cost of the isolators, the cost of a double base slab and costs associated with accommodating large displacements in the attached pipe work.

The consequences of failure of a liquefied natural gas (LNG) storage tank are especially severe. Several LNG have been seismically isolated to protect against earthquake damage and a catastrophic release of contents. Bomhard and Stempniewski (1993) have described the major design issues associated with isolation of LNG tanks. A schematic diagram of a LNG tank is shown in figure 1-6. The tank consists of an inner steel tank that contains the LNG, and an outer reinforced concrete tank that protects the inner tank. The capacity of LNG tanks can be as high as 150,000 $m^3$.

Two recent LNG tank projects have incorporated seismic isolation. The first is located in Inchon, Korea, where three tanks with a capacity of 100,000$m^3$ each are being constructed (Koh 1997). The seismic isolation devices are elastomeric bearings measuring 600 mm in diameter by 228 mm high, with an isolated period of approximately 3 seconds. The second project is located on Revithoussa Island, Greece, and is described in section 1.3. Two tanks have been constructed, each with a capacity of 65,000 $m^3$. The isolation system consists of 212 friction pendulum system (FPS) isolators beneath each tank. The radius of curvature of each isolator is 1880 mm, and the lateral displacement capacity is 300 mm.

### *2.4.2 Nuclear reactor vessels*

Programs in several countries are aimed at developing seismic isolation systems for nuclear reactor vessels (Tajirian, et al. 1990). The most important advantage to using seismic isolation in nuclear power plants is that the overall reliability and safety of reactor vessels can be improved. Another advantage is that the design of the reactor vessel can be standardized: varying seismic conditions can be accommodated by adjustments to the isolation system.

There are currently six seismically isolated Pressurized Water Reactor (PWR) units: four in France and two in South Africa. At the Cruas plant in France, each of the four units has been constructed on 1,800 neoprene pads measuring 500 by 500 by 65 mm. The seismicity at these plants is moderate, with a safe shutdown earthquake (SSE) design acceleration of 0.2g (Postollec 1983). In Koeberg, South Africa two units are isolated on a total of 2000 neoprene pads measuring 700 by 700 by 100 mm. In this case the SSE design acceleration is 0.3g. The pads are outfitted with sliders on the top surface, consisting 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 (Jolivet and Richli 1977).

In the United States the Department of Energy has sponsored a program to develop the Advanced Liquid Metal Reactor (ALMR). In this program seismic isolation has been incorporated in the ALMR design to improve safety and to allow the development of a standard design for varying regions of seismicity (Tajirian and Patel 1993). The prototype ALMR design incorporates 66 high damping rubber bearings. Prototypes of these bearings have been tested extensively (Tajirian, et al. 1990, Clark, et al. 1995). The Department of Energy has also sponsored development of the Sodium Advanced Fast Reactor (SAFR), which incorporates seismic isolation. The prototype design is supported on 100 elastomeric isolators. Reduced scale isolators have been tested to verify their performance (Aiken, et al. 1989)

In Japan design guidelines for seismically isolating Fast Breeder Plants have been developed by the Central Research Institute of Electric Power Industry (CRIEPI). The guidelines have also been modified to apply to Light Water Reactors. Although there are currently no seismically isolated nuclear reactors in Japan, these guidelines make them a possibility. Work has also commenced in Japan on applying seismic isolation to the International Thermonuclear Experimental Reactor (ITER) (Fujita 1997).

### *2.4.3 Components and equipment*

An early application of seismic isolation of components was the isolation of 230kV circuit breakers in Southern California (Kircher 1979). This led to shake table testing of other power plant components and light secondary systems (Kelly 1983). Electric circuit breakers have also been isolated in Japan and Italy (Bonacina, et al. 1994).

An example of a large piece of isolated sensitive scientific equipment is the Mark II Detector at the Stanford Linear Accelerator Center. The mass of the detector is approximately 1360 tonnes (3000 kips) and it measures 7.6m W x 10.5m L x 9.25m H. The isolation system consists of four lead rubber bearings.

At the J. Paul Getty Museum in Malibu, California, fragile art objects have been isolated on devices consisting of steel ball rollers on steel plates, with an additional re-centering device (Yaghoubian 1991).

In Japan raised floors in computer rooms have been outfitted with seismic isolation systems. There are several different systems in use. These incorporate components such as springs, pneumatic isolators, multi-stage rubber bearings, sliders and dampers (Fujita 1991). Although isolated floor systems have not been implemented in the United States, a proprietary system has been developed and subjected to shake table tests. The system consists of elastomeric bearings and PTFE/stainless steel sliders (Tajirian 1990). Another floor isolation system consisting of friction pendulum system (FPS) isolators and dampers has been tested on a shake table by Lambrou and Constantinou (1994).

