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Review

A Review of Seismic Isolation for Buildings: Historical Development and Research Needs

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Abstract: Seismic isolation is a technique that has been used around the world to protect building structures, nonstructural components and content from the damaging effects of earthquake ground shaking. This paper summarizes current practices, describes widely used seismic isolation hardware, chronicles the history and development of modern seismic isolation through shake table testing of isolated buildings, and reviews past efforts to achieve three-dimensional seismic isolation. The review of current practices and past research are synthesized with recent developments from full-scale shake table testing to highlight areas where research is needed to achieve full seismic damage protection of buildings. The emphasis of this paper is on the application of passive seismic isolation for buildings primarily as practiced in the United States, though systems used in other countries will be discussed.

Keywords: isolation; elastomeric; sliding; seismic protection

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1. Introduction

Seismic isolation is a technique to shift the fundamental natural period of a structure to the long period range, e.g., two to four seconds, by placing horizontally flexible isolation devices at the base of the structure to physically decouple it from the ground. For earthquake excitation this period shift translates into reduced floor acceleration and inter-story drift demands on the superstructure (structure above the isolation system) by comparison to the equivalent non-isolated structure. The reduced demands allow the superstructure to remain elastic, or nearly elastic, following a design level event. Furthermore, the reduced demands minimize the likelihood of damage to displacement sensitive and acceleration sensitive equipment, nonstructural components, and content. The period shift, however, does result in increased displacement demands concentrated at the isolation interface that must be accommodated by the seismic isolation devices. The simultaneous reduction in acceleration and drift demands achieved with seismic isolation makes it one of the most effective strategies to achieve "Operational" or "Fully Operational" performance following a large and infrequent earthquake event.

Though the concept of seismic isolation dates back more than one hundred years, it has only been practiced in the United States for the last three decades. The first record of seismic isolation is an 1870 U.S. Patent filed in San Francisco for a double concave rolling ball bearing, described as an "Earthquake-proof building" [1,2]. The 1870 double concave rolling ball bearing is remarkably similar to modern double concave Friction Pendulum™ bearings [2,3]. In 1985, one hundred and fifteen years later, the Foothill Communities Law and Justice Center in Rancho Cucamonga, California was the first seismically isolated building constructed in the United States [4]. The Law and Justice Center is isolated on 98 high damping natural rubber bearings. A thorough historical perspective and chronology of seismic isolation can be found in Naiem and Kelly [4]. Construction of seismically isolated buildings has increased at an almost exponential rate since the 1980s in Japan and China, while construction in the United States has remained modest [5] irrespective of the demonstrated effectiveness of this technology for protecting structures, nonstructural components, and content from horizontal earthquake ground shaking through more than thirty years of research. In addition to buildings, seismic isolation has been used for the protection of critical, non-building, structures such as bridges, liquefied natural gas (LNG) tanks, and offshore platforms [6]. However these applications are not discussed further as the scope of this paper is on seismic isolation of buildings.

A number of sources, both recent and dated, have provided comprehensive reviews of various aspects of the development, theory, and application of seismic isolation technology. Early reviews were excellent and thorough. For instance, Kelly [7] provided a historical perspective dating back to the rudimentary beginnings of seismic isolation technology, followed by a complete chronology of research and development efforts. Buckle and Mayes [8] also included a thoughtful historical discussion as well as a comprehensive list of the early applications that paved the way for acceptance and wider adoption. Taylor *et al*. [9] presented a review of the use of elastomers in seismic isolation bearings, with emphasis on their long-term behavior. A mid-1990s report provided information on several subtopics including theory, experiments, and application of sliding bearings, hybrid testing, and development and practice in several countries [10].

As the volume of information on seismic isolation has grown exponentially over the past 10–15 years, the attempts at a comprehensive review have diminished. Yet, several focused reviews have emerged. Kunde and Jangid [11] prepared a comprehensive review of research and application of seismic isolation to bridges, including analytical, experimental and parametric studies. Symans *et al*. [12] reviewed the development and application of seismic isolation and damping systems for wood frame structures, which are uniquely challenging to isolate due to the inherent flexibility of the framing system and relatively light mass. A recent primer, developed by the American Society of Civil Engineers (ASCE) [13], discusses the theory, hardware, analysis, design, and testing requirements specific to the United States. Finally, a collaboration of international experts acting as part of the International Council for Research and Innovation in Building and Construction (CIB) Task Group 44, prepared a report comparing devices, design codes, and current state of seismic isolation practice among countries that have been forward in adoption of seismic isolation technology [14].

Complete protection of a building system poses some unique challenges, which include: (1) protection of the nonstructural components and content, which are more sensitive to vertical excitation than the structural system; (2) mitigation of local uplift or tension demands in the isolation system that may be generated by overturning forces in moderate to slender structures; and (3) shifting of displacement demands from the isolation system to the structure in extreme events that can lead to superstructure yielding. While the vast majority of shake table tests have been conducted to demonstrate the effectiveness of seismic isolation to provide a linear elastic response of the structure under horizontal ground motion, only a few experimental studies have focused on the advanced aspects mentioned above.

The current volume of literature precludes a comprehensive review in a single paper. Instead, the aim of this paper is to summarize the current practice, describe widely used seismic isolation hardware, chronicle the history and development of modern seismic isolation through shake table testing of isolated buildings, review past efforts to achieve three-dimensional isolation, and discuss recently identified research needs. This article focuses on the application of passive seismic isolation for buildings primarily as is practiced in the United States, though systems used in other countries will be discussed. The paper is organized into the following major sections: seismic isolation hardware, experimental demonstration, past efforts to achieve 3D seismic isolation, and research needs.

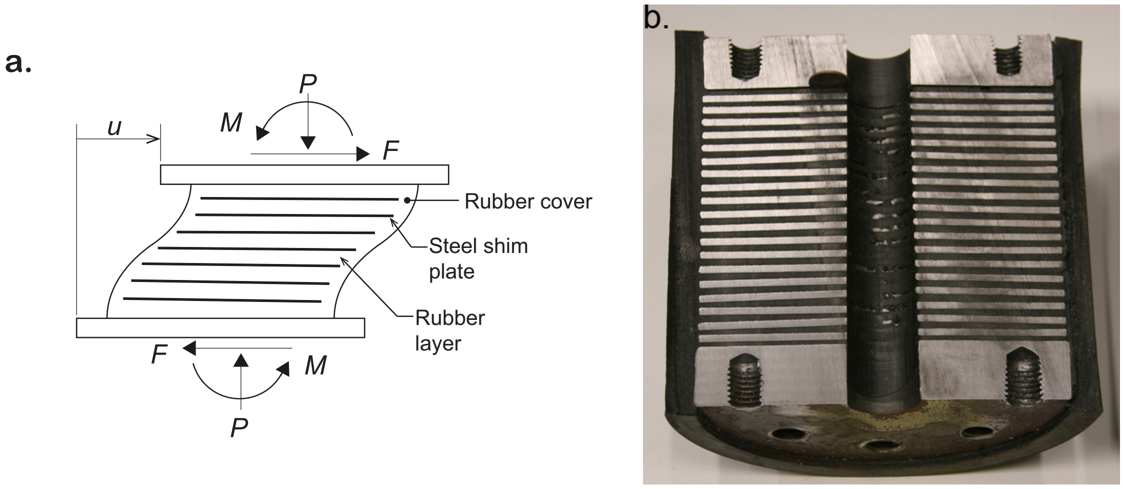
2. Seismic Isolation Hardware

A variety of seismic isolation bearings have been developed and implemented around the world for the seismic protection of structures. In this section, a brief description of the basic construction, mechanical behavior and analytical modeling is presented for each type of bearing. In general, bearings are classified into two categories: (1) elastomeric and (2) sliding.

