# CHAPTER C17
# SEISMIC DESIGN REQUIREMENTS FOR SEISMICALLY ISOLATED STRUCTURES

## C17.1 GENERAL

Seismic isolation, also referred to as base isolation because of its common use at the base of building structures, is a design method used to substantially decouple the response of a structure from potentially damaging horizontal components of earthquake motions. This decoupling can result in response that is significantly reduced from that of a conventional, fixed-base building.

The significant damage to buildings and infrastructure following large earthquakes over the last three decades has led to the rapid growth of seismic isolation technology and the development of specific guidelines for the design and construction of seismically isolated buildings and bridges in the United States, as well as standardized testing procedures of isolation devices.

Design requirements for seismically isolated building structures were first codified in the United States as an appendix to the 1991 Uniform Building Code, based on "General Requirements for the Design and Construction of Seismic-Isolated Structures" developed by the State Seismology Committee of the Structural Engineers Association of California. In the intervening years, those provisions have developed along two parallel tracks into the design requirements in Chapter 17 of the ASCE/SEI 7 standard and the rehabilitation requirements in Section 9.2 of ASCE/SEI 41 (2007), *Seismic Rehabilitation of Existing Buildings*. The design and analysis methods of both standards are similar, but ASCE/SEI 41 allows more relaxed design requirements for the superstructure of rehabilitated buildings. The basic concepts and design principles of seismic isolation of highway bridge structures were developed in parallel and first codified in the United States in the 1990 AASHTO provisions *Guide Specifications for Seismic Isolation Design*. The subsequent version of this code (AASHTO 1999) provides a systematic approach to determining bounding limits for analysis and design of isolator mechanical properties.

The present edition of the ASCE/SEI 7, Chapter 17, provisions contains significant modifications with respect to superseded versions, intended to facilitate the design and implementation process of seismic isolation, thus promoting the expanded use of the technology. Rather than addressing a specific method of seismic isolation, the standard provides general design requirements applicable to a wide range of seismic isolation systems. Because the design requirements are general, testing of isolation-system hardware is required to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Use of isolation systems whose adequacy is not proved by testing is prohibited. In general, acceptable systems (a) maintain horizontal and vertical stability when subjected to design displacements, (b) have an inherent restoring force defined as increasing resistance with increasing displacement, (c) do not degrade significantly under repeated cyclic load, and (d) have quantifiable engineering parameters (such as force-deflection characteristics and damping).

The lateral force-displacement behavior of isolation systems can be classified into four categories, as shown in Fig. C17.1-1, where each idealized curve has the same design displacement, $D_D$.

A linear isolation system (Curve A) has an effective period that is constant and independent of the displacement demand, where the force generated in the superstructure is directly proportional to the displacement of the isolation system.

A hardening isolation system (Curve B) has a low initial lateral stiffness (or equivalently a long effective period) followed by a relatively high second stiffness (or a shorter effective period) at higher displacement demands. Where displacements exceed the design displacement, the superstructure is subjected to increased force demands, while the isolation system is subject to reduced displacements, compared to an equivalent linear system with equal design displacement, as shown in Fig. C17.1-1.

A softening isolation system (Curve C) has a relatively high initial stiffness (short effective period) followed by a relatively low second stiffness (longer effective period) at higher displacements. Where displacements exceed the design displacement, the superstructure is subjected to reduced force demands, while the isolation system is subject to increased displacement demand than for a comparable linear system.

The response of a purely sliding isolation system without lateral restoring force capabilities (Curve D) is governed by friction forces developed at the sliding interface. With increasing

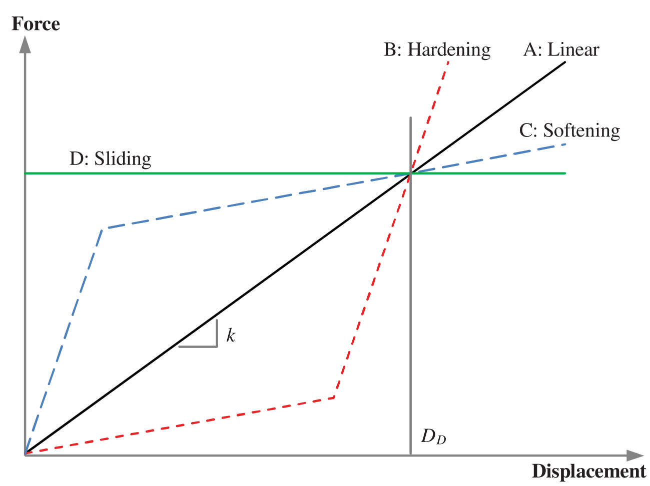


**FIGURE C17.1-1 Idealized Force-Deflection Relationships for Isolation Systems (Stiffness Effects of Sacrificial Wind-Restraint Systems Not Shown for Clarity)**

displacements, the effective period lengthens while loads on the superstructure remain constant. For such systems, the total displacement caused by repeated earthquake cycles is highly dependent on the characteristics of the ground motion and may exceed the design displacement, $D_D$. Since these systems do not have increasing resistance with increasing displacement, which helps to recenter the structure and prevent collapse, the procedures of the standard cannot be applied, and use of the system is prohibited.

Chapter 17 establishes isolator design displacements, shear forces for structural design, and other specific requirements for seismically isolated structures based on $MCE_R$ only. All other design requirements, including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal shear distribution, are the same as those for conventional, fixed-base structures. The main changes incorporated in this edition of the provisions include the following:

- Modified calculation procedure for the elastic design base shear forces from the design earthquake (DE) event to the $MCE_R$ event using a consistent set of upper and lower bound stiffness properties and displacements. This modification simplifies the design and analysis process by focusing only on the $MCE_R$ event.
- Relaxed permissible limits and criteria for the use of the equivalent lateral force (ELF) procedure. This modification minimizes the need to perform complex and computationally expensive nonlinear time history analyses to design the superstructure and isolation system on many base-isolated structures.
- Enhanced definitions of design properties of the isolation system.
- Use of nominal properties in the design process of typical isolation bearings specified by the manufacturers based on prior prototype testing.
- These nominal properties are adjusted using the newly incorporated AASHTO (1999) lambda factor concept to account for response uncertainties and obtain upper and lower bound properties of the isolation system for the design process.
- New method for the vertical distribution of lateral forces associated with the ELF method of design.
- Simplified approach for incorporating a 5% accidental mass eccentricity in nonlinear time history analyses.
- Reduction in the required number of peer reviewers on a seismic isolation project from the current three to five to a minimum of one peer reviewer. Also, peer reviewers are not required to attend the prototype tests.
- Calculation procedure to estimate permanent residual displacements that may occur in seismic isolation applications with relatively long period high yield/friction levels, and small yield displacements under a wide range of earthquake intensity.

## C17.2 GENERAL DESIGN REQUIREMENTS

In an ideal seismic isolation application, the lateral displacement of the structure is primarily accommodated through large lateral displacement or deformation of the isolation system rather than internal deformation of the superstructure above. Accordingly, the lateral force-resisting system of the superstructure above the isolation system is designed to have sufficient stiffness and strength to prevent large, inelastic displacements. Therefore, the standard contains criteria that limit the inelastic response of the superstructure. Although damage control is not an explicit objective of the standard, design to limit inelastic response of the structural system directly reduces the level of damage that would otherwise occur during an earthquake. In general, isolated structures designed in accordance with the standard are expected to

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

Isolated structures are expected to perform considerably better than fixed-based structures during moderate and major earthquakes. Table C17.2-1 compares the expected performance of isolated and fixed-based structures designed in accordance with the standard. Actual performance of an isolated structure should be determined by performing nonlinear time history analyses and computing interstory drifts and floor acceleration demands for an array of ground motions. Those results can be used to compute postearthquake repair costs of the structure using the FEMA P-58 performance-based earthquake engineering (PBEE) methodology (FEMA 2012) and/or large-scale simulations of direct and indirect costs using HAZUS software (FEMA 1999). Evaluation of seismic performance enhancement using seismic isolation should include its impact on floor accelerations, as well as interstory drifts, because these elements are key engineering demand parameters affecting damage in mechanical, electrical, and plumbing (MEP) equipment, ceilings and partitions, and building contents.

Loss of function or discontinued building operation is not included in Table C17.2-1. For certain fixed-based facilities, loss of function would not be expected unless there is significant structural and nonstructural damage that causes closure or restricted access to the building. In other cases, a facility with only limited or no structural damage would not be functional as a result of damage to vital nonstructural components or contents. Seismic isolation, designed in accordance with these provisions, would be expected to mitigate structural and nonstructural damage and to protect the facility against loss of function. The postearthquake repair time required to rehabilitate the structure can also be determined through a FEMA P-58 PBEE evaluation.

Observed structural or nonstructural damage in fixed-based buildings caused by moderate and large earthquakes around the world have typically been associated with high-intensity lateral ground motion excitation rather than vertical acceleration. Gravity design procedures for typical structures result in structural sections and dimensions with relatively high safety factors for

**Table C17.2-1 Performance Expected for Minor, Moderate, and Major Earthquakes**

| Performance Measure | Earthquake Ground Motion Level[a] Minor | Moderate | Major |
|---|---|---|---|
| Life safety: Loss of life or serious injury is not expected | F, I | F, I | F, I |
| Structural damage: Significant structural damage is not expected | F, I | F, I | I |
| Nonstructural damage: Significant nonstructural or content damage is not expected | F, I | I | I |

[a]F indicates fixed base; I indicates isolated.

seismic resistance. Therefore, current code provisions for fixed-based (or isolated) buildings only require use of a vertical earthquake component, $E_v$, obtained from static analysis procedures per Sections 12.2.4.6 and 12.2.7.1, defined as $0.2\,S_{DS}D$ under the design earthquake, where $D$ is the tributary dead load rather than explicit incorporation of vertical ground motions in the design analysis process. For seismic isolation, it should be noted that the term $0.2\,S_{DS}$ is replaced with $0.2\,S_{MS}$.