### *2.4.4 Reduction of foundation loads*

Seismic isolation has been employed to retrofit components and equipment with the specific aim of reducing foundation loads. This technique is useful either to reduce the seismic foundation loads imposed by existing equipment, or to reduce the foundation seismic loads imposed by new equipment. Tajirian (1998) lists three examples.

The first example concerns components of Titan IV launch vehicles that are stored in a building at Vandenburg Air force Base in California. These components weigh from 67 to 167 tonnes (148 to 368 kips) each. It was determined that a seismic upgrade of the storage facility was required to prevent injury to personnel or damage to the components in an earthquake. It was determined that if the components were rigidly attached to the building foundation, upgrading of the foundations would be required. Seismic isolation of the heaviest components was adopted as an alternative. Each component was placed on a steel frame that was supported on four high damping rubber bearings.

In the second example mobile exciter equipment, necessary in an emergency for restoring power generating capacity at the Diablo Canyon Power Plant in California, was seismically isolated. The mobile exciter units consist of transformers and switchgear mounted on a truck trailer. The weight of each exciter unit is approximately 29 tonnes (64 kips). It was determined that to resist seismic loads the truck trailers and equipment anchors would need to be strengthened, a large mounting skid would be required, and the number of foundation mounting bolts would be excessive. To avoid these measures, and to decrease the time required to put the mobile exciter units into service after an earthquake, it was decided to seismically isolate the truck trailers by mounting each trailer on four high damping rubber bearings.

The third example is an application of seismic isolation in the offshore oil drilling industry (Medeot and Infanti 1997). An offshore drilling platform, located in the Caspian Sea, was partially completed when it was determined that the foundation piles would be subject to uplift due to seismic overturning forces. The first solution considered was to install additional piles on the sea floor to provide additional uplift resistance. This was determined to be too costly and not possible within the available schedule. The solution that was adopted was to place the upper deck structure on seismic isolators, thereby reducing the seismic demand on the lower tower, or jacket, of the platform. Spherical PTFE sliding bearings were employed, in parallel with steel hysteretic dampers. A sacrificial restrainer system was provided that was capable of resisting wind and wave loads, but that would break under earthquake loads, allowing activation of the isolation system.

## 2.5 ISSUES RELATED TO SELECTION OF DESIGN GROUND MOTIONS

If a seismically isolated building lies near an active fault it is required that "near fault effects" be reflected in the ground motion at the site. These near fault ground motions may contain large velocity pulses, which have the potential to cause large displacements in the isolation system. These displacements can usually be accommodated, provided they are considered early in the design process.

Site specific response spectra and site specific acceleration time histories, which are derived from a probabilistic seismic hazard analysis of the site, and include the effects of fault proximity and site soil conditions, are developed for the range of strong motion events expected at the site. Normally two levels of ground shaking are considered: a "design basis earthquake", usually corresponding to 10% probability of exceedance in 50 years; and a "maximum considered earthquake", usually corresponding to a 2% or 5% probability of exceedance in 50 years. The seismic isolation system must be designed to meet pre-determined performance criteria (see Seismic Performance Goals, Section 2.2.2) under both levels of ground shaking.

Experience has shown that delays often result from misunderstandings about the design ground motions. The engineer, peer reviewer, architect, owner and regulatory agencies should all come to a clear agreement on the design ground motions early in the design process.

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# 3. Analysis

## 3.1 INTRODUCTION

This chapter presents a brief overview of the current analytical modeling techniques for isolated structures. Perhaps the biggest challenge in the design of an isolated structure is incorporating the effects of nonlinear isolator behavior in the analysis. Furthermore, seismic response of an isolated structure can be sensitive to variations in the properties of the isolators. Therefore bounding analyses are required to assess the expected range of performance of the structure. Finally, while design codes usually contain provisions allowing equivalent static or response spectrum methods of analysis under limited conditions, for most practical structures, time-history analysis is required.

## 3.2 GENERAL APPROACH

The analysis of an isolated structure - whether a new structure or a retrofitted one - is an iterative process. As a first step, preliminary characteristics of the isolation system and superstructure are estimated. Using these properties, the global response of the isolated structure is computed and expressed in terms of maximum displacements and story shears (at the base and within the superstructure) in the design level earthquake and under the maximum considered earthquake. This first analysis step is often accomplished using equivalent static analysis procedures described in the governing building code, and incorporating preliminary isolator properties supplied by isolator manufacturers (often these preliminary properties are based on test data from previous projects). Second, using the approximate displacements and story shears obtained from the equivalent static analysis, the performance of the isolation system and superstructure are evaluated with respect to pre-established performance criteria (see section 2.2.2 for a discussion of performance criteria). The characteristics of the isolation system and/or superstructure are then modified to improve performance, and the approximate analysis of the structure is repeated. Once satisfactory performance is indicated, a refined analysis of the structure is conducted to verify the performance. In most cases this refined analysis involves time-history analysis, incorporating nonlinear characteristics of the isolation system.