2.1. Elastomeric Bearings

Elastomeric bearings are composed of alternating layers of natural, or synthetic, rubber bonded to intermediate steel shim plates as shown in Figure 1. The bearings are constructed by placing un-vulcanized rubber sheets and steel shims in a mold, then subjecting the mold to elevated temperature and pressure to simultaneously vulcanize and bond the rubber. A rubber cover is provided to protect the internal rubber layers and steel plates from environmental degradation due to ozone attack [15] and corrosion, respectively. Elastomeric bearings can be categorized as: (1) low-damping natural or synthetic rubber; and (2) high-damping rubber. Low-damping natural rubber material exhibits nearly linear shear stress-strain behavior up to, approximately, 150% shear strain, after which the material stiffens. Natural rubber with type A durometer hardness of 50 is typically used for seismic applications having a shear modulus (G) that ranges from 0.65 MPa to 0.9 MPa. The equivalent damping ratio ξ, for low-damping natural rubber ranges between 2% and 3% at 100% shear strain. To control, or limit, displacements across the isolation interface, external supplemental damping devices such as yielding steel bars, plates, or viscous fluid dampers are typically used in parallel with low-damping natural rubber bearings. A higher level of damping is achieved through the addition of carbon black and other fillers to the raw rubber during the mixing process to produce high-damping rubber bearings [4]. The equivalent damping ratio of high-damping rubber bearings can range from 10% to 20% at 100% shear strain. Though the range of shear modulus for high-damping rubber is similar to that for low-damping rubber, fillers increase the hardness and thus the shear modulus of the rubber so that it can be difficult to achieve low shear modulus and high levels of damping.

Figure 1. (**a**) Illustration of elastomeric bearing in the horizontally deformed configuration; (**b**) Photograph of elastomeric bearing cross-section.



The total thickness of rubber (T_r) provides the low horizontal stiffness needed to lengthen the fundamental natural period of the system, whereas the close spacing of the intermediate steel shim plates provides a large vertical stiffness and critical load capacity for a given G and bonded rubber area (A_b). However, the steel shim plates have no effect on the horizontal stiffness of the bearing, calculated as:

$$K_h = \frac{GA_b}{T_r} \quad (1)$$

The steel shims restrain the rubber at the bond interface and the spacing of the shims (or individual rubber layer thickness) controls the bulging around the perimeter and thus the compression modulus of the elastomeric layer [16,17]. For example, the compression modulus for an individual, solid, circular rubber layer, assuming the rubber is incompressible, is:

$$E_c = 6GS^2 \quad (2)$$

where S is the shape factor, a dimensionless geometric parameter, defined for a single rubber layer as:

$$S = \frac{\text{Loaded Area}}{\text{Area free to bulge}} \quad (3)$$

Although bulk compressibility was not considered in Equation 2 for simplicity, Chalhoub and Kelly [16] demonstrated that the assumption of incompressibility overestimates the compression stiffness of an elastomeric bearing, where the degree of over-estimation increases with increasing shape factor. Based on their work, Chalhoub and Kelly [16] concluded the effect of bulk compressibility should be considered in the analysis of seismic isolation. The close spacing of steel shim plates, *i.e.*, thin rubber layers, produces a high shape factor that in turn results in a large vertical stiffness:

$$K_v = \frac{E_c A_b}{T_r} = \frac{6GS^2 A_b}{T_r} \tag{4}$$

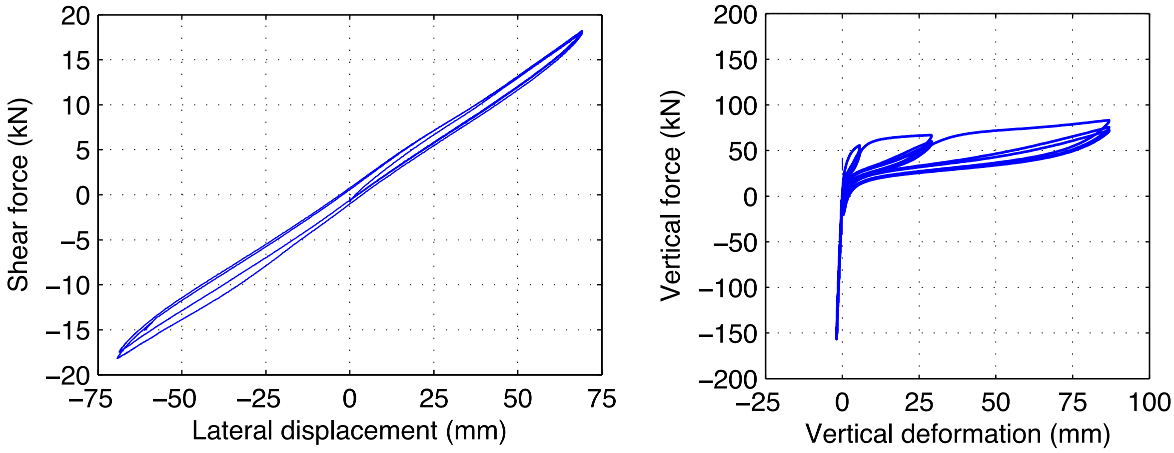
Inspection of Equations 1,4 reveal the horizontal and vertical stiffness of the elastomeric bearing are related. For example, for circular bearings:

$$\frac{K_v}{K_h} = 6S^2 \tag{5}$$

Traditionally, elastomeric seismic isolation bearings have been designed with shape factors from 15 to 25 [18,19] and as high as 30 [20]. Such high shape factors have been used for the historically perceived benefits of mitigating rocking motion and enhancing stability [21,22], among others. However, according to Equation 5, this range of shape factors results in vertical stiffness that ranges from one to several thousand times greater than the horizontal stiffness. As a result, isolation systems composed of bearings detailed with high shape factors (15–30) typically have vertical isolation periods ranging from 0.03 to 0.15 second. Consequently, the vertical isolation frequencies can align with the dominant frequencies of the vertical spectrum, leading to significant amplification of the vertical acceleration. Therefore, isolation systems composed of elastomeric bearings detailed with high shape factors, e.g., 15–30, provide only horizontal isolation.