However, similar to fixed-based buildings, consideration of horizontal ground motion excitation alone may underestimate the acceleration response of floors and other building components. Portions of fixed-based and isolated structures may be especially sensitive to adverse structural response amplification induced by vertical ground motions including long spans, vertical discontinuities, or large cantilever elements. Certain nonstructural components, such as acoustic tile suspended ceiling systems, are also particularly vulnerable to the combination of vertical and horizontal ground motion effects. These building subassemblies or components may warrant additional vertical considerations. In addition, isolators with relatively low tributary gravity load and isolators located below columns that form part of the lateral force-resisting system can potentially have net uplift or tensile displacements caused by combined large vertical ground motion accelerations and global overturning. This uplift or bearing tension may induce high impact forces on the substructure, jeopardize the stability of the bearings, or result in bearing rupture.

Base-isolated structures located near certain fault characteristics that produce large vertical accelerations (e.g., hanging wall in reverse and reverse/oblique faults) are also more vulnerable and therefore may also require consideration of vertical ground motion excitation.

Vertical ground acceleration may affect the behavior of axial-load dependent isolation systems in the horizontal direction caused by potential coupling between horizontal and vertical response of the building structure.

Building response parameters that are expected to be affected by vertical excitation are vertical floor spectra and axial load demand on isolation bearings and columns, as discussed in Section C17.2.4.6. Isolated buildings with significant horizontal–vertical coupling are also expected to impart additional horizontal accelerations to the building at the frequencies of coupled modes that match the vertical motions.

If it is elected to investigate the effect of vertical ground motion acceleration on building response, one of the following analysis methods is suggested:

- Response spectrum analysis using horizontal and vertical spectrum (upward and downward).
- Response spectrum analysis using a vertical spectrum, combined with horizontal response spectrum analysis results using orthogonal combinations corresponding to the 100%–30%–30% rule.
- Three-dimensional response history analysis following the recommendations of Section C17.3.3 with explicit inclusion of vertical ground motion acceleration records.
- Horizontal response history analysis following the provisions of Section 17.3.3 considering the two limiting initial gravity load conditions defined per Section 17.2.7.1. Note that this analysis affects the effective characteristics of axial load-dependent isolators with resulting changes in base shear and displacement demands.

The structural model in these analyses should be capable of capturing the effects of vertical response and vertical mass participation, and should include the modeling recommendations in Section C17.6.2.

### C17.2.4 Isolation System

**C17.2.4.1 Environmental Conditions.** Environmental conditions that may adversely affect isolation system performance must be investigated thoroughly. Specific requirements for environmental considerations on isolators are included in the new Section 17.2.8. Unlike conventional materials whose properties do not vary substantially with time, the materials used in seismic isolators are typically subject to significant aging effects over the life span of a building structure. Because the testing protocol of Section 17.8 does not account for the effects of aging, contamination, scragging (temporary degradation of mechanical properties with repeated cycling), temperature, velocity effects, and wear, the designer must account for these effects by explicit analysis. The approach to accommodate these effects, introduced in the AASHTO specifications (AASHTO 1999), is to use property modification factors as specified in Section 17.2.8.4.

**C17.2.4.2 Wind Forces.** Lateral displacement over the depth of the isolation region resulting from wind loads must be limited to a value similar to that required for other stories of the superstructure.

**C17.2.4.3 Fire Resistance.** Where fire may adversely affect the lateral performance of the isolation system, the system must be protected to maintain the gravity-load resistance and stability required for the other elements of the superstructure supported by the isolation system.

**C17.2.4.4 Lateral Restoring Force.** The restoring force requirement is intended to limit residual displacements in the isolation system resulting from any earthquake event so that the isolated structure will adequately withstand aftershocks and future earthquakes. The potential for residual displacements is addressed in Section C17.2.6.

**C17.2.4.5 Displacement Restraint.** The use of a displacement restraint to limit displacements beyond the design displacement is discouraged. Where a displacement restraint system is used, explicit nonlinear response history analysis of the isolated structure for the $MCE_R$ level is required using the provisions of Chapter 16 to account for the effects of engaging the displacement restraint.

**C17.2.4.6 Vertical-Load Stability.** The vertical loads used to assess the stability of a given isolator should be calculated using bounding values of dead load, live load, and the peak earthquake demand at the $MCE_R$ level. Because earthquake loads are reversible in nature, peak earthquake load should be combined with bounding values of dead and live load in a manner that produces both the maximum downward force and the maximum upward force on any isolator. Stability of each isolator should be verified for these two extreme values of vertical load at peak $MCE_R$ displacement of the isolation system. In addition, all elements of the isolation system require testing or equivalent measures that demonstrate their stability for the $MCE_R$ ground motion levels. This stability can be demonstrated by performing a nonlinear static analysis for an $MCE_R$ response displacement of the entire structural system, including the isolation system, and showing that lateral and vertical stability are maintained. Alternatively, this stability can be demonstrated by performing a nonlinear dynamic analysis for the $MCE_R$ motions using the same inelastic reductions as for the design earthquake (DE) and acceptable capacities except that member and connection strengths can be taken as their nominal strengths with resistance factors, ϕ, taken as 1.0.

Vertical ground motion excitation affects bounding axial loads on isolation bearings and vertical stability design checks. The $E$ component of load combination 5 of Section 2.3.2 should consider the maximum of $E_v$ per code or the dynamic amplification from analysis when significant vertical acceleration is anticipated per Section C17.2.

**C17.2.4.7 Overturning.** The intent of this requirement is to prevent both global structural overturning and overstress of elements caused by localized uplift. Isolator uplift is acceptable as long as the isolation system does not disengage from its horizontal-resisting connection details. The connection details used in certain isolation systems do not develop tension resistance, a condition which should be accounted for in the analysis and design. Where the tension capacity of an isolator is used to resist uplift forces, design and testing in accordance with Sections 17.2.4.6 and 17.8.2.5 must be performed to demonstrate the adequacy of the system to resist tension forces at the total maximum displacement.

**C17.2.4.8 Inspection and Replacement.** Although most isolation systems do not require replacement following an earthquake event, access for inspection, repair, and replacement must be provided. In some cases (Section 17.2.6), recentering may be required. The isolation system should be inspected periodically as well as following significant earthquake events, and any damaged elements should be repaired or replaced.

**C17.2.4.9 Quality Control.** A testing and inspection program is necessary for both fabrication and installation of the isolator units. Because of the rapidly evolving technological advances of seismic isolation, reference to specific standards for testing and inspection is difficult for some systems, while reference for some systems is possible (e.g., elastomeric bearings should follow ASTM D4014 requirements (ASTM 2012). Similar standards are yet to be developed for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality should therefore be developed for each project. The requirements may vary depending on the type of isolation system used. Specific requirements for quality control testing are now given in Section 17.8.5.

### C17.2.5 Structural System

**C17.2.5.2 Minimum Building Separations.** A minimum separation between the isolated structure and other structures or rigid obstructions is required to allow unrestricted horizontal translation of the superstructure in all directions during an earthquake event. The separation dimension should be determined based on the total design displacement of the isolation system, the maximum lateral displacement of the superstructure above the isolation, and the lateral deformation of the adjacent structures.

**C17.2.5.4 Steel Ordinary Concentrically Braced Frames.** Section 17.5.4.2 of this standard implies that only seismic force-resisting systems permitted for fixed-based building applications are permitted to be used in seismic isolation applications. Table 12.2-1 limits the height of steel ordinary concentrically braced frames (OCBFs) in fixed-based multistory buildings assigned Seismic Design Categories D and E to 35 ft (10.7 m) and does not permit them in buildings assigned to Seismic Design Category F. Section 17.2.5.4 permits them to be used for seismic isolation applications to heights of 160 ft (48.8 m) in buildings assigned to Seismic Design Categories D, E, and F, provided that certain additional requirements are satisfied. The additional design requirements that must be satisfied include that the building must remain elastic at the design earthquake level (i.e., $R_I = 1.0$), that the moat clearance displacement, $D_{TM}$, be increased by 20%, and that the braced frame be designed to satisfy Section F1.7 of AISC 341. It should be noted that currently permitted OCBFs in seismically isolated buildings assigned to Seismic Design Categories D and E also need to satisfy Section F1.7 of AISC 341.

Seismic isolation has the benefit of absorbing most of the displacement of earthquake ground motions, allowing the seismic force-resisting system to remain essentially elastic. Restrictions in Chapter 17 on the seismic force-resisting system limit the inelastic reduction factor to a value of 2 or less to ensure essentially elastic behavior. A steel OCBF provides the benefit of providing a stiff superstructure with reduced drift demands on drift-sensitive nonstructural components while providing significant cost savings as compared to special systems. Steel OCBFs have been used in the United States for numerous (perhaps most) new seismically isolated essential facility buildings since the seismic isolation was first introduced in the 1980s. Some of these buildings have had heights as high as 130 ft (39.6 m). The 160-ft (48.8-m) height limit was permitted for seismic isolation with OCBFs in high seismic zones when seismic isolation was first introduced in the building code as an appendix to the UBC in 1991. When height limits were restricted for fixed-based OCBFs in the 2000 NEHRP Recommended Provisions, it was not recognized the effect the restriction could have on the design of seismically isolated buildings. The Section 17.2.5.4 change rectifies that oversight. It is the judgment of this committee that height limits should be increased to the 160-ft (48.8-m) level, provided that the additional conditions are met.