Non-linear time history analysis could be performed using non-linear programs such as SAP2000 (CSI 2000), ETABS (Wilson, et al. 1975, CSI 1996), DRAIN-2DX (Prakash, et al. 1993), ANSR-I (Mondkar and Powell 1975), 3D-BASIS (Tsopelas, et al. 1991, Nagarajaiah, et al. 1991, Nagarajaiah, et al. 1993, Tsopelas, et al. 1994, Reinhorn, et al. 1994), and RUAUMOKO (Carr 1998). Mechanical properties of the elastomeric isolators and sliding isolators are described in section 3.5.

Generally speaking, the analysis of an isolated structure should consider the following factors.

1) The distribution of superstructure stiffness and mass
2) The influence of torsional movements of the isolator/superstructure system
3) The spatial distribution of isolators
4) The effects of bi-directional loading on the performance of isolators
5) The influence of the rate of loading on the performance of the isolators
6) The influence of varying vertical loads on the performance of the isolators
7) The potential for overturning of the superstructure and consequent uplift of isolators

## 3.3 VARIATIONS IN ISOLATOR PROPERTIES

Because seismic response can be sensitive to variations in properties of the isolation system, it is important that the analysis reflect as accurately as possible the actual properties of the isolators installed in the structure. The isolation system must be modeled using parameters verified by representative tests on prototype isolators (see Chapter 4 for a discussion of isolator testing). Furthermore, bounding analyses must be performed to assess the influence of expected variations in isolator properties. These variations can result from the manufacturing process, or from factors such as aging, temperature, contamination, etc. The 1999 Guidelines for Seismic Isolation Design published by American Association of State Highway and Transportation Officials (AASHTO 1999) describes a procedure to account for aging and environmental effects.

## 3.4 GROUND MOTIONS

The 1997 Uniform Building Code (ICBO 1997) describes the requirements for ground motion time-histories used in analysis. First, the ground motions must be compatible with design response spectra for the "design basis earthquake" (DBE) and "maximum considered earthquake" (MCE). Criteria for determining if ground motions are compatible with the design spectra are presented. The UBC requires the use of a minimum of three pairs of orthogonal time-histories. When three pairs of time-histories are used, the superstructure and the isolation system are designed for the pair of time-histories that provides the maximum response. If seven pairs of time histories are used, the superstructure and isolation system are designed for an average of the seven responses. For an isolated structure located near an active fault, the ground motion time-histories must incorporate near-field effects.

## 3.5 MODELING COMMON ISOLATION SYSTEMS

An understanding of the nonlinear behavior of the isolation system is important to proper analytical modeling of the system. In this section the force-deformation characteristics of common isolation systems are described.

### *3.5.1 Elastomeric Isolators*

***Lead-rubber Isolators.*** Lead-rubber isolators are made up of low damping natural rubber with a lead core. The lead core is provided to increase the energy dissipation capacity from the range of 2 to 5% of critical to about the range of 20 to 30% of critical (at the design displacement). The idealized force displacement behavior of a lead-rubber isolator can be characterized as bilinear hysteretic as shown in Fig. 3-1. The high initial stiffness, $K_e$, offers rigidity under wind load and low-level seismic load. It typically ranges between 6.5 to 10 times the post-yield stiffness, $K_p$. The yield displacement $D_y$ is typically in the range of 6 to 25 mm (0.25 to 1 inch). The characteristic strength $Q$ is defined to be the yield force of the lead core; it is given by $Q = A_p\sigma_{YL}$, where $A_p$ is the lead plug area and $\sigma_{YL}$ is the effective shear yield stress of lead. The post-yield stiffness of the rubber portion of the isolator, computed without including the effects of the lead core, would be given by the product of the bonded rubber area, $A_r$, and the rubber shear modulus, $G$, divided by the total rubber thickness, $\sum t$. The post-yield stiffness of the isolator including the rubber core, $K_p$, is slightly higher and is expressed by

$$K_p = \frac{A_r G}{\sum t} f \tag{3.1}$$

where $f$ is a factor larger than unity. Under proper conditions, $f$ may be equal to or less than 1.15.