Figure 2 presents the shear force-horizontal deformation and vertical force-vertical deformation response of a low-damping natural rubber bearing [23]. While damping in natural rubber bearings is neither hysteretic nor viscous the shear force-horizontal deformation relationship is often modeled using either: (1) a linear viscous representation; or (2) a bilinear hysteretic representation. In Figure 2b, positive vertical force corresponds to tension. The vertical force-deformation behavior is highly nonlinear in tension due to softening or loss of stiffness resulting from multi-chain damage, damage of the micro-structure, and micro-void formation in cross-link polymers subjected to tensile strains [24,25]. Uplift, or tension, in elastomeric bearings is considered undesirable and efforts are made in the design process to avoid uplift on the isolation system. Therefore a simple linear force-deformation relationship is typically used to model the vertical force-deformation behavior where the stiffness is specified as the compression stiffness calculated using Equation 4, or similar, for different bearing plan geometries. If uplift cannot be avoided the vertical force-deformation relationship plotted in Figure 2b can be approximated with user-defined, multi-spring, models or, for example, a self-centering material model available in OpenSees [26], an open source earthquake simulation software.

Figure 2. (**a**) Cyclic shear force-horizontal deformation response of low-damping natural rubber bearing; (**b**) Cyclic vertical force-deformation response of low-damping natural rubber bearing.



(a) **(b)**

2.2. Lead-Rubber Bearings

Lead-rubber bearings were first introduced and used in New Zealand in the late 1970s [27,28]. Since then, lead-rubber bearings have been widely used for seismic isolation around the world including the United States and Japan [29]. From a construction perspective, lead-rubber bearings differ from low-damping natural rubber bearings only by the addition of a lead-plug that is press-fit into a central hole in the bearing. The lead-plug deforms plastically under shear deformation, enhancing the energy dissipation capabilities compared to the low-damping natural rubber bearing.

The horizontal force-deformation relationship of a lead-rubber bearing is characterized using bilinear behavior as shown in Figure 3. The zero-displacement force-intercept, Q_d, for a lead-rubber bearing is controlled by the yield strength of the lead in shear, σ_L, and the cross-sectional area of the lead-plug, A_L, or:

$$Q_d = \sigma_L A_L \tag{6}$$

The second-slope stiffness, K_d, is the stiffness of the elastomeric component of the bearing, determined using Equation 1. At a given horizontal displacement, d, the effective, or secant stiffness, of the lead-rubber bearing is:

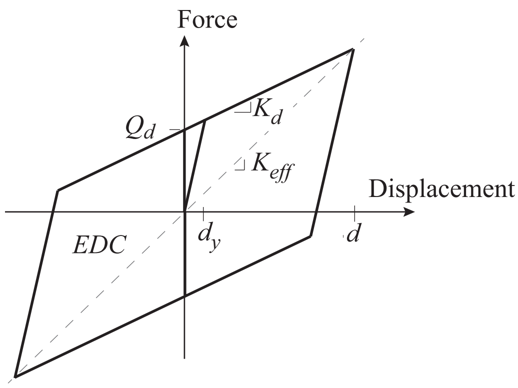
$$K_{eff} = \frac{Q_d}{d} + K_d \tag{7}$$

The vertical stiffness of lead-rubber bearings is calculated from Equation 4 using the effective shear modulus, G_{eff}, calculated from the effective stiffness, K_{eff}, to approximately account for the increase in vertical stiffness from the lead-plug. Similar to elastomeric bearings, the vertical stiffness of a lead-rubber bearing is typically thousands of times larger than the horizontal stiffness so that isolation systems composed of lead-rubber bearings provide isolation only from the horizontal components of ground shaking.

The energy dissipation mechanism is primary hysteretic due to plastic deformation of the lead core. A Bouc-Wen or rate independent plasticity model [30] is typically used for analytically modeling the shear force-horizontal deformation response of lead-rubber bearings. Under bi-directional loading, the bearing model is coupled in the two orthogonal horizontal directions through a circular yield surface. The Bouc-Wen based hysteretic models, however, do not account for the effect of heating in the lead-core with repeated cycling that leads to degradation in the characteristic strength [2,31–33]. Theoretical models to account for the effects of heating in lead-rubber bearings have been developed and experimentally verified [31,32]. However, thermo-mechanical models [33] that account for heating and strength degradation in lead-rubber bearings have not been widely implemented for the analysis and design of lead-rubber isolation systems. Kalpakidis and Constantinou [34] developed a theory of scaling based on similitude and provide recommendations for testing reduced scaled bearings to properly account for the effects of heating.

The vertical force-deformation behavior is typically assumed to be linear with stiffness equal to the compressive stiffness of the bearing (Equation 4), though multi-linear models are possible as previously discussed. A model that includes the influence of vertical load on the effective horizontal stiffness and lead-core yield strength was developed by Ryan *et al*. [35] and has been implemented in OpenSees [26]. Models that account for the second-order effects due to vertical load at large horizontal displacement have been developed [36–38] and are capable of exhibiting zero or negative tangential horizontal stiffness as has been experimentally demonstrated [39]. However, these models have not been widely adopted due to the extensive experimental data required to calibrate the model parameters.

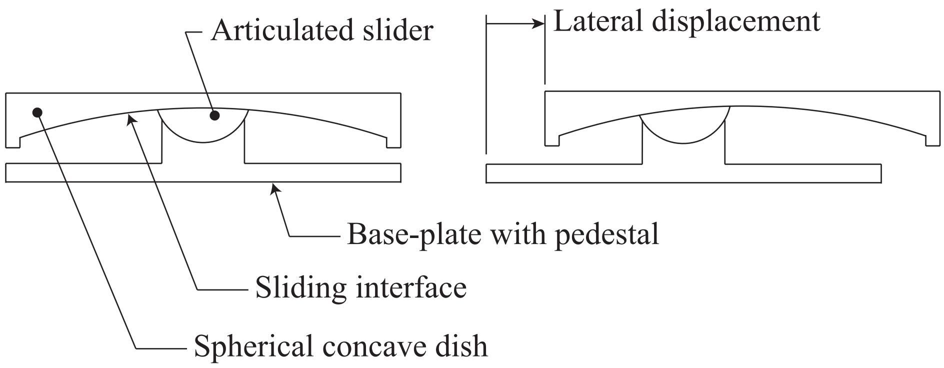
Figure 3. Bilinear horizontal force-displacement characterization of a seismic isolation bearing.