The AISC Seismic Committee (Task Committee-9) studied the concept of steel OCBFs in building applications to heights of 160 ft (48.8 m) in high seismic areas. They decided that additional detailing requirements are required, which are found in Section F1.7 of AISC 341.

There has been some concern that steel ordinary concentrically braced frames may have an unacceptable collapse hazard if ground motions greater than $MCE_R$ cause the isolation system to impact the surrounding moat wall. While there has not been a full FEMA P-695 (FEMA 2009) study of ordinary steel concentrically braced frame systems, a recent conservative study of one structure using OCBFs with $R_I = 1$ on isolation systems performed by Armin Masroor at SUNY Buffalo (Masroor and Mosqueda 2015) indicates that an acceptable risk of collapse (10% risk of collapse given MCE ground motions) is achieved if a 15–20% larger isolator displacement is provided. The study does not include the backup capacity of gravity connections or the influence of concrete-filled metal deck floor systems on the collapse capacity. Even though there is no requirement to consider ground motions beyond the risk-targeted maximum considered earthquake ground motion in design, it was the judgment of this committee to provide additional conservatism by requiring 20% in moat clearance. It is possible that further P-695 studies will demonstrate that the additional 1.2 factor of displacement capacity may not be needed.

**C17.2.5.5 Isolation System Connections.** This section addresses the connections of the structural elements that join isolators together. The isolators, joining elements, and connections comprise the isolation system. The joining elements are typically located immediately above the isolators; however, there are many ways to provide this framing, and this section is not meant to exclude other types of systems. It is important to note that the elements and the connections of the isolation system are designed for $V_b$ level forces, while elements immediately above the isolation system are designed for $V_s$ level forces.

Although ductility detailing for the connections in the isolation system is not required, and these elements are designed to remain elastic with $V_b$ level forces using $R = 1.0$, in some cases it may still be prudent to incorporate ductility detailing in these connections (where possible) to protect against unforeseen loading. This incorporation has been accomplished in the past by providing connection details similar to those used for a seismic force-resisting system of Table 12.2-1, with connection moment and shear strengths beyond the code minimum requirements. Ways of accomplishing this include factoring up the design forces for these connections, or providing connections with moment and shear strengths capable of developing the expected plastic moment strength of the beam, similar to AISC 341 or ACI 318 requirements for ordinary moment frames (OMFs).

**C17.2.6 Elements of Structures and Nonstructural Components.** To accommodate the differential horizontal and vertical movement between the isolated building and the ground, flexible utility connections are required. In addition, stiff elements crossing the isolation interface (such as stairs, elevator shafts, and walls) must be detailed to accommodate the total maximum displacement without compromising life safety provisions.

The effectiveness and performance of different isolation devices in building structures under a wide range of ground motion excitations have been assessed through numerous experimental and analytical studies (Kelly et al. 1980, Kelly and Hodder 1981, Kelly and Chaloub 1990; Zayas et al. 1987; Constantinou et al. 1999; Warn and Whittaker 2006; Buckle et al. 2002; Kelly and Konstantinidis 2011). The experimental programs included in these studies have typically consisted of reduced-scale test specimens, constructed with relatively high precision under laboratory conditions. These studies initially focused on elastomeric bearing devices, although in recent years the attention has shifted to the single- and multiconcave friction pendulum bearings. The latter system provides the option for longer isolated periods.

Recent full-scale shake table tests (Ryan et al. 2012) and analytical studies (Katsaras 2008) have shown that the isolation systems included in these studies with a combination of longer periods, relatively high yield/friction levels and small yield displacements will result in postearthquake residual displacements. In these studies, residual displacements ranging from 2 to 6 in. (50 to 150 mm) were measured and computed for isolated building structures with a period of 4 seconds or greater and a yield level in the range of 8 to 15% of the structure's weight. This permanent offset may affect the serviceability of the structure and possibly jeopardize the functionality of elements crossing the isolation plane (such as fire protection and weatherproofing elements, egress/entrance details, elevators, and joints of primary piping systems). Since it may not be possible to recenter some isolation systems, isolated structures with such characteristics should be detailed to accommodate these permanent offsets.

The Katsaras report (2008) provides recommendations for estimating the permanent residual displacement in any isolation system based on an extensive analytical and parametric study. The residual displacements measured in full-scale tests (Ryan et al. 2012) are reasonably predicted by this procedure, which uses an idealized bilinear isolation system, shown in Fig. C17.2-1. The three variables that affect the residual displacement are the isolated period (based on the second slope stiffness $K_D$), the yield/friction level ($F_0$), and the yield displacement ($D_y$).

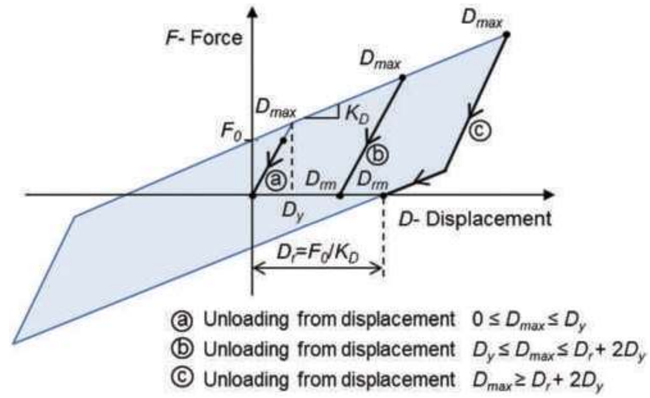


**FIGURE C17.2-1 Definitions of Static Residual Displacement $D_{rm}$ for a Bilinear Hysteretic System**

The procedure for estimating the permanent residual displacement, $D_{rd}$ (see Eq. C17.2-1) is a function of the system yield displacement $D_y$, the static residual displacement, $D_r = F_0/K_p$, and $D_{rm}$, which is a function of $D_m$, the maximum earthquake displacement shown in Table C17.2-2. For most applications, $D_{rm}$ is typically equal to $D_r$.

$$D_{rd} = \frac{0.87 D_{rm}}{\left(1 + 4.3\frac{D_{rm}}{D_r}\right)\left(1 + 31.7\frac{D_y}{D_r}\right)} \qquad \text{(C17.2-1)}$$

Thus, there is a simple two-step process to estimate the permanent residual displacement, $D_{rd}$:

- Calculate the static residual displacement, $D_r$, based on the isolated period (using the second slope stiffness, $K_D$) and the yield/friction levels. Table C17.2-3 provides values of $D_r$ for a range of periods from 2.5 to 20 seconds and a range of yield/friction levels from 0.03 $W$ to 0.15 $W$.
- Using the value of $D_r$ calculated for the isolation system and the yield displacement, $D_y$, of the system, the permanent residual displacement, $D_{rd}$, can be calculated from Eq. (C17.2-1), and Tables C17.2-4 and C17.2-5 provide the residual displacements for earthquake displacements ($D_m$) of 10 in. and 20 in. (250 mm to 500 mm), respectively.

The cells with bold type in Tables C17.2-4 and C17.2-5 correspond to permanent residual displacements exceeding 2.0 in. (50 mm). Note that for yield displacements of approximately 2.0 in. (50 mm), residual displacements will not occur for most isolation systems.

**Table C17.2-2 Values of Static Residual Displacement, $D_{rm}$**

| Range of Maximum Displacement, $D_{max}$ | Static Residual Displacement, $D_{rm}$ |
|---|---|
| $0 \le D_{max} \le D_y$ | 0 |
| $D_y \le D_{max} < D_r + 2D_y$ | $D_r(D_{max} - D_y)/(D_r + D_y)$ |
| $D_r + 2D_y \le D_{max}$ | $D_r$ |

**Table C17.2-3 Values of Static Residual Displacement, $D_r$ (in.), for Various Isolated Periods, $T$ (s), and Yield/Friction Levels, $F_0$**

| | $F_0$ | | | | |
|---|---|---|---|---|---|
| $T$ (s) | 0.03 | 0.06 | 0.09 | 0.12 | 0.15 |
| 2.5 | 1.8 | 3.6 | 5.3 | 7.1 | 8.9 |
| 2.8 | 2.4 | 4.7 | 7.1 | 9.5 | 11.9 |
| 3.5 | 3.6 | 7.1 | 10.7 | 14.2 | 17.8 |
| 4.0 | 4.7 | 9.5 | 14.2 | 19.0 | 23.7 |
| 5.0 | 7.2 | 14.5 | 21.7 | 28.9 | 36.1 |
| 5.6 | 9.2 | 18.5 | 27.7 | 37.0 | 46.2 |
| 6.0 | 10.7 | 21.3 | 32.0 | 42.7 | 53.3 |
| 7.0 | 14.2 | 28.4 | 42.7 | 56.9 | 71.1 |
| 8.0 | 18.7 | 37.4 | 56.2 | 74.9 | 93.6 |
| 9.0 | 23.7 | 47.4 | 71.1 | 94.8 | 118.5 |
| 20.1 | 118.5 | 237.0 | 355.5 | 474.0 | 592.5 |

*Note:* 1 in. = 25 mm.