***High Damping Rubber Isolators.*** The stiffness and energy dissipation characteristics of high damping isolators are highly nonlinear and dependent on shear strain as

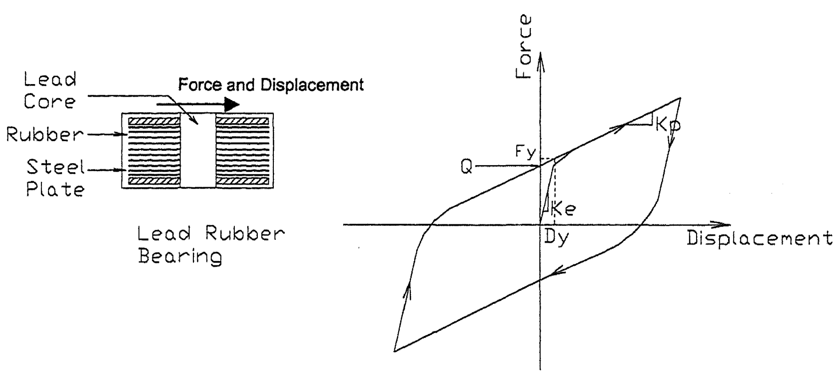


Figure 3-1. Typical force-displacement loop of a lead-rubber isolator

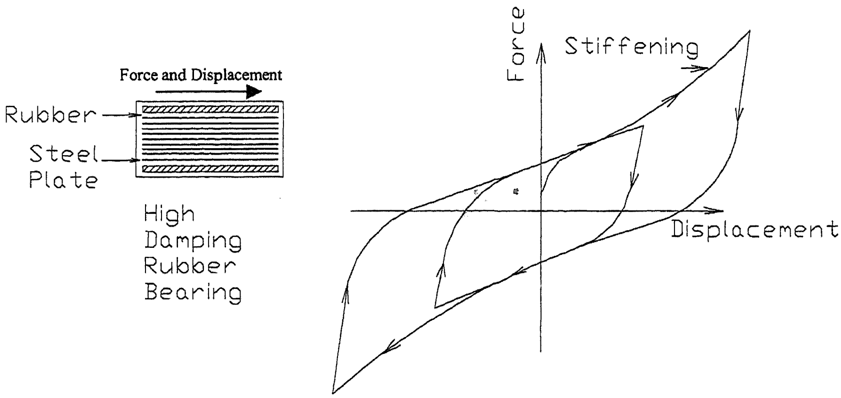


Figure 3-2. Typical force-displacement loop of a high-damping rubber isolator

shown in Fig. 3-2. The high damping isolators are made up of specially compounded rubber, which provides effective damping of 8 to 16 % of critical (at the design displacement). The high damping isolators have high shear stiffness at low shear strains (< 20%) for rigidity under wind load and low level seismic load. The shear stiffness is typically lower in the range of 20 to 120 % shear strains. At large shear strains, the shear stiffness increases due to strain crystallization process in the rubber. The damping in high damping isolators is best characterized by a combination of hysteretic and viscous behavior.

In the virgin stage and during the first cycle of movement, the isolators exhibit higher stiffness and damping than in the following cycles. The stiffness stabilizes by the third cycle, resulting in stable properties termed as "scragged" properties. "Scragging" of the isolators is the result of internal changes in the rubber. Recovery to the unscragged (virgin) properties occurs following sufficient time. The scragged state of the isolators can be modeled by a bilinear hysteretic model for shear strains of up to 200%. The stiffening behavior (see Fig. 3-2) beyond this strain can also be modeled using more complex models (Tsopelas, et al. 1994). The current technique used to model high damping isolators is to perform multiple analyses with bilinear hysteretic models; the parameters of the bilinear hysteretic models are determined at specific shear strain amplitudes. The bilinear model parameters can be established from test data of prototype bearings. These properties are the shear modulus, $G$, and the equivalent damping ratio, $\xi$ (defined as the energy dissipated in a cycle of motion divided by $4\pi$ and by the maximum kinetic energy) under scragged conditions. The shear modulus, $G$, is related to the post yielding stiffness $K'_p$

$$K'_p = \frac{GA_r}{\Sigma t} \tag{3.2}$$

The parameters of the model may be determined by use of the mechanical properties of $G$ and $\xi$ at a specific strain—for example parameters corresponding to the design displacement. The post yielding stiffness, $K'_p$, is determined from (3.2), whereas the characteristic strength, $Q$, may be related to the mechanical properties by assuming bilinear hysteretic behavior expressed as

$$Q = \frac{\pi\, \xi\, K'_p D^2}{(2 - \pi\, \xi) D - 2D_y} \tag{3.3}$$

where the yield displacement, $D_y$, is between 0.05 and 0.1 times the total rubber thickness and $D$ is the design displacement. The yield force, $F_y$, is given by

$$F_y = Q + K'_p D_y \tag{3.4}$$

and the post to pre-yielding stiffness ratio is given by

$$\alpha = \frac{K'_p D_y}{F_y} \tag{3.5}$$

### *3.5.2 Sliding Isolators*

The two types of sliding isolators are flat sliding isolators and friction pendulum system (FPS) isolators. Flat sliding isolators are made up of PTFE sliding surface bearing on a flat stainless steel surface. The recentering capability is provided by