2.3. Sliding Bearings

Sliding bearings support the weight of the structure on a bearing that rests on a sliding interface. The sliding interface is designed with a low coefficient of friction, which limits the resistance to horizontal forces. Most sliding bearings use polytetrafluorethylene (PTFE) type material and stainless steel for the bearing material at the sliding interface. Restoring force is provided either by added springs or through geometry as with the Friction Pendulum™ (FP) bearing [40]. The FP bearing or derivatives such as the multi-spherical Friction Pendulum™ bearings [3,41–43] are among the most widely used seismic isolation bearings in the United States. The single FP bearing consists of a base-plate (ductile iron), an articulated slider (ductile iron with bonded PTFE type bearing material) and a spherical concave dish (cast steel with stainless steel overlay) as illustrated in Figure 4. As shown in Figure 4, under horizontal motion the spherical concave dish displaces horizontally relative to the articulated slider and base-plate. Friction between the PTFE type material and stainless steel surface provides frictional resistance and energy dissipation, whereas the radius of curvature of the spherical concave dish provides a restoring force. The FP bearing can be installed upside down from the configuration shown in Figure 4. However the upside down configuration results in the *P-Δ* moment being distributed to the structural element below the FP isolator rather than the element above the FP isolator as would occur with the configuration shown in Figure 4.

Figure 4. Illustration of Friction Pendulum™ bearing.



The shear force-horizontal deformation behavior of FP bearings is characterized using the bilinear relationship shown in Figure 3. The horizontal strength, or zero-displacement force-intercept, Q_d, is controlled by the bearing material and the weight W carried by the isolators, according to:

$$Q_d = \mu W \tag{8}$$

Where μ is the sliding coefficient of friction of the bearing interface. The sliding coefficient of friction for un-lubricated Teflon type material mated to stainless steel typically ranges between 0.07–0.18 depending on bearing pressure, peak velocity and material [44]. However, a bearing manufacturer reports that sliding coefficients of friction from 0.03 to 0.2 are possible [45]. The second-slope stiffness, K_d, of the FP bearing is controlled by weight acting on the isolator and the radius of curvature, R, of the spherical concave dish according to:.

$$K_d = \frac{W}{R} \tag{9}$$

The effective stiffness of an FP bearing can be determined by substituting Equations 8,9 into Equation 7. The FP bearing is unique in that the period based on the second-slope stiffness, K_d, is controlled only by the radius of the concave dish:

$$T_d = 2\pi\sqrt{\frac{R}{g}} \tag{10}$$

The significance of the weight independent property is that FP bearings can be effective for isolating light-weight structures. Furthermore, mass irregularities are naturally balanced by corresponding spatial variation in the restoring force, such that torsional response is minimal.

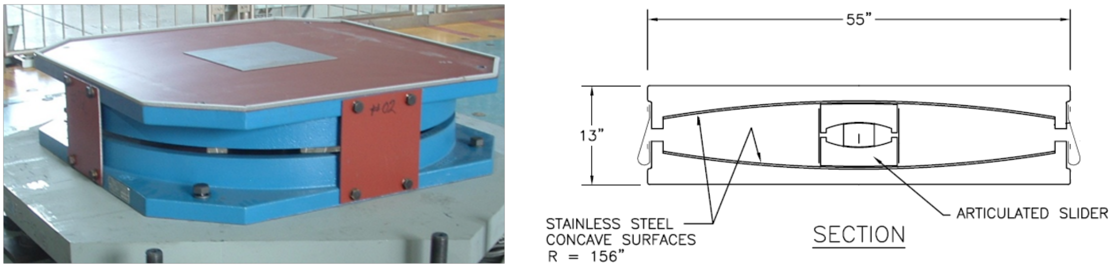
Similar to lead-rubber bearings, a bi-directionally coupled, Bouc-Wen or rate independent plasticity model is commonly used to analytically describe the force-deformation behavior of FP bearing undergoing horizontal planer motion [30]. The influence of bearing pressure and instantaneous velocity on the coefficient of sliding friction are considered using a model proposed by Constantinou *et al*. [46]. Similar to lead-rubber bearings, the characteristic strength of friction pendulum bearings is affected by heating [2]. The coefficient of friction is affected by the ambient, or bulk, temperature of the isolators as well as heating that develops at the isolation interface during high velocity sliding [2]. Heating due to sliding is directly related to the sliding velocity and must be carefully considered when designing reduced scale tests or extrapolating the results of characterization tests performed on reduced scale bearings to those of the prototype [2].

The vertical stiffness of FP bearings is several times larger than high shape factor elastomeric bearings due to the ductile iron, stainless steel and thin layer of PTFE used to construct the bearing. Typically the vertical stiffness of FP bearings is estimated using a simple axial stiffness equation where the modulus is that of ductile cast iron, the area is that of the articulated slider and the length is that of the combined height of the inner pedestal and articulated slider. The resulting vertical isolation period is around 0.03 seconds [45]. Therefore, isolation systems composed of FP bearings with vertical isolation periods around 0.03 seconds provide only horizontal isolation.

More recently, multi-spherical derivatives of the FP bearing have been developed, characterized [3,41–43] and implemented, e.g., [47]. The benefit of the multi-spherical sliding bearings over single FP bearings is that they allow an adaptive force-deformation behavior, whereby the stiffness and damping properties of the bearing can change at predetermined displacement amplitudes [41]. Variations of multi-spherical sliding bearings include the Double Pendulum™ (DP) and Triple Pendulum™ (TP), among others. Details of the DP bearing can be found in Fenz and Constantinou [3]. The TP bearing, consisting of four spherical sliding surfaces and three independent pendulum mechanisms, offers the most adaptable behavior of the multi-spherical sliding bearings [41–43]. A photograph of a TP bearing used in the NEES TIPS/E-Defense tests [48] is shown in Figure 5a with a cross-section view in Figure 5b. The internal pendulum mechanism with two concave plates and a rigid slider, denoted collectively as the “articulated slider” (Figure 5b), determines the response during low intensity shaking. The outer stainless steel concave surfaces, when designed with different curvatures and friction coefficients, provide two independent pendulum mechanisms that determine the response during medium to high intensity shaking. Details pertaining to the construction and force-deformation behavior of the TP bearing can be found in Fenz and Constantinou [41].

The force-deformation behavior of the TP bearing can be modeled analytically as three single FP elements in series [49]. Each of the individual series elements is modeled using the Bouc-Wen type plasticity model developed for the single FP bearing [30]. The vertical force-deformation behavior of the Triple FP bearings is assumed to be linear [49], and calculated according to the dimensions of the inner-most rigid slider.

Figure 5. (**a**) photograph of Triple Pendulum™ bearing; (**b**) view of cross-section.