**Table C17.2-4 Permanent Residual Displacement, $D_{rd}$, for a Maximum Earthquake Displacement, $D_m$, of 10 in. (250 mm)**

| | $D_y$ (in.) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $D_r$ (in.) | 0.005 | 0.01 | 0.02 | 0.20 | 0.39 | 0.59 | 0.98 | 1.97 |
| 4.0 | 0.63 | 0.60 | 0.56 | 0.25 | 0.16 | 0.11 | 0.07 | 0.04 |
| 7.9 | 1.28 | 1.25 | 1.21 | 0.73 | 0.50 | 0.39 | 0.26 | 0.14 |
| 11.9 | 1.86 | 1.84 | 1.79 | 1.22 | 0.90 | 0.71 | 0.50 | 0.27 |
| 15.8 | **2.32** | **2.30** | **2.25** | 1.67 | 1.29 | 1.04 | 0.75 | 0.43 |
| 19.8 | **2.72** | **2.70** | **2.66** | **2.07** | 1.65 | 1.37 | 1.01 | 0.59 |
| 23.7 | **3.08** | **3.06** | **3.02** | **2.43** | **1.99** | 1.68 | 1.27 | 0.76 |
| 27.7 | **3.39** | **3.37** | **3.34** | **2.75** | **2.30** | **1.97** | 1.51 | 0.92 |
| 31.6 | **3.68** | **3.66** | **3.62** | **3.05** | **2.59** | **2.24** | 1.75 | 1.09 |
| 35.6 | **3.93** | **3.91** | **3.87** | **3.32** | **2.85** | **2.49** | **1.97** | 1.25 |
| 39.5 | **4.16** | **4.14** | **4.11** | **3.56** | **3.09** | **2.73** | **2.19** | **1.41** |

*Note:* 1 in. = 25 mm.
Bold values designate $D_{rd}$ values of 2 inches or more.

**Table C17.2-5 Permanent Residual Displacements, $D_{rd}$, for a Maximum Earthquake Displacement, $D_m$, of 20 in. (500 mm)**

| | $D_y$ (in.) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $D_r$ (in.) | 0.005 | 0.01 | 0.02 | 0.20 | 0.39 | 0.59 | 0.98 | 1.97 |
| 4.0 | 0.63 | 0.60 | 0.56 | 0.25 | 0.16 | 0.11 | 0.07 | 0.04 |
| 7.9 | 1.28 | 1.25 | 1.21 | 0.73 | 0.50 | 0.39 | 0.26 | 0.15 |
| 11.9 | 1.93 | 1.90 | 1.85 | 1.28 | 0.95 | 0.76 | 0.54 | 0.31 |
| 15.8 | **2.58** | **2.55** | **2.50** | 1.86 | 1.45 | 1.19 | 0.87 | 0.52 |
| 19.8 | **3.23** | **3.20** | **3.15** | **2.47** | 1.98 | 1.65 | 1.24 | 0.75 |
| 23.7 | **3.75** | **3.72** | **3.67** | **2.97** | **2.45** | **2.08** | 1.59 | 0.99 |
| 27.7 | **4.22** | **4.20** | **4.15** | **3.45** | **2.90** | **2.50** | 1.95 | 1.24 |
| 31.6 | **4.67** | **4.64** | **4.60** | **3.90** | **3.33** | **2.90** | **2.30** | 1.50 |
| 35.6 | **5.08** | **5.06** | **5.02** | **4.32** | **3.74** | **3.30** | **2.65** | 1.76 |
| 39.5 | **5.47** | **5.45** | **5.41** | **4.72** | **4.13** | **3.67** | **2.99** | **2.02** |

*Note:* 1 in. = 25 mm.
Bold values designate $D_{rd}$ values of 2 inches or more.

**C17.2.8 Isolation System Properties.** This section defines and combines sources of variability in isolation system mechanical properties measured by prototype testing, permitted by manufacturing specification tolerances, and occurring over the life span of the structure because of aging and environmental effects. Upper bound and lower bound values of isolation system component behavior (e.g., for use in response history analysis procedures) and maximum and minimum values of isolation system effective stiffness and damping based on these bounding properties (e.g., for use in equivalent lateral force procedures) are established in this section. Values of property modification factors vary by product and cannot be specified generically in the provisions. Typical "default" values for the more commonly used systems are provided below. The designer and peer reviewer are responsible for determining appropriate values of these factors on a project-specific and product-specific basis.

This section also refines the concept of bounding (upper bound and lower bound) values of isolation system component behavior by

1. Explicitly including variability caused by manufacturing tolerances, aging, and environmental effects. ASCE/SEI 7-10 only addressed variability associated with prototype testing and
2. Simplifying design by basing bounding measures of amplitude-dependent behavior on only $MCE_R$ ground motions. ASCE/SEI 7-10 used both design earthquake (DE) and $MCE_R$ ground motions.

The new section also refines the concept of maximum and minimum effective stiffness and damping of the isolation system by use of revised formulas that

1. Define effective properties of the isolation system on bounding values of component behavior (i.e., same two refinements, described above) and
2. Eliminates the intentional conservatism of ASCE/SEI 7-10 that defines minimum effective damping in terms of maximum effective stiffness.

**C17.2.8.2 Isolator Unit Nominal Properties.** Isolator manufacturers typically supply nominal design properties that are reasonably accurate and can be confirmed by prototype tests in the design and construction phases. These nominal properties should be based on past prototype tests as defined in Section 17.8.2; see Fig. C17.2-2.

**C17.2.8.3 Bounding Properties of Isolation System Components.** The methodology for establishing lower and upper bound values for isolator basic mechanical properties based on property modification factors was first presented in Constantinou et al. (1999). It has since then been revised in Constantinou et al. (2007) based on the latest knowledge of

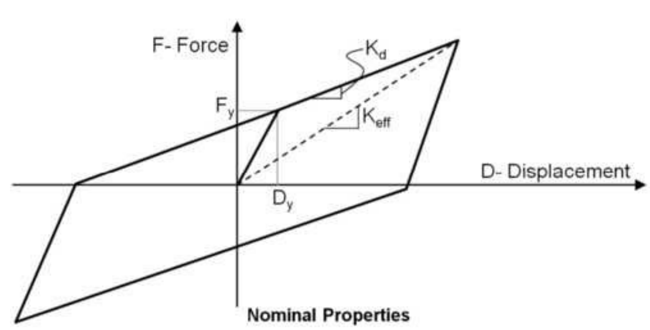


**FIGURE C17.2-2 Example of the Nominal Properties of a Bilinear Force Deflection System**

lifetime behavior of isolators. The methodology presented uses property modification factors to adjust isolator nominal properties based on considerations of natural variability in properties, effects of heating during cyclic motion, and the effects of aging, contamination, ambient temperature and duration of exposure to that temperature, and history of loading. The nominal mechanical properties should be based on prototype (or representative) testing on isolators not previously tested, at normal temperature and under dynamic loading.

The methodology also modifies the property modification factors to account for the unlikely situation of having several events of low probability of occurrence occur at the same time (i.e., maximum earthquake, aging, and low temperature) by use of property adjustment factors that are dependent on the significance of the structure analyzed (values range from 0.66 for a typical structure to 1.0 for a critical structure). This standard presumes that the property adjustment factor is 0.75. However, the registered design professional may opt to use the value of 1.0 based on the significance of the structure (e.g., health-care facilities or emergency operation centers) or based on the number of extreme events considered in the establishment of the property modification factor. For example, if only aging is considered, then a property adjustment factor of unity is appropriate.

Examples of application in the analysis and design of bridges may be found in Constantinou et al. (2011). These examples may serve as guidance in the application of the methodology in this standard. Constantinou et al. (2011) also presents procedures for estimating the nominal properties of lead-rubber and friction pendulum isolators, again based on the assumption that prototype test data are not available. Data used in the estimation of the range of properties were based on available test data, all of which were selected to heighten heating effects. Such data would be appropriate for cases of high-velocity motion and large lead core size or high friction values.

Recommended values for the specification tolerance on the average properties of all isolators of a given size isolator are typically in the ±10% to ±15% range. For a ±10% specification tolerance, the corresponding lambda factors would be $\lambda_{(\text{spec, max})} = 1.10$ and $\lambda_{(\text{spec, min})} = 0.90$. Variations in individual isolator properties are typically greater than the tolerance on the average properties of all isolators of a given size as presented in Section 17.2.8.4. It is recommended that the isolator manufacturer be consulted when establishing these tolerance values.

Section 17.2.8.4 requires the isolation system to be designed with consideration given to environmental conditions, including aging effects, creep, fatigue, and operating temperatures. The individual aging and environmental factors are multiplied together and then the portion of the lambda factor differing from unity is reduced by 0.75 based on the assumption that not all of the maximum values will occur simultaneously. As part of the design process, it is important to recognize that there will be additional variations in the nominal properties because of manufacturing. The next section specifies the property modification factors corresponding to the manufacturing process or default values if manufacturer-specific data are not available. These factors are combined with the property modification factors (Section 17.2.8.4) to determine the maximum and minimum properties of the isolators (Section 17.2.8.5) for use in the design and analysis process.