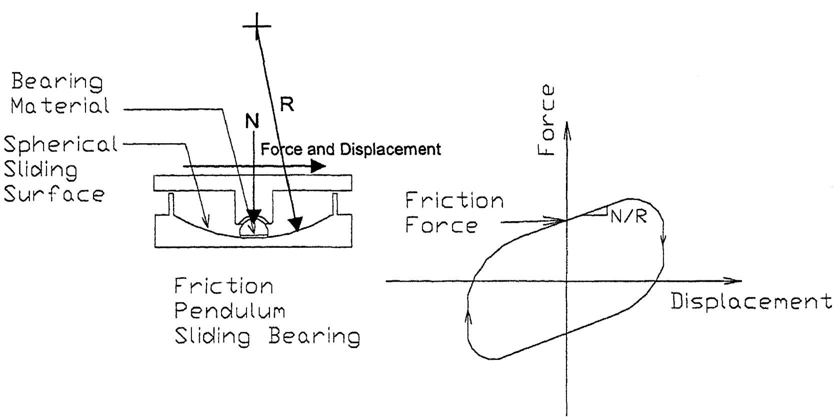


Figure 3-3 Typical force-displacement loop for a FPS isolator

additional elastic springs. The FPS isolator, shown in Fig. 3-3, is made up of a composite material sliding on a spherical surface with radius of curvature R, which provides the recentering force. The behavior of FPS isolators can be represented by (Zayas, et al. 1991)

$$F = \frac{N}{R}U + \mu_s N \operatorname{sgn}(\dot{U}) \tag{3.6}$$

where $F$ is the force in the isolator, $U$ and $\dot{U}$ are the displacement and velocity, respectively, $\mu_s$ is the coefficient of sliding friction and $N$ is the normal load on the isolator. It should be noted that for flat sliding isolators $R$ is infinite, and equation 3.6 reduces to

$$F = \mu_s N \operatorname{sgn}(\dot{U}) \tag{3.7}$$

The coefficient of friction of sliding bearings depends on a number of parameters of which the composition of the sliding interface, bearing pressure and velocity of sliding are the most important. For interfaces consisting of polished stainless steel in contact with Teflon or composites the coefficient of friction may be described by (Constantinou, et al. 1990)

$$\mu_s = f_{max} - (f_{max} - f_{min}) \exp\left(-a\left|\dot{U}\right|\right) \tag{3.8}$$

Here, $f_{min}$ and $f_{max}$ are the coefficients of friction at zero and large sliding velocities under constant normal forces. Parameters $f_{min}$, $f_{max}$ and $a$ depend on the bearing pressure, although only the dependency of $f_{max}$ on pressure is of practical significance.

The coefficient of friction at maximum velocity usually ranges from 0.02 to 0.15, resulting in maximum effective viscous damping of about 25% to 30% of critical at design displacement. The yield displacement is usually in the range of about 0.1 inches to 0.2 inches.

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# 4. Testing

## 4.1 PROCEDURES FOR TESTING ISOLATORS

A program of testing isolators is normally a part of every seismic isolation project. The nature and extent of testing varies, depending on the isolation system and the project. For established isolation systems, testing usually includes preliminary tests of prototype isolators, to verify the assumed design properties, and quality control tests of materials and completed isolators prior to installation in the structure. If an isolation system is new and unproved, then more extensive testing is necessary to determine the basic properties of the system such as dependence on rate of loading, dependence on temperature, and so forth.

### *4.1.1 Testing Requirements*

The Uniform Building Code (ICBO 1997), International Building Code (ICBO 2003), the National Earthquake Hazards Reduction Program *Recommended Provisions for Seismic Regulations for New Buildings* (NEHRP 1995), and the American Association of State Highway and Transportation Officials *Guide Specifications for Seismic Isolation Design*, (AASHTO 1999) all require testing of seismic isolation systems.

In 1992 researchers at the National Institute of Standards and Technology (NIST) began a program of developing recommended procedures for testing seismic isolation systems. The results of this study were reported in "Guidelines for Pre-Qualification, Prototype and Quality Control Testing of Seismic Isolation Systems" (Shenton 1996). In 1994, HITEC (Highway Innovative Technology Evaluation Center), Caltrans and the Federal Highway Administration began a program for testing and evaluating seismic isolation and energy dissipation systems specifically for highway bridges. Below is presented a discussion of testing requirements for isolation systems, as well as brief descriptions of the NIST testing guidelines and the HITEC technical evaluation plan.