(a) **(b)**

Both the single FP and the TP bearings provide no resistance to tensile forces and thus are free to uplift. In certain situations uplift in the bearings could occur, e.g., bearings on the perimeter of slender structures or those located under braced frames. For these situations resistance to uplift is considered desirable and a tension-capable sliding bearing, based on the principles of the single FP bearing, was developed [50,51]. The tension-capable device, referred to as the "XY-FP" isolator, consists of two orthogonal concave sliding rails connected through a sliding device that couples the rails in the vertical direction, allowing for the development of tensile forces in the bearing. The XY-FP isolator uniquely provides uplift resistance and an uncoupled bi-directional response [52] due to the separate orthogonal rails.

3. Experimental Demonstration

Extensive shake table testing of base-isolated buildings has been conducted over the past 30 plus years, which has paralleled the development of suitable isolation devices for large scale structures and the evolution of base isolation practice in the United States and other countries. The earliest tests focused on verification of different isolation devices and served mostly as proof-of-concept tests, without rigorous standards for evaluation of the building response. Elastomeric bearings matured more quickly than friction-based sliding systems. Several of the earliest systems were evaluated by shake table testing at the University of California's Earthquake Engineering Research Center (EERC). These systems included lightly damped elastomeric bearings, elastomeric bearings with steel dampers [53], and elastomeric bearings with enhanced damping through a friction fail-safe device [54,55]. The lead-rubber bearing was developed in New Zealand [28], and evaluated also by shake table testing at EERC [56]. All systems were tested on a 5-story frame structure, allowing the development of higher mode response, which enabled the effectiveness of various implementation approaches for seismic isolation to be assessed. These research reports frequently observed that isolation systems with higher levels of damping, especially nonlinear damping, were effective at controlling isolator displacements, but increased floor accelerations and high frequency response.

While the concept of a friction-based system was very simple and attractive, the lack of a suitable restoring force delayed the development of sliding systems. Kelly and Chalhoub [57] tested an isolation system utilizing a combination of elastomeric bearings and flat sliders. The now mature FP system was first tested on a 2-story frame with the isolators located at the top of the first story columns; variations in structural height and aspect ratio were considered [40]. Around this time, several concepts for incorporating restoring force into friction base isolation systems were proposed [58], and an era of shake table testing of friction-based systems was launched at the State University of New York at Buffalo's National Center for Earthquake Engineering Research. A 6-story frame structure was used in evaluation tests of FP isolators [59–61] and teflon-disc bearings with helical steel springs [62,63]. Also, a hybrid elastomeric-friction device called resilient friction base-isolator (R-FBI) was tested at EERC [64,65]. All these systems were found to provide effective horizontal isolation.

Alongside the research and development in the United States, a major interest in base isolation developed in Japan, with research carried out mainly in the private sector by the most reputable construction companies [66]. Since Japan is an earthquake-prone region whose construction industry is dominated by a few companies, those companies have and continue to invest in the development of new technologies, including base isolation. Although some private companies and/or government agencies utilized shake table testing (e.g., Kajima, Mitsubishi, Nuclear Power Engineering Test Center), an alternative form of development adopted in Japan was the concept of a demonstration building. Kajima, Oiles, Shimizu, Obayashi, Takenaka, Taisei and Okumura all constructed base-isolation demonstration buildings, many of which were used as laboratory facilities on corporate research campuses. These demonstration buildings were continually monitored by forced vibration, free vibration, and multiple instances of seismic shaking every year, albeit mostly small intensity, as described and referenced in Kelly [66]. Most companies focused on natural rubber bearings with steel dampers, lead-rubber bearings, and eventually the application of rubber compounds with high damping fillers that could provide sufficient damping in a single device to eliminate the need for supplemental damping. Bridgestone emerged as one of the successful developers of the high damping rubber bearing, and also popularized the bolted connection detail, which was shown to improve stability and eventually replaced the doweled connection [20]. Taisei Corporation developed the friction based Taisei Shake Suppression System (TASS) that used TFE elastomeric bearings (bearings with a sliding interface) and Neoprene horizontal springs for a restoring force [67].

As the years have progressed, researchers have continued to invent new concepts for base isolation, many of them taken to the point of successful validation with shake table testing. While an exhaustive treatment of this subject is nearly impossible, a few examples are: (1) suspended pendulum isolator (SPI) [68]; (2) rolling ball isolation [69]; (3) almost lifted structure concept (ALSC) [70]; (4) stable unbonded fiber reinforced elastomeric isolator (SU-FREI) [71]; and (5) core suspended isolation system (CSI) [72]. Few of these systems have advanced to the point of widespread practical implementation. However, the new variations of the original FP isolator described earlier, including the tension capable bearing and the TP bearing, have achieved market acceptance and international implementation. The tension capable bearing was tested and validated by Roussis and Constantinou [50], and two comprehensive programs have evaluated the performance of multi-stage FP devices [73,74]. Some specifics of these test programs are mentioned below.

A few experimental studies focused specifically on evaluation of secondary system response [75–78]. The studies by Kelly [75] and Juhn *et al*. [77] designed oscillators (vertical cantilevers with mass) mounted on various floors of the frame to simulate the response of light equipment. Kelly [76] found that supplementary damping reduced the benefit of isolation on the secondary system response (floor acceleration and especially floor spectra), while Juhn *et* al. [77] found that the isolation system was very effective in reducing the response of an oscillator tuned to the natural frequency of the fixed-base structure, but could slightly increase the response of the high frequency detuned oscillator. Wolff and Constantinou [78] focused on secondary system response by evaluating floor accelerations and floor spectra for 8 different isolation systems, including combinations of low damping elastomeric bearings, lead-rubber bearings, friction pendulum bearings, and flat sliders with and without viscous dampers. The authors concluded that highly nonlinear systems incorporating friction pendulum bearings or flat sliders were not as detrimental to content response as suggested by earlier research, e.g., [29]. However, meaningful evaluation of the effect of nonlinearity is difficult when the isolation systems considered have significantly different isolation periods, thus providing different levels of force reduction, which was the case in these experiments. While useful, these studies lack a connection to the physical performance of realistic nonstructural components and content that appears in building structures.

Eventually, shake table experiments extended beyond basic validation to examine performance of the overall isolated building under more challenging loading conditions. One consideration was the possibility of uplift in slender structures under large horizontal loading. The effect of column uplift was examined in a 1/5 length scale 7-story R/C isolated structure [79] and a 1/4 length scale 9-story braced-steel frame isolated structure [80]. Different types of elastomeric bearings (natural rubber or lead-rubber) were designed for each test, and doweled connections were used to allow shear transfer while providing no restraint against uplift. The studies found that an uplift occurrence caused a vertical pulse to propagate through the structure, and likewise increased the higher frequency peaks in the horizontal floor spectra. The uplift occurrence sometimes produced a temporary instability in one or more bearing hysteresis loops, but this was a localized effect that was balanced by the other bearings and did not lead to global instability. In one case, the shear key disengaged from the bearing since the uplift of 1.25 cm, which exceeded the height of the shear key, occurred at the same time as the peak bearing displacement [79]. In tests utilizing softer bearings, a combination of moderate uplift and bearing roll out failure was observed, which occurred at a horizontal displacement less than the predicted roll out displacement. The tests suggested that uplift should be anticipated and accounted for in superstructure response and bearing connection details.