The lambda-test values $\lambda_{\text{test, max}}$ and $\lambda_{\text{test, min}}$ are determined from prototype testing and shall bound the variability and degradation in properties caused by speed of motion, heating effects, and scragging from Item 2 of Section 17.8.2.2. The registered design professional (RDP) shall specify whether this testing is performed quasi-statically, as in Item 2(a), or dynamically, as in Item 2(b). When testing is performed quasi-statically, the dynamic effects shall be accounted for in analysis and design using appropriate adjustment of the lambda-test values.

Item 3 of the testing requirements of Section 17.8.2.2 is important for property determination since it is common to Item 2. Using this testing, the lambda-test values $\lambda_{\text{test, max}}$ and $\lambda_{\text{test, min}}$ may be determined by three fully reversed cycles of dynamic (at the effective period $T_M$) loading at the maximum displacement $1.0D_M$ on full-scale specimens. This test regime incorporates the effects of high-speed motion. The upper and lower bound values of $K_d$ shall also envelop the $0.67D_M$ and $1.0D_M$ tests of Item 2 of Section 17.8.2.2. Therefore, the lambda-test values bound the effects of heating and scragging. As defined by Section 17.2.8.2, the nominal property of interest is defined as the average among the three cycles of loading. $\lambda_{\text{test, max}}$ shall be determined as the ratio of the first cycle property to the nominal property value. $\lambda_{\text{test, min}}$ shall be determined as the ratio of the property value at a representative cycle, determined by the RDP, to the nominal property value. The number of cycles shall be representative of the accepted performance of the isolation system for the local seismic hazard conditions, with the default cycle being the third cycle. A critique and guidance are provided in McVitty and Constantinou (2015).

**C17.2.8.4 Property Modification Factors.** The lambda factors are used to establish maximum and minimum mathematical models for analysis, the simplest form of which is the linear static procedure used to assess the minimum required design base shear and system displacements. More complex mathematical models account for various property variation effects explicitly (e.g., velocity, axial load, bilateral displacement, and instantaneous temperature). In this case, the cumulative effect of the lambda factors reduces (the combined lambda factor is closer to 1.0). However, some effects, such as specification tolerance and aging, are likely to always remain since they cannot be accounted for in mathematical models. Default lambda factors are provided in Table C17.2-6 as isolators from unknown manufacturers that do not have qualification test data. Default lambda factors are provided in Table C17.2-7 for most common types of isolators fabricated by quality manufacturers. Note that this table does not have any values of property modification factors for the actual stiffness ($K_d$) of sliding isolators. It is presumed that sliding isolators, whether flat or spherical, are produced with sufficiently high accuracy that their actual stiffness characteristics are known. The RDP may assign values of property modification factors different than unity for the actual stiffness of sliding bearings on the basis of data obtained in the prototype testing or on the basis of lack of experience with unknown manufacturers. Also note that this table provides values of property modification factors to approximately account for uncertainties in the materials and manufacturing methods used. These values presume lack of test data or incomplete test data and unknown manufacturers. For example, the values in Table C17.2-6 for sliding bearings presume unknown materials for the sliding interfaces so that there is considerable uncertainty in the friction coefficient values. Also, the data presume that elastomers used in elastomeric bearings have significant scragging and aging. Moreover, for lead-rubber bearings, the data in the table presume that there is considerable uncertainty in the starting value (before any hysteretic heating effects) of the effective yield strength of lead.

Accordingly, there is a considerable range in the upper and lower values of the property modification factors. Yet, these values should be used with caution since low-quality fabricators

**Table C17.2-6 Default Upper and Lower Bound Multipliers for Unknown Manufacturers**

| Variable | Unlubricated Interfaces, μ or $Q_d$ | Lubricated (Liquid) Interfaces, μ or $Q_d$ | Plain Low Damping Elastomeric, $K$ | Lead Rubber Bearing (LRB), $K_d$ | Lead Rubber Bearing (LRB), $Q_d$ | High-Damping Rubber (HDR), $K_d$ | High-Damping Rubber (HDR), $Q_d$ |
|---|---|---|---|---|---|---|---|
| **Example: Aging and Environmental Factors** | | | | | | | |
| Aging, $\lambda_a$ | 1.3 | 1.8 | 1.3 | 1.3 | 1 | 1.4 | 1.3 |
| Contamination, $\lambda_c$ | 1.2 | 1.4 | 1 | 1 | 1 | 1 | 1 |
| Example Upper Bound, $\lambda_{(ae,max)}$ | 1.56 | 2.52 | 1.3 | 1.3 | 1 | 1.4 | 1.3 |
| Example Lower Bound, $\lambda_{(ae,min)}$ | 1 | 1 | 1 | 1 | 1 | 1 | 1 |
| **Example: Testing Factors** | | | | | | | |
| All cyclic effects, Upper | 1.3 | 1.3 | 1.3 | 1.3 | 1.6 | 1.5 | 1.3 |
| All cyclic effects, Lower | 0.7 | 0.7 | 0.9 | 0.9 | 0.9 | 0.9 | 0.9 |
| Example Upper Bound, $\lambda_{(test,max)}$ | 1.3 | 1.3 | 1.3 | 1.3 | 1.6 | 1.5 | 1.3 |
| Example Lower Bound, $\lambda_{(test,min)}$ | 0.7 | 0.7 | 0.9 | 0.9 | 0.9 | 0.9 | 0.9 |
| $\lambda_{(PM,\,max)} = (1 + (0.75 * (\lambda_{(ae,\,max)} - 1))) * \lambda_{(test,\,max)}$ | 1.85 | 2.78 | 1.59 | 1.59 | 1.6 | 1.95 | 1.59 |
| $\lambda_{(PM,\,min)} = (1 - (0.75 * (1 - \lambda_{(ae,\,min)})) * \lambda_{(test,\,min)}$ | 0.7 | 0.7 | 0.9 | 0.9 | 0.9 | 0.9 | 0.9 |
| Lambda factor for Spec. Tolerance, $\lambda_{(spec,max)}$ | 1.15 | 1.15 | 1.15 | 1.15 | 1.15 | 1.15 | 1.15 |
| Lambda factor for Spec. Tolerance, $\lambda_{(spec,min)}$ | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 |
| Upper Bound Design Property Multiplier | 2.12 | 3.2 | 1.83 | 1.83 | 1.84 | 2.24 | 1.83 |
| Lower Bound Design Property Multiplier | 0.6 | 0.6 | 0.77 | 0.77 | 0.77 | 0.77 | 0.77 |
| Default Upper Bound Design Property Multiplier | 2.1 | 3.2 | 1.8 | 1.8 | 1.8 | 2.2 | 1.8 |
| Default Lower Bound Design Property Multiplier | 0.6 | 0.6 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |

*Note:* $\lambda_{PM}$ is the lambda value for testing and environmental effects.

**Table C17.2-7 Default Upper and Lower Bound Multipliers for Quality Manufacturers**

| Variable | Unlubricated PTFE, μ | Lubricated PTFE, μ | Rolling/ Sliding, $K2$ | Plain Elastomerics, $K$ | Lead rubber bearing (LRB), $K2$ | Lead rubber bearing (LRB), $Q_d$ | High-Damping Rubber (HDR), $Q_d$ | High-Damping Rubber (HDR),$K_d$ |
|---|---|---|---|---|---|---|---|---|
| **Example: Aging and Environmental Factors** | | | | | | | | |
| Aging, $\lambda_a$ | 1.10 | 1.50 | 1.00 | 1.10 | 1.10 | 1.00 | 1.20 | 1.20 |
| Contamination, $\lambda +$ | 1.10 | 1.10 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| Example Upper Bound, $\lambda_{(ae,max)}$ | 1.21 | 1.65 | 1.00 | 1.10 | 1.10 | 1.00 | 1.20 | 1.20 |
| Example Lower Bound, $\lambda_{(ae,min)}$ | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 | 1.00 |
| **Example: Testing Factors** | | | | | | | | |
| All cyclic effects, Upper | 1.20 | 1.30 | 1.00 | 1.03 | 1.03 | 1.30 | 1.50 | 1.30 |
| All cyclic effects, Lower | 0.95 | 0.95 | 1.00 | 0.98 | 0.98 | 0.95 | 0.95 | 0.95 |
| Example Upper Bound, $\lambda_{(test,max)}$ | 1.20 | 1.30 | 1.00 | 1.03 | 1.03 | 1.30 | 1.50 | 1.30 |
| Example Lower Bound, $\lambda_{(test,min)}$ | 0.95 | 0.95 | 1.00 | 0.98 | 0.98 | 0.95 | 0.95 | 0.95 |
| $\lambda_{(PM,\,max)} = (1 + (0.75 * (\lambda_{(ae,max)} - 1))) * \lambda_{(test,\,max)}$ | 1.39 | 1.93 | 1.00 | 1.11 | 1.11 | 1.30 | 1.73 | 1.50 |
| $\lambda_{(PM,\,min)} = (1 - (0.75 * (1 - \lambda_{(ae,\,min)})) * \lambda_{(test,\,min)}$ | 0.95 | 0.95 | 1.00 | 0.98 | 0.98 | 0.95 | 0.95 | 0.95 |
| Lambda factor for Spec. Tolerance, $\lambda_{(spec,max)}$ | 1.15 | 1.15 | 1.00 | 1.15 | 1.15 | 1.15 | 1.15 | 1.15 |
| Lambda factor for Spec. Tolerance, $\lambda_{(spec,min)}$ | 0.85 | 0.85 | 1.00 | 0.85 | 0.85 | 0.85 | 0.85 | 0.85 |
| Upper Bound Design Property Multiplier | 1.60 | 2.22 | 1.00 | 1.27 | 1.27 | 1.50 | 1.98 | 1.72 |
| Lower Bound Design Property Multiplier | 0.81 | 0.81 | 1.00 | 0.83 | 0.83 | 0.81 | 0.81 | 0.81 |
| Default Upper Bound Design Property Multiplier | 1.6 | 2.25 | 1 | 1.3 | 1.3 | 1.5 | 2 | 1.7 |
| Default Lower Bound Design Property Multiplier | 0.8 | 0.8 | 1 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |

*Note:* $\lambda_{PM}$ is the lambda value for testing and environmental effects.

could use materials and vulcanization and manufacturing processes that result in even greater property variations. The preferred approach for establishing property modification factors is through rigorous qualification testing of materials and manufacturing methods by a quality manufacturer, and dynamic prototype testing of full-size specimens, and by quality control testing at project-specific loads and displacements. These test data on similar-sized isolators take precedence over the default values.