### *4.1.2 Test Configurations*

The three most common test configurations for seismic isolation devices are illustrated in Fig. 4-1 (the shaded boxes in Fig. 4-1 represent generic seismic isolation devices). The "single" test configuration, Fig. 4-1(a), most closely represents the conditions imposed on an isolator in the field. However, the boundary conditions at the top of the isolator - free lateral translation and fixity against rotation - are, practically speaking, difficult to achieve. A rolling or sliding interface must be provided at the top of the isolator, which requires a specialized testing apparatus. The "dual" test configuration, Fig. 4-1(b), is easier to implement than the "single" configuration because a rolling or sliding interface is not required. However, a reaction system is still required to apply the lateral force. Furthermore, since data

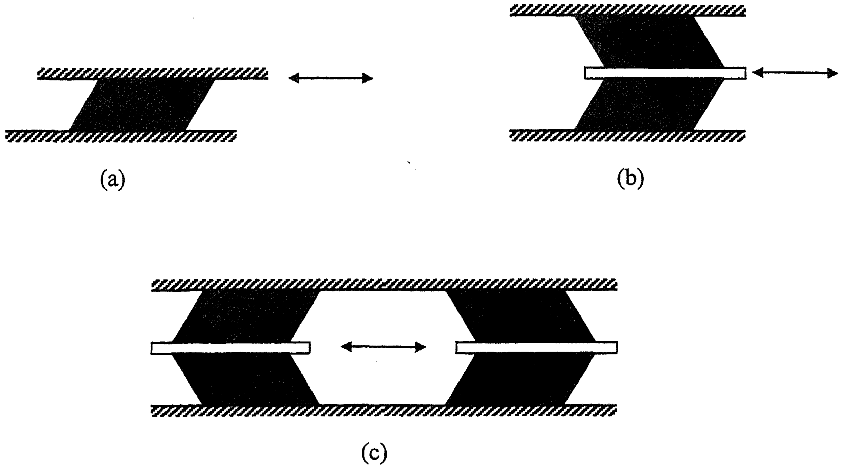

Figure 4-1. Common testing configurations

obtained from the "dual" configuration reflects the average performance of two isolators, the properties of a single isolator cannot be determined. The "quadruple" test configuration, Fig. 4-1(c), requires neither a sliding interface nor an external lateral reaction system, since lateral forces in the test frame are self-equilibrating. With this configuration the test data obtained represents the average behavior of four isolators.

Vertical loading is normally maintained at a constant value during application of cyclic lateral loads, to simulate the self-weight of the structure. (If the performance of an isolation system is dependent on variations in vertical load, tests are repeated over a range of vertical loads). With some isolation systems, the vertical load platen may displace slightly during application of cyclic lateral loads, and this movement must be accommodated by the test apparatus. For example, the friction pendulum system (Zayas, et al. 1990) consists of a slider riding in a spherical dish, which imposes small vertical deformations in conjunction with the lateral deformations. Another common isolation system, laminated elastomeric bearings, shorten slightly as shear deformations are applied, so the point of application of the vertical load must fall and rise slightly during each cycle of shear deformation.

Regarding lateral loading, to realistically simulate earthquake loading conditions lateral loads would have to be applied in a "force controlled" mode. That is, lateral forces would be applied, and the test control system would provide the displacement necessary to develop that force. However, this mode of testing is seldom employed because it is dangerous: large, sudden lateral deformations could occur if the isolation system failed or became unstable. Consequently, most tests are performed in a "displacement controlled" mode, in which a specific lateral displacement is applied, and the test control system provides the force required to develop that displacement.

### *4.1.3 NIST Testing Guidelines*

The NIST testing guidelines (Shenton 1996) contain five categories of tests (I, II, III, IV and V), as shown in Table 4-1. Whether or not a specific category of test is performed depends on the purpose of the testing program. Three types of test programs are described in the guidelines: Pre-Qualification (Basic Property) tests; Prototype tests; and Quality Control Tests. (The Basic Property test series was originally referred to in the NIST guidelines as "Pre-Qualification" testing. However, the term "Pre-Qualification" may be confusing, since most designers consider Pre-Qualification to be the process of testing various proprietary isolation systems to qualify the systems for use in a specific project. Therefore, the term Basic Property Tests is preferred, and is used here.) These categories are outlined below.

The NIST tests are intended to be independent of the type of isolation system and the type of isolated structure. That is, with the exception of the system specific tests in category V, the recommended test procedures apply equally to elastomeric, sliding or hybrid isolation systems, in applications involving bridges, buildings, tanks, or other civil structures. The guidelines include general requirements of the test facility, instrumentation, calibration, data acquisition, data analysis, and reporting of results.

***Basic Property tests (Category I and II tests).*** Basic Property tests are performed only once for a new isolation system, or for a substantially revised version of an existing system. The purpose of the tests is to establish the fundamental characteristics of the system, such as the effects of virgin loading (scragged vs. un-scragged behavior); effects of frequency of load; effects of vertical load; effects of load direction; etc. To develop a complete picture of the characteristics of a new or substantially revised isolation system, the Basic Property test requirements include a complete series of Prototype tests (Categories III and IV) and Quality Control tests (Category V) on a typical isolator.