To minimize the effects of building overturning, researchers have developed and tested various uplift restraint mechanisms or mechanisms that carry tension. A thorough review of uplift restraint mechanisms studied to date is provided by Roussis [52]. Occasionally, uplift restraint has been provided by a mechanism independent of the isolation device. For example, Sumitomo Construction developed a mechanism—untested but implemented—to effectively tie the building down through interconnected steel arms attached to the structure and the foundation [52]. More commonly, uplift restraint mechanisms have been incorporated into the isolation device. An uplift restraint device embedded in the central hole of an annular elastomeric bearing was proposed and tested [81,82]. The restraint consisted of high strength bolts contained within a sleeve, wherein the bolts were detailed to allow vertical slip. The device could engage in tension and/or during large horizontal displacements to act as a fail-safe displacement restraint. The combined horizontal-vertical restraint provided enhanced functionality to the isolator, but ultimately limited the horizontal displacements that could be accommodated. An uplift restraint for a flat slider or FP isolator was tested by Nagarajaiah *et al*. [83]. The restraint was provided by L-shaped arms that extended from and engaged the underside of the isolator on two sides. The approach was demonstrated for unidirectional excitation, but is difficult to extend to bidirectional excitation and thus has limited application. The use of pre-stressing tendons was evaluated by Kasalanati and Constantinou [84]. Another solution is the tension capable XY-FP bearing mentioned earlier [50,51] that has been implemented in several applications. An alternative practical solution is to pair elastomeric bearings with tension capable cross-linear bearings [85]. The cross-linear bearings, developed in Japan, are essentially low friction sliders that use a double rail mechanism for tensile resistance. The friction coefficient in the cross-linear bearing is minimized by use of linear guides with recirculating ball technology. Shake table verification of this technology was a component of recent tests performed at E-Defense [48], that are described below.

Another project used shake table testing to engage the ultimate capacity of the isolated structure [86]. A three-story R/C building was designed and built to replicate the performance of a base-isolated demonstration building in Sendai, Japan (length scale factor = 0.4). The high damping rubber isolation system stiffened at large displacements, allowing demands to shift to the superstructure to develop the typical ductile degradation modes expected under large earthquakes. The experiment showed that design strategies can be adopted to ensure that the isolation system is not the weak link. In these tests, the isolators withstood significant tension due to the structure overturning.

At least two studies performed shake table tests where the intensity of the excitation was increased until the bearings ruptured [87,88]. Sato *et al.* [87] tested three different types of isolation devices (natural rubber bearings with steel dampers, lead rubber bearings, and high damping rubber bearings) to failure. Several of the tests were stopped only after all 4 bearings had failed; however, in each case the first failure was reported as rupture of the rubber compound under a combination of tension and shear. The failure shear strains in dynamic tests were found to be greater than the failure strains under static, monotonic loading. Takaoka *et al.* [88] performed shake table tests of a slender 1/9 length scale structure with lead rubber bearings, varying the structure slenderness ratio and the bearing aspect (diameter to height) ratio to engage different bearing failure modes. Bearing rupture was reported under extreme cases of buckling, tension, and combinations of tension and shear. Buckling rupture, accompanied by a "sinking" deformation of the bearing, occurred after the horizontal deformation had exceeded the diameter of the bearings. Large tension occurrences were accompanied by a notable increase in horizontal acceleration at the roof level when the uplifted structure landed on the bearings.

Limit state response of single and multi-spherical FP isolators has been observed in several of the previous test programs, e.g., [40,73,74], although engaging the limit states has never been a main focus of these programs. A short duration uplift occurrence of the bearings on one side of the building was observed in Morgan and Mahin [74], during which the slider assembly remained stable and the bearing hysteresis was not affected other than a localized (single bearing) reduction in horizontal shear due to the coupling of shear and supported axial load through friction. In Fenz and Constantinou [73], an accidental aggressive contact of the slider with the displacement restrainer in a DP bearing appeared to cause the building to rock off its supports on one side. The impact increased the base shear of the system, but not to the point of negating the effectiveness of base isolation relative to a non-isolated building; and the authors hypothesized that the model might have collapsed if a displacement restraining ring had not been present. The authors also suggested that an uplift occurrence could be particularly serious if the separation distance exceeds the height of the restrainer ring, causing the top concave plate to rise over the edge of the intermediate slider. However, such behavior has never been observed, and uplift occurences have typically been short duration, limiting the consequences of bearing separation.

In all tests described above, evaluation of the influence of vertical excitation was not a primary focus. However, many of the tests included a vertical component of excitation, and in some cases the test protocol included direct comparisons of shaking with and without vertical excitation. Most test programs have concluded that vertical excitation has no influence or very minor influence on the horizontal response and the effectiveness of base isolation. In tests with elastomeric bearings, Griffith *et al.* [79] noted a very minor (5%–11%) increase in horizontal floor accelerations with the inclusion of vertical excitation, and suggested that a very light coupling may have occurred, but the data was inconclusive. Responses due to horizontal only and horizontal plus vertical excitation were found to be similar in tests of the R-FBI system [65]. For multi-stage FP bearings, the influence of vertical excitation on the floor response spectra appeared to be negligible in Morgan and Mahin [74], and the influence of vertical excitation was also concluded to be “very minor” in Fenz and Constantinou [73]. Some instances of coupling between the system horizontal response and vertical ground excitation have been observed. For instance, in tests of a 3-story building isolated with lead-rubber and natural rubber bearings, Hwang and Hsu [89] found that for a structure with an asymmetric configured isolation system, vertical ground acceleration significantly increased the horizontal floor accelerations in the superstructure. No attempt was made to explain why the floor accelerations were amplified in the asymmetric configuration. Furthermore, data presented in Fenz and Constantinou [73] showed that peak horizontal floor accelerations were amplified by a factor of 2 in some of the tests with the largest peak vertical ground acceleration, which is in contrast to their conclusions.

All of the aforementioned studies involved reduced scale structural models and reduced scale isolation bearings. The multi-story structural models utilized in previous test programs employed length scale factors ranging from about 0.1 to 0.4, with a maximum supported structural weight on the order of 500 kN. Recently, two independent test programs on full scale isolated buildings with weights of 5,000–10,000 kN have been conducted at National Institute for Earth Science and Disaster Prevention (NIED) E-Defense shaking table of Japan. The first of these test programs evaluated the response of a 4-story R/C hospital building isolated with 4 high damping rubber bearings and 4 natural rubber bearings paired with steel dampers [90]. The second of these test programs, a NEES/E-Defense collaboration, evaluated the response of a 5-story steel moment frame building isolated with (1) 9 triple pendulum bearings (TPBs); and (2) 4 lead-rubber bearings (LRBs) paired with 5 cross linear bearings (CLBs). The results of the NEES/E-Defense collaboration have been reported only preliminarily to date [48].