For elastomeric isolators, lambda factors and prototype tests may need to address axial–shear interaction, bilateral deformation, load history including first cycle effects and the effects of scragging of virgin elastomeric isolators, ambient temperature, other environmental loads, and aging effects over the design life of the isolator.

For sliding isolators, lambda factors and prototype tests may need to address contact pressure, rate of loading or sliding velocity, bilateral deformation, ambient temperature, contamination, other environmental loads, and aging effects over the design life of the isolator.

Rate of loading or velocity effects are best accounted for by dynamic prototype testing of full-scale isolators. Property modification factors for accounting for these effects may be used in lieu of dynamic testing.

Generally, ambient temperature effects can be ignored for most isolation systems if they are in conditioned space where the expected temperature varies between 30°F and 100°F.

The following comments are provided in the approach to be followed for the determination of the bounding values of mechanical properties of isolators:

1. Heating effects (hysteretic or frictional) may be accounted for on the basis of a rational theory (e.g., Kalpakidis and Constantinou 2008, 2009; Kalpakidis et al. 2010) so that only the effects of uncertainty in the nominal values of the properties, aging, scragging, and contamination need to be considered. This is true for lead-rubber bearings where lead of high purity and of known thermomechanical properties is used. For sliding bearings, the composition of the sliding interface affects the relation of friction to temperature and therefore cannot be predicted by theory alone. Moreover, heating generated during high-speed motion may affect the bond strength of liners. Given that there are numerous sliding interfaces (and that they are typically proprietary), that heating effects in sliding bearings are directly dependent on pressure and velocity, and that size is important in the heating effects (Constantinou et al. 2007), full-scale dynamic prototype and production testing are very important for sliding bearings.
2. Heating effects are important for sliding bearings and the lead core in lead-rubber bearings. They are not important and need not be considered for elastomeric bearings of either low or high damping. The reason for this is described in Constantinou et al. (2007), where it has been shown, based on theory and experimental evidence, that the rise in temperature of elastomeric bearings during cyclic motion (about one degree centigrade per cycle) is too small to significantly affect their mechanical properties. Prototype and production testing of full-size specimens at the expected loads and displacements should be sufficient to detect poor material quality and poor material bonding in plain elastomeric bearings, even if done quasi-statically.
3. Scragging and recovery to the virgin rubber properties (see Constantinou et al. 2007 for details) are dependent on the rubber compound, size of the isolator, the vulcanization process, and the experience of the manufacturer. Also, it has been observed that scragging effects are more pronounced for rubber of low shear modulus and that the damping capacity of the rubber has a small effect. It has also been observed that some manufacturers are capable of producing low-modulus rubber without significant scragging effects, whereas others cannot. It is therefore recommended that the manufacturer should present data on the behavior of the rubber under virgin conditions (not previously tested and immediately after vulcanization) so that scragging property modification factors can be determined. This factor is defined as the ratio of the effective stiffness in the first cycle to the effectiveness stiffness in the third cycle, typically obtained at a representative rubber shear strain (e.g., 100%). It has been observed that this factor can be as high as, or can exceed, a value of 2.0 for shear-modulus rubber less than or equal to 0.45 MPa (65 psi). Also, it has been observed that some manufacturers can produce rubber with a shear modulus of 0.45 MPa (65 psi) and a scragging factor of approximately 1.2 or less. Accordingly, it is preferred to establish this factor by testing for each project or to use materials qualified in past projects.
4. Aging in elastomeric bearings has in general small effects (typically increases in stiffness and strength of the order of 10% to 30% over the lifetime of the structure), provided that scragging is also minor. It is believed that scragging is mostly the result of incomplete vulcanization, which is thus associated with aging as chemical processes in the rubber continue over time. Inexperienced manufacturers may produce low shear modulus elastomers by incomplete vulcanization, which should result in significant aging.
5. Aging in sliding bearings depends on the composition of the sliding interface. There are important concerns with bimetallic interfaces (Constantinou et al. 2007), even in the absence of corrosion, so that they should be penalized by large aging property modification factors or simply not used. Also, lubricated interfaces warrant higher aging and contamination property modification factors. The designer can refer to Constantinou et al. (2007) for detailed values of the factor depending on the conditions of operation and the environment of exposure. Note that lubrication is meant to be *liquid* lubrication typically applied either directly at the interface or within dimples. Solid lubrication in the form of graphite or similar materials that are integrated in the fabric of liners and used in contact with stainless steel for the sliding interface does not have the problems experienced by liquid lubrication.

**C17.2.8.5 Upper Bound and Lower Bound Force-Deflection Behavior of Isolation System Components.** An upper and lower bound representation of each type of isolation system component shall be developed using the lambda factors developed in Section 17.2.8.4. An example of a bilinear force deflection loop is shown in Fig. C17.2-2. In C17.2-3, the upper and lower bound lambda factors are applied to the nominal properties of the yield/friction level and the second or bilinear slope of the lateral force-displacement curve to determine the

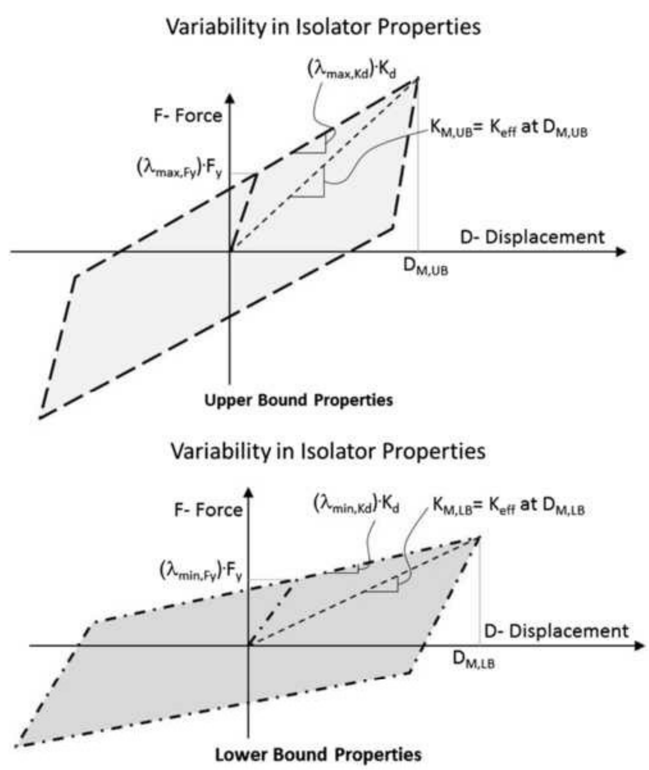


**FIGURE C17.2-3 Example of the Upper and Lower Bound Properties of a Bilinear Force Deflection System**

upper and lower bound representation of an isolation system component. The nomenclature shown in Fig. C17.2-3 is important to note. The effective stiffness and effective damping are calculated for both the upper and lower bound properties at the corresponding $D_M$. The maximum and minimum effective stiffness and effective damping are then developed from these upper and lower bound lateral force-displacement relationships in Section 17.2.8.6.

## C17.3 SEISMIC GROUND MOTION CRITERIA

**C17.3.1 Site-Specific Seismic Hazard.** This new section consolidates existing site-specific hazard requirements from other sections.

**C17.3.3 MCE$_R$ Ground Motion Records.** The MCE$_R$ spectrum is constructed from the $S_{MS}$, $S_{M1}$ parameters of Section 11.4.5, or 11.4.6, or 11.4.7.

When vertical excitation is included in isolated building response history analysis or response spectrum analysis, it is recommended that the vertical design spectra be computed by one of the following methods:

1. 2009 NEHRP Provisions (FEMA 2009) in new Chapter 23, equivalent to Annex A of Chapter 15, where the term $S_{DS}$ is replaced with $S_{MS}$. The vertical spectrum is computed based on near-fault or far-fault conditions through the parameter $S_s$ (short-period horizontal spectral acceleration for the site), as well as soil conditions (site classification).
2. Site-specific seismic hazard analysis using ground motion prediction equations for vertical shaking.
3. Multiplying the ordinates of the target spectrum corresponding to horizontal shaking by empirically based vertical-to-horizontal ratios that may be dependent on vertical period, site class, and proximity to fault.
4. Other approaches discussed in NIST GCR 11-917-15 (NIST 2011) consisting of a vertical conditional spectrum or conditional mean spectrum, envelope scaling, and mean spectral matching, or others.

Where response history analysis procedures are used, MCE$_R$ ground motions should consist of not less than seven pairs of appropriate horizontal acceleration components.