***Prototype Tests (Category III and IV tests).*** Prototype tests are project specific, and are conducted to verify the design properties of the isolation system prior to manufacture of all the isolators for a project. Tests are performed on two specimens of each type and size isolator to be used in the project. Prototype tests determine isolator behavior under seismic loads (Category III) and non-seismic loads (Category IV). The Prototype test series also includes a set of Quality Control tests (Category V) to develop an understanding of the range of Quality Control test results to be expected during production of the isolators for the project.

***Quality Control Tests (Category V tests).*** Quality Control tests are project specific and are conducted to verify the quality of manufacture and as-built properties of the isolation system prior to installation in the structure. Separate Quality Control tests are specified for elastomeric and for sliding isolation systems. This is because the mechanics of the systems, and the materials used in the systems, differ. Thus, for elastomeric systems tests of rubber components are required, while for sliding

**Table 4-1.** Schedule of tests in the NIST guidelines

| Category | Designation | Test |
|---|---|---|
| I | I.1 | Establish effect of virgin loading |
| | I.2 | Establish effect of frequency of load |
| | I.3 | Establish effect of vertical load |
| | I.4 | Establish effect of load direction |
| | I.5 | Establish effect of load plane rotation |
| | I.6 | Establish effect of bilateral load |
| | I.7 | Establish effect of temperature |
| | I.8 | Establish effect of creep |
| | I.9 | Establish effect of aging |
| | I.10 | Establish effect of load cycling |
| | I.11 | Establish effect of load cycle history |
| | I.12 | Establish re-centering capability |
| II | II.1 | Ultimate compression under zero lateral load |
| | II.2 | Compression in displaced position |
| | II.3 | Ultimate tension under zero lateral load |
| | II.4 | Tension in displaced position |
| | II.5 | Lateral load and displacement capacity under design |
| III | III.1 | Effective stiffness and energy dissipation |
| | III.2 | Stability against degradation |
| | III.3 | Stability at maximum lateral displacement |
| IV | IV.1 | Wind load |
| | IV.2 | Thermal displacement |
| | IV.3 | Stability with thermal cycling |
| | IV.4 | Braking/Centrifugal force |
| V | V.1 | Sustained compression |
| | V.2 | Effective stiffness and energy dissipation |
| | V.3 | Compression stiffness |

systems tests of the sliding interface materials are specified. Compression stiffness tests are required for elastomeric systems, while compression stiffness tests are not required for sliding systems, because sliding systems normally are extremely stiff in the vertical direction.

### *4.1.4 HITEC Technical Evaluation Plan*

The HITEC testing evaluation plan has three major objectives: implement a program of testing seismic isolation and energy dissipating devices for highway bridges, provide guidance for selecting and designing these devices depending on the level of performance, and provide information sufficient to guide the development of guide specifications for the use of the devices in new construction and retrofit applications for highway bridges. The evaluation program included the following eight tests, several of which specifically address issues related to highway bridges.

1) *Performance Benchmark:* Verification of initial stiffness and damping/fraction of the device, with a record of the number of loading cycles needed to stabilize response.

2) *Load-Deflection Characterization:* Determination of the load-deflection performance behavior.

3) *Frequency Dependent Characteristics:* Determination of the static and dynamic performance behavior and verification of the constitutive laws.

4) *Fatigue and Wear:* Determination of durability under repetitive, small-displacement displacement cycles, that simulate temperature and non-seismic live load fluctuations, with attention to debonding and other signs of fatigue and wear.

5) *Environmental Aging:* Determination of durability under a salt spray environment as encountered under open bridge deck joints on bridges subject to salting.

6) *Dynamic Performance Characteristics at Temperature Extremes:* Determination of the effects of temperature on device performance.

7) *Durability Test:* Determination of durability under a moderate number of strong motion cycles to assess performance where a large number of high-amplitude aftershocks are anticipated.

8) *Displacement Stability Test to Failure:* Determination of failure loads and associated margins of safety.

### *4.1.5 ASCE Standards Development*

The NIST testing guidelines are a set of recommended procedures, rather than a formal testing standard. Following completion of the NIST guidelines, the American

Society of Civil Engineers (ASCE) formed a standards development committee, which was charged with writing a national consensus standard for testing of seismic isolation systems. The standards committee adopted the NIST guidelines as a pre-standard document, to serve as the basis for development of a revised, formal standard. The committee is now developing the standard.

### *4.1.6 Need for Further Development of Tests*

There is a need to develop tests for the long-term performance of all types of isolators, as discussed in Section 4.4 below. While accelerated aging tests provide limited information about long-term performance of isolators, these tests only approximate the aging process. Presently, the only accepted method for evaluating the long-term performance of isolators is to remove aged isolators from service and re-test them.