Several features offered by full scale testing have led to significant new observations from the E-Defense tests even considering the wealth of data on reduced-scale systems. First and foremost, each full scale building served as a testbed for evaluation of physical nonstructural components and content. Thus, the effectiveness of seismic isolation to protect a variety of building content was evaluated directly, rather than by speculation of the influence of recorded responses, such as floor accelerations. In fact, the primary objective of the hospital test program [90] was to examine the performance of hospital equipment, and evaluation of the seismic performance of interacting partition walls, suspended ceilings, and sprinkler piping was a key component of the NEES/E-Defense tests [91]. Second, full scale models are required to realistically reproduce the effects of vertical vibration of the floor system and thus assess the influence of vertical excitation on building content. In most prior reduced scale tests, supplementary mass was bolted directly to a bare frame specimen, causing significant distortion of the vertical vibration modes and frequencies.

Based on observations from the hospital test program, functionality of base-isolated hospitals utilizing elastomeric isolation systems can be achieved following a near-fault motion. However, the same cannot be said for a long duration, long period ground motion generated from a subduction earthquake, due to significant motion of furniture items and medical appliances supported by casters. In such motions, velocities measured in the floors of the isolated building were larger than those in the comparable fixed-base building. The tests have generated serious discussion in Japan about how to maintain the function of a base-isolated hospital after a large, long-duration, earthquake. The influence of vertical excitation in this test program has not yet been reported [90].

The significant influence of vertical excitation was a dominant outcome of the NEES/E-Defense test program [48]. Input table excitations with peak vertical accelerations greater than about 0.5g resulted in some damage to suspended ceilings, causing ceiling panels to fall, or in the most extreme cases localized failures of the ceiling grid support system. The performance of the suspended ceilings was similar in all configurations (isolated with TPB, isolated with LRB/CLB, or fixed-base), and the damage seemed to be a direct result of the vertical acceleration recorded in the slabs, which was also similar in all configurations. In the building isolated with TPB, an occurrence of near simultaneous uplift of all 9 TPBs (*i.e.*, bouncing of the entire building on the shake table) induced by a large vertical pulse with PGA = 1.3g. While the input sent a high frequency shock wave up through the columns, it was not transmitted to the slabs, as the slab vibration was dominated by single frequency vibration at the slab natural frequency. This suggests that the vertical motion transmitted to the nonstructural components and content is relatively insensitive to the base conditions (complete fixity, isolators with tensile restraint, or isolators free to uplift). Amplification of horizontal floor accelerations was observed in all configurations when subjected to combined horizontal/vertical table acceleration, although the extent to which this contributed to ceiling damage and content disruption is unclear. The amplification of horizontal acceleration was caused by at least two sources of horizontal-vertical coupling: coupling in the mode shapes due to building irregularities, and coupling in the TPB isolators (horizontal shear is proportional to vertical force). Though investigations are ongoing, the observations overwhelmingly suggest that mitigation of the effect of vertical component of excitation (e.g., 3D isolation, engineered floor systems, improved anchorage of nonstructural components and equipment) may be required to maintain functionality in essential base-isolated buildings.

4. Development and Testing of 3-Dimensional Isolation Systems

Efforts to combine horizontal seismic isolation with vertical vibration isolation date back to the early development of seismic isolation systems, driven by the power and utility industries focused on protection of large equipment. Multiple researchers have independently developed 3-dimensional isolation systems by modifying the design parameters of laminated rubber bearings. As mentioned previously, the bearing shape factor, controlled by the thickness of the individual rubber layers, determines the vertical stiffness of the system [92]. Hence, a natural approach to provide vertical isolation is to decrease the bearing shape factor (see Equation 4) in order to increase the vertical fundamental natural period. The earliest isolation system developed by Kajima in Japan utilized steel laminated natural rubber bearings that provided a vertical frequency = 5 Hz, which is substantially more vertically flexible than a typical modern design. Oil dampers were included to reduce vertical and rocking motions, and a 2-story R/C acoustic laboratory demonstration building was constructed using this approach [66]. The use of laminated rubber bearings for 3-dimensional isolation was also explored for the United States nuclear industry by Aiken *et al.* [93]. The research progressed to the point of designing a 3D isolation system for a nuclear facility and performing characterization tests of individual 1/4 length scale bearings [94]. The target horizontal and vertical periods of the system were 2 seconds and 0.33 seconds, respectively. The investigation concluded that elastomeric isolators could possibly be designed to effectively provide isolation in the horizontal and vertical directions [93,94]. Application of 3D isolation to elastomeric devices is by nature limiting due to stability issues. As the thickness of the rubber layers increases, the critical load capacity of the bearing P_{cr} decreases proportionally. Furthermore, P_{cr} has been experimentally demonstrated to reduce with increasing horizontal displacement [22,95–97]. For a lightweight building, balancing the design objectives for displacement capacity and stability in the deformed configuration can be extremely difficult for elastomeric bearings even without consideration of vertical isolation. Thus, successful implementation of 3D isolation using elastomeric bearings to any building would likely constrain the horizontal isolation period to be somewhat smaller than typically used today.

The GERB system for 3-dimensional earthquake protection of structures was developed by a company that specializes in vibration isolation [98]. The system utilizes helical springs—with similar flexibility in all three directions—and viscous dampers, and was developed for seismic and vibration isolation of diesel and turbo generators. Shake table testing was performed on a 5-story building model in Yugoslavia [98], and the system was also tested by Mitsubishi Industries in Japan [66]. The system can be designed with a vertical fundamental frequency as low as 1.2 Hz, leading to a flexible rocking mode around 0.65 Hz. The authors acknowledged that horizontal floor accelerations increased with the 3D isolation system compared to horizontal only seismic isolation, but argued that the responses were acceptable. A residential building in California isolated with the GERB system was shaken strongly in the 1994 Northridge Earthquake [99]. The maximum recorded floor acceleration was about 0.63g, relative to input PGA < 0.5g; thus the horizontal isolation was less effective than a typical isolation system. Observed vertical ground acceleration was not significant (PGA ≈ 0.1 g).