Where vertical excitation is included in isolated building response history analysis, scaling of the vertical ground motion component may follow one of the following recommended procedures:

- The vertical motions are spectrally matched to the design vertical spectrum using a vertical period range of $0.2T_v$ to $1.5T_v$, where $T_v$ is the building's primary vertical period of vibration. A wider period range may be considered because of uncertainty in the estimation of the primary vertical period of the building.
- The vertical component should be scaled by the same factor as the horizontal ground motion component(s). If the vertical component is included in the response of the structure, the response spectra of the vertical components of the records should be evaluated for reasonableness by comparing their spectra with a design vertical spectrum (NIST 2011).

If achieving a spectral fit to the vertical component spectrum is desirable, the vertical components of the selected records can be scaled by different factors than those used for horizontal components. Amplitude scaling of vertical components to a target vertical spectrum can be used using a least square error fit to a vertical period range of $0.2T_v$ to $1.5T_v$, where $T_v$ is the building's primary vertical period of vibration. A wider period range may be considered in this case because of uncertainty in the estimation of the primary vertical period of the building.

## C17.4 ANALYSIS PROCEDURE SELECTION

Three different analysis procedures are available for determining design-level seismic loads: the equivalent lateral force (ELF) procedure, the response spectrum procedure, and the response history procedure. For the ELF procedure, simple equations computing the lateral force demand at each level of the building structure (similar to those for conventional, fixed-base structures) are used to determine peak lateral displacement and design forces as a function of spectral acceleration and isolated-structure period and damping. The provisions of this section permit increased use of the ELF procedure, recognizing that the ELF procedure is adequate for isolated structures whose response is dominated by a single translational mode of vibration and whose superstructure is designed to remain essentially elastic (limited ductility demand and inelastic deformations) even for MCE$_R$ level ground motions. The ELF procedure is now permitted for the design of isolated structures at all sites (except Site Class F) as long as the superstructure is regular (as defined in new Section 17.2.2), has a fixed-base period ($T$) that is well separated from the isolated period ($T_{\min}$), and the isolation system meets certain "response predictability" criteria with which typical and commonly used isolation systems comply.

The design requirements for the structural system are based on the forces and drifts obtained from the MCE$_R$ earthquake using a consistent set of upper and lower bound isolation system properties, as discussed in Section C17.5. The isolation system—including all connections, supporting structural elements, and the "gap"—is required to be designed (and tested) for 100% of MCE$_R$ demand. Structural elements above the isolation system are now designed to remain essentially elastic for the MCE$_R$ earthquake. A similar fixed-base structure would be designed for design earthquake loads ($2/3$MCE$_R$) reduced by a factor of 6 to 8 rather than the MCE$_R$ demand reduced by a factor of up to 2 for a base-isolated structure.

## C17.5 EQUIVALENT LATERAL FORCE PROCEDURE

The lateral displacements given in this section approximate peak earthquake displacements of a single-degree-of-freedom, linear-elastic system of period, $T$, and effective damping, $\beta$. Eqs. (17.5-1) and (17.5-3) of ASCE 7-10 provided the peak displacement in the isolation system at the center of mass for both the DE and MCE$_R$ earthquakes, respectively. In these prior equations, as well as the current equation, the spectral acceleration terms at the isolated period are based on the premise that the longer period portion of the response spectra decayed as $1/T$. This is a conservative assumption and is the same as that required for design of a conventional, fixed-base structure of period $T_M$. A damping factor $B$, is used to decrease (or increase) the computed displacement demand where the effective damping coefficient of the isolation system is greater (or smaller) than 5% of critical damping. A comparison of values obtained from Eq. (17.5-1) and those obtained from nonlinear time history analyses are given in Kircher et al. (1988) and Constantinou et al. (1993).

The ELF formulas in this new edition compute minimum lateral displacements and forces required for isolation system design based only on MCE$_R$ level demands, rather than on a combination of design earthquake and $MCE_R$ levels, as in earlier editions of the provisions.

The calculations are performed separately for upper bound and lower bound isolation system properties, and the governing case shall be considered for design. Upper bound properties typically, but not always, result in a lower maximum displacement ($D_M$), higher damping ($\beta_M$), and higher lateral forces ($V_b$, $V_{st}$, $V_s$, and $k$).

Section 17.2.8 relates bounding values of effective period, stiffness, and damping of the isolation system to upper bound and lower bound lateral force-displacement behavior of the isolators.

## C17.5.3 Minimum Lateral Displacements Required for Design

**C17.5.3.1 Maximum Displacement.** The provisions of this section reflect the $MCE_R$-only basis for design and define maximum $MCE_R$ displacement in terms of $MCE_R$ response spectral acceleration, $S_{M1}$, at the appropriate $T$.

In addition, and of equal significance, the maximum displacement ($D_M$) and the damping modification factor ($B_M$) are determined separately for upper bound and lower bound isolation system properties. In earlier provisions, the maximum displacement ($D_M$) was defined only in terms of the damping associated with lower bound displacement, and this damping was combined with the upper bound stiffness to determine the design forces. This change is theoretically more correct, but it removes a significant conservatism in the ELF design of the superstructure. This reduction in superstructure design conservatism is offset by the change from design earthquake to $MCE_R$ ground motions as the basis for superstructure design forces.

**C17.5.3.2 Effective Period at the Maximum Displacement.** The provisions of this section are revised to reflect the $MCE_R$-only basis for design and associated changes in terminology (although maintaining the concept of effective period). The effective period $T_M$ is also determined separately for the upper and lower bound isolation properties.

**C17.5.3.3 Total Maximum Displacement.** The provisions of this section are revised to reflect the $MCE_R$-only basis for design and associated changes in terminology. Additionally, the formula for calculating total (translational and torsional) maximum $MCE_R$ displacement has been revised to include a term and corresponding equations that reward isolation systems configured to resist torsion.

The isolation system for a seismically isolated structure should be configured to minimize eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system, thus reducing the effects of torsion on the displacement of isolation elements. For conventional structures, allowance must be made for accidental eccentricity in both horizontal directions. Fig. C17.5-1 illustrates the terminology used in the standard. Eq. (17.5-3) provides a simplified formula for estimating the response caused by torsion in lieu of a more refined analysis. The additional component of displacement caused by torsion increases the design displacement at the corner of a structure by about 15% (for one perfectly square in plan) to about 30% (for one long and rectangular in plan) if the eccentricity is 5% of the maximum plan dimension. These calculated torsional displacements correspond to structures with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the structure, or certain sliding systems that minimize the effects of mass eccentricity, result in smaller torsional displacements.

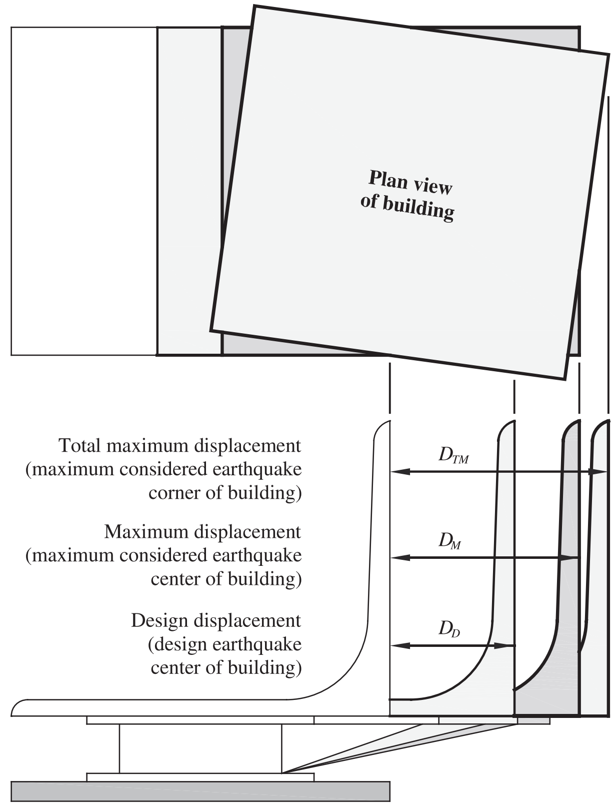


**FIGURE C17.5-1 Displacement Terminology**

The standard permits values of $D_{TM}$ as small as $1.15D_M$, with proper justification.

**C17.5.4 Minimum Lateral Forces Required for Design.** Fig. C17.5-2 illustrates the terminology for elements at, below, and above the isolation system. Eq. (17.5-5) specifies the peak elastic seismic shear for design of all structural elements at or below the isolation system (without reduction for ductile response). Eq. (17.5-7) specifies the peak elastic seismic shear for design of structural elements above the isolation system. For structures that have appreciable inelastic-deformation capability, this equation includes an effective reduction factor ($R_I = 3R/8$ not exceeding 2). This factor ensures essentially elastic behavior of the superstructure above the isolators.

These provisions include two significant philosophic changes in the method of calculating the elastic base shear for the structure. In ASCE 7-10 and earlier versions of the provisions, the elastic design base shear forces were determined from the design earthquake (DE) using a mixture of the upper bound effective stiffness and the maximum displacement obtained using the lower bound properties of the isolation system, as shown schematically in Fig. C17.5-3. This was known to be conservative. The elastic design base shear is now calculated from the $MCE_R$ event with a consistent set of upper and lower bound stiffness properties, as shown in Eq. (17.5-5) and Fig. C17.5-3.