There is a need to develop better-standardized procedures for testing some characteristics of sliding systems. This includes trueness of surface; sliding interface material properties, friction properties and wear properties; attachment of the bearing pad to the backing plate; and attachment of the sliding material to the backing plate.

## 4.2 PRACTICAL LIMITATIONS ON TESTING

An important, and often overlooked, consideration in specifying tests for seismic isolation systems is the physical limitations of test facilities. It is possible to conceive of tests that cannot be performed at any test facility available in the world today. Disputes sometimes arise between isolator manufacturers and design engineers over the types and extent of testing to be performed on isolation systems. This is because test procedures are sometimes specified that require load capacities or load rates which are available at only a few facilities in the world, or are not available everywhere. Often, similar but less demanding tests can provide nearly the same information, but at a lower cost.

For example, cyclic lateral load tests of full-scale isolators at rates simulating actual earthquake response (on the order of 0.5 to 2.5 Hz) are difficult or impossible to perform with most test facilities available today. This is simply because the hydraulic power requirements for testing at such rates greatly exceed the capacities of hydraulic pumps and servo valves available at most test facilities. Therefore, cyclic lateral load tests of full-scale isolators are normally carried out at slow rates, if it has been determined that the isolator response is essentially rate-independent. Alternatively, if isolator response is rate-dependent, it may be necessary to test reduced-scale specimens at more realistic rates.

The Seismic Response Modification Device (SRMD) Testing Facility, located on the campus of the University of California, San Diego and owned by Caltrans, was developed specifically to address some of the testing needs for seismic isolators. It is capable of imposing real-time, six-degrees-of-freedom seismic ground motion input to full-scale base isolation bearings used to retrofit major long-span bridges in California.

## 4.3 SHAKE TABLE TESTING

The most conclusive way to evaluate the performance of an isolation system is to observe the behavior of full-scale isolated structures subject to actual earthquakes. However, at present such data is limited, and the next best alternative is to study isolated structures tested on laboratory shaking tables (earthquake simulators).

Shake table testing is usually conducted with scale model isolators and a scale model structure, since few shake tables in the world are capable of testing full-scale structures. In evaluating the results of tests on scale model structures and isolation systems, scaling effects must be carefully evaluated.

While shake table testing is useful to demonstrate the performance of a new isolation system, shake table testing is not practical on a project-by-project basis. Since shake table testing is expensive and time consuming, project-specific shake table tests are prohibitive. Since the basic response characteristics of the isolation system are known from previous shake table tests, and project-specific isolator properties, such as stiffness and damping, can be determined from low-rate tests of individual isolators, these properties can be incorporated in computer models of the isolator/superstructure system. These models can be used to study the anticipated structure response to a range of earthquake ground motions, and if necessary the structure and isolators can be altered to achieve optimal response.

While a short-term shake table-testing program can be used to evaluate the short-term performance of an isolation system, it does not reveal information on the effects of aging of the isolation system. Long-term performance of isolation systems is discussed below.

## 4.4 LONG TERM PERFORMANCE

Presently there are no widely accepted tests for evaluating the long-term performance of isolators, other than removing aged isolators from service and re-testing them. Accelerated aging tests have been proposed for both elastomeric and sliding systems, but these only approximate the chemical and mechanical changes that might occur in isolators over time. As more experience with aged isolators is accumulated and more tests are performed, the long-term performance of isolators will be better understood, reducing the need for accelerated aging tests.

Clark, et al. (1997) tested high damping rubber isolators that had been in service for 12 years at the Foothill Communities Law and Justice Center (FCLJC) in San Bernardino, California. The behavior of these isolators was compared with the data from production tests performed 12 years earlier. The test data indicated that, for the particular rubber compound used in the FCLJC isolators, the shear stiffness increased by as much as 20 percent over 12 years. However, there is some uncertainty associated with the shear stiffness measurements. This is because the pairs of isolators tested at the time the FCLJC was constructed, and the pairs of isolators tested after 12 years of aging, were not the same isolators. That is, direct comparison of shear stiffness data from original and aged isolators was not possible.

Another aging effect observed with elastomeric isolators is known as "scragging and recovery." The shear stiffness of an elastomeric bearing that is loaded in shear for the first time, known as the "unscragged stiffness," is always higher in the first cycle than the shear stiffness in the second and subsequent load cycles, known as the "scragged stiffness". After a period of time that is highly dependent on the elastomer compound, the isolator often recovers some or all of its unscragged stiffness. This process is referred to as "recovery." The effects and importance of scragging and recovery should be evaluated on a project-specific basis.

Presently, long-term performance issues are often addressed in design using the λ-factor procedures described in the AASHTO (1999) *Guide Specifications for Seismic Isolation Design.*

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