Prior to the disaster at Fukushima in the 2011 Great East Japan Earthquake and Tsunami, Japan actively pursued 3D seismic isolation approaches for nuclear facilities. Several approaches have been developed that use air springs or pressurized air in the vertical direction [100]. The rolling seal type air spring is a steel/concrete cylinder lowered into an air cavity and attached with a rolling rubber seal, and is configured in series with a laminated rubber bearing for horizontal isolation [101,102]. The cable reinforced air spring consists of an inner cylinder attached to the base and an outer cylinder attached to the structure separated by an air cavity bounded by a flexible rubber sheet [103,104]. The distance between the cylinders allows the device to move both horizontally and vertically and thus accommodate horizontal and vertical motion. Also used in series with laminated rubber bearings, the hydraulic system consists of load carrying hydraulic cylinders filled with nitrogen gas, to which fluctuating pressure can be transmitted by the attached accumulator units [105]. As the flexibility of the isolation system in the vertical direction increases, the structure tends to develop a rocking mode with a substantial modal mass participation factor due to increased coupling in the horizontal and vertical directions. The proposed systems have vertical isolation periods on the order of 1–2 seconds, and generally utilize dampers (oil dampers or viscous wall dampers) and rocking suppression devices to control both vertical and rocking displacements.

A commercial solution for 3D isolation is available through Shimizu Corporation in Japan, and has been implemented in at least one 3-story apartment building [106,107]. Each device consists of a laminated rubber bearing on a steel frame that transmits the loads to three air springs and three shear force transmitting vertical sliders. An oil damper system—two oil dampers connected by cross-coupled pipes—provides both vertical and rocking suppression. The system is quite complicated and costly.

5. Conclusions

This review of shake table testing of base-isolated building has shown that a multitude of devices have been experimentally validated to perform as intended, which is to provide high attenuation of the input excitation to allow the structural system to remain elastic under large horizontal ground shaking. Of the many devices that have been tested, a handful of devices (elastomeric bearings, lead-rubber bearings, Friction Pendulum™, and Triple Pendulum™) claim the majority of the market share in the United States and around the world. Sufficient research has been conducted to suggest that base-isolated buildings can be detailed to survive earthquake events larger than anticipated in design, through activation of one or more ultimate limit state behaviors such as: unrestrained uplift, rupture of elastomeric bearings under tension and shear, buckling of elastomeric bearings, engagement of a displacement restraint or large displacement hardening. Many of these ultimate behaviors of the isolation system will redistribute demands to the superstructure, which should ultimately be detailed for ductile response.

The recent shake table tests of full-scale base isolated buildings at E-Defense have provided insight into the performance of the LRB-CLB and TP isolation systems subjected to tri-directional earthquake ground shaking and have underscored existing knowledge gaps regarding the performance of secondary systems in isolated buildings during representative earthquake ground shaking. Specifically, nonstructural components and content were shown to be vulnerable to damage or disruption under long duration motions generating large floor velocities and motions with large vertical components of excitation. Damage and disruption to these systems can adversely affect the post-event functionality of critical base-isolated facilities, and prevent the performance objectives from being achieved. While the tests at E-Defense provided some data on the demands imposed on these systems, the data is limited due to the large cost associated with performing these tests.

The results from testing at E-Defense have several implications. First, improved understanding of the response of nonstructural components and sensitive equipment is needed for analysis/design of base-isolated buildings. Seismic fragilities for nonstructural systems has generally associated damage with a single demand parameter, such as horizontal floor acceleration. However, the testing suggests that observed damage may be more complex, relating to multiple factors such as peak horizontal acceleration, peak vertical acceleration, and frequency or spectral content of the accelerations. The demands imposed on nonstructural components in base-isolated buildings have different proportions of horizontal to vertical response and different frequency content than conventional buildings, and seismic fragility data of nonstructural systems should be developed in such a way that the results are broadly applicable to all types of buildings.

Second, for buildings that aim to provide post-event functionality following a large earthquake, it is clear that vertical ground acceleration needs to be considered in design. Currently, vertical ground acceleration is rarely incorporated into the response history analysis methods that are often used for the design of critical facilities. To effectively consider vertical excitation, the engineering profession is in need of guidance on: (1) selection and scaling of vertical ground motions that are compatible with the horizontal motions; (2) analytical modeling techniques that can be employed in commercial software to accurately reflect local vertical modes and frequencies related to vibration of the floor system; and (3) accurate analytical models for vertical force-deformation behavior. Floor system modeling techniques should be verified with vibration data of full-scale slabs in realistic structural systems under seismic demands. In particular, for steel framing with concrete floor slabs, the influence of slab to girder connection configuration (fully-composite, partially composite, and non-composite) should be considered. With regard to modeling of the bearings, analytical models for the horizontal force-displacement behavior have outpaced the analytical models for the vertical force-displacement behavior. For example, a simple linear model is typically used to represent the vertical force-displacement behavior of elastomeric bearings while the true behavior is highly nonlinear (see Figure 2b). For FP class of bearings, techniques for evaluating compressive stiffness should be verified and techniques to account for impact upon uplift should be developed and experimentally verified.

Finally, closely associated with the need to evaluate the influence of vertical excitation in the design process is the desire to mitigate the influence of vertical excitation through design. Multiple approaches may be possible. For instance, vertical floor vibrations may be reduced in part through improved slab design. A fundamental understanding of the influence of slab vibration characteristics on the vertical vibration demands and nonstructural component response is needed. Furthermore, limited evidence—described in this review—has suggested that structural configuration irregularities (torsional and vertical) have contributed to horizontal-vertical coupling phenomena observed in some tests.

An alternative is to develop a packaged approach to base isolation that can provide both horizontal and vertical attenuation. To support the revival of the United States nuclear industry, base isolation is being investigated as a viable solution to satisfy Nuclear Regulatory Commission (NRC) guidelines [108] for seismic design based on a 100,000 year return period [109], and a packaged 3-dimensional base-isolation solution is considered highly desirable [109]. With vertical stiffness that is several thousand times the horizontal stiffness, current widely used seismic isolation devices provide isolation only in the horizontal direction, and may actually align the system with dominant frequencies of the vertical ground shaking. Past efforts to achieve 3-dimensional isolation have not produced a viable or cost-effective system. Stability considerations preclude low shape factor bearings from being a viable solution as only modest vertical periods, e.g., 0.33 seconds [94], can be achieved. While the Japanese systems might be effective, they are complex, have not been independently verified and their high cost and proprietary nature reduce the likelihood of adoption outside of Japan.

In summary, more research is needed to: (1) develop data on the 3-dimensional response of buildings isolated on current isolation systems; (2) evaluate existing analytical and numerical models of conventional isolation hardware for predicting demands imposed on the primary structural system as well as secondary and nonstructural systems; and (3) develop improved data on the seismic fragility of secondary and nonstructural systems under a variety of demands. Further research could show that complete protection of secondary and nonstructural systems using conventional isolation approaches might not be possible. Recognizing that the integrity of these systems is paramount to the post-event functionality of critical facilities, alternative or new systems to achieve complete 3-dimensional protection might be necessary.

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