A comparison of the old elastic design base shears for a range of isolation system design parameters and lambda factors using the ASCE 7-10 provisions and those using these new provisions is shown in Table C17.5-1. This comparison assumes that the DE is $2/3$ the $MCE_R$ and the longer period portion of both spectra decay as $S_1/T$. Table C17.5-1 shows a comparison between elastic design base shear calculated using the ASCE/SEI 7-10

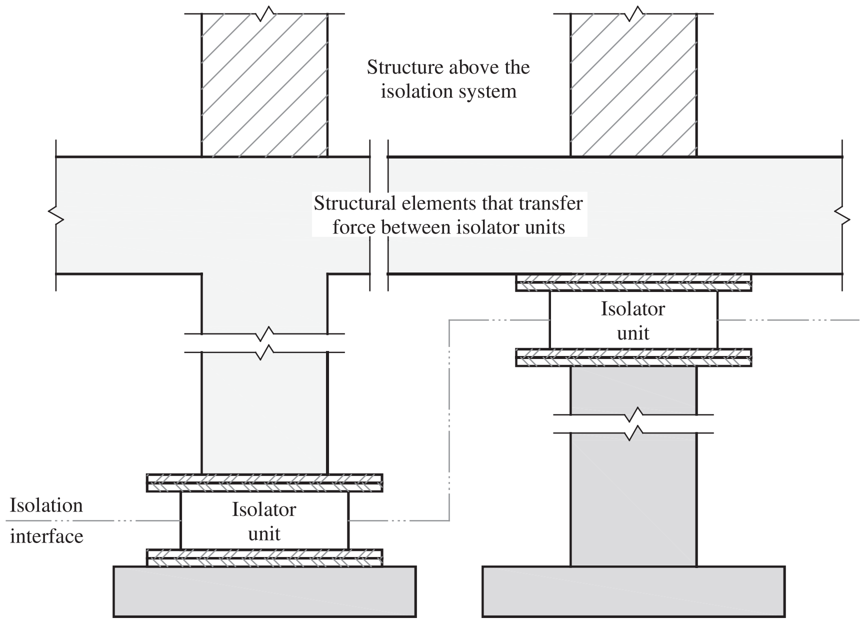


**FIGURE C17.5-2 Isolation System Terminology**

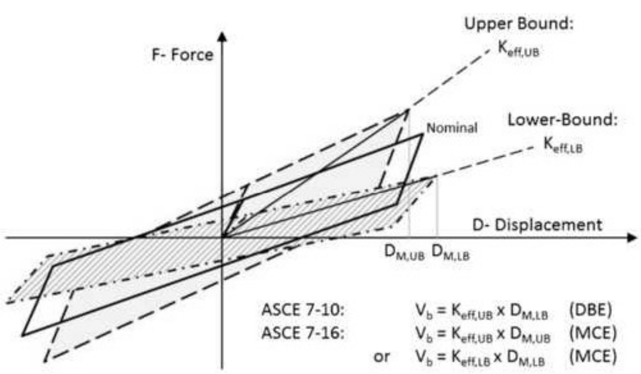


**FIGURE C17.5-3 Nominal, Upper Bound, and Lower Bound Bilinear Hysteretic Properties of Typical Isolator Bearing**

and 7-16 editions for a range of yield levels, second slopes, and bounding property multipliers.

The dark gray cells in Table C17.5-1 indicate that the new elastic design base shears are more than 10% higher than the old provisions; the light gray cells indicate that the new elastic base shears are 0 to 10% higher than the old provisions; and the white cells indicate that the new elastic base shears are less than the old provisions.

**C17.5.4.1 Isolation System and Structural Elements below the Base Level.** The provisions of this section are revised to reflect the $MCE_R$-only basis for design and associated changes in terminology. A new paragraph was added to this section to clarify that unreduced lateral loads should be used to determine overturning forces on the isolation system.

**C17.5.4.2 Structural Elements above the Base Level.** The provisions of this section are revised to reflect the $MCE_R$-only basis for design and associated changes in terminology, including the new concept of the "base level" as the first floor immediately above the isolation system.

An exception has been added to allow values of $R_I$ to exceed the current limit of 2.0, provided that the pushover strength of the superstructure at the $MCE_R$ drift or $0.015h_{sx}$ story drift exceeds (by 10%) the maximum $MCE_R$ force at the isolation interface ($V_b$). This exception directly addresses required strength and associated limits on inelastic displacement for $MCE_R$ demands. The pushover method is addressed in ASCE 41 (2007).

A new formula (Eq. (17.5-7)) now defines lateral force on elements above the base level in terms of reduced seismic weight (seismic weight excluding the base level), and the effective damping of the isolation system, based on recent work (York and Ryan 2008). In this formulation, it is assumed that the base level is located immediately (within 3.0 ft (0.9m) of top of isolator) above the isolation interface. When the base level is not located immediately above the isolation interface (e.g., there is no floor slab just above the isolators), the full (unreduced) seismic weight of the structure above the isolation interface is used in Eq. (17.5-7) to conservatively define lateral forces on elements above the base level.

**C17.5.4.3 Limits on $V_s$.** The provisions of this section are revised to reflect the $MCE_R$-only basis for design and associated changes in terminology.

In Section 17.5.4.3, the limits given on $V_s$ are revised to clarify that the force required to fully activate the isolation system should be based on either the upper bound force-deflection properties of the isolation system or 1.5 times nominal properties, whichever is greater. Other limits include (a) the yield/friction level to fully activate the isolation system and (b) the ultimate capacity of a sacrificial wind-restraint system that is intended to fail and release the superstructure during significant lateral load.

These limits are needed so that the superstructure does not yield prematurely before the isolation system has been activated and significantly displaced.

**C17.5.5 Vertical Distribution of Force.** The provisions of this section are revised to incorporate a more accurate distribution of shear over height considering the period of the superstructure and the effective damping of the isolation system. The specified

**Table C17.5-1 Comparison of Elastic Design Base Shears between ASCE 7-10 and 7-16**

| | Upper Bound Multipliers | | | $K_d$ | Yield Level | Lower Bound Multipliers | | | $K_d$ | Yield Level |
|---|---|---|---|---|---|---|---|---|---|---|
| **MCE$_R$ $S_1$ = 1.5** | | | | **1.15** | **1.6** | | | | **0.85** | **0.85** |
| $T2$ (s) | 2.00 | 2.00 | 3.00 | 3.00 | 4.00 | 4.00 | 5.00 | 5 | 6 | 6 |
| Yield Level | 0.05 | 0.10 | 0.05 | 0.10 | 0.05 | 0.10 | 0.05 | 0.1 | 0.05 | 0.1 |
| New, $V_b/W$ | 0.80 | 0.66 | 0.47 | 0.42 | 0.33 | 0.33 | 0.26 | 0.28 | 0.21 | 0.26 |
| ASCE 7-16/ASCE 7-10 | 1.14 | 1.02 | 1.08 | 0.91 | 1.02 | 0.84 | 0.96 | 0.83 | 0.91 | 0.82 |
| | | | | **1.0** | **1.6** | | | | **1.0** | **0.85** |
| New, $V_b/W$ | 0.77 | 0.71 | 0.52 | 0.42 | 0.35 | 0.31 | 0.26 | 0.27 | 0.21 | 0.25 |
| ASCE 7-16/ASCE 7-10 | 1.32 | 1.25 | 1.39 | 1.01 | 1.25 | 0.88 | 1.24 | 1.02 | 1.16 | 1.12 |
| **MCE$_R$ $S_1$ = 1.0** | | | | **1.15** | **1.6** | | | | **0.85** | **0.85** |
| $T2$ (s) | 2.00 | 2.00 | 3.00 | 3.00 | 4.00 | 4.00 | 5.00 | 5 | 6 | 6 |
| Yield Level | 0.05 | 0.10 | 0.05 | 0.10 | 0.05 | 0.10 | 0.05 | 0.1 | 0.05 | 0.1 |
| New, $V_b/W$ | 0.47 | 0.43 | 0.29 | 0.30 | 0.21 | 0.23 | 0.17 | 0.23 | 0.15 | 0.21 |
| ASCE 7-16/ASCE 7-10 | 1.08 | 0.91 | 0.99 | 0.83 | 0.91 | 0.65 | 0.84 | 0.76 | 0.84 | 0.71 |
| | | | | **1.35** | **1.5** | | | | **0.85** | **0.85** |
| New, $V_b/W$ | 0.54 | 0.47 | 0.33 | 0.32 | 0.24 | 0.29 | 0.19 | 0.22 | 0.16 | 0.20 |
| ASCE 7-16/ASCE 7-10 | 1.12 | 0.99 | 1.05 | 0.90 | 0.99 | 0.92 | 0.94 | 0.82 | 0.90 | 0.81 |
| | | | | **1.3** | **1.3** | | | | **0.85** | **0.85** |
| New, $V_b/W$ | 0.55 | 0.47 | 0.33 | 0.31 | 0.24 | 0.24 | 0.18 | 0.20 | 0.15 | 0.18 |
| ASCE 7-16/ASCE 7-10 | 1.22 | 1.10 | 1.16 | 1.01 | 1.10 | 0.94 | 1.05 | 0.91 | 1.01 | 0.89 |

*Note:* Dark gray cells indicate that the new elastic design base shears are more than 10% higher than the old provisions; light gray cells indicate 0–10% higher than old provisions.



method for vertical distribution of forces calculates the force at the base level immediately above the base isolation plane, then distributes the remainder of the base shear among the levels above. That is, the mass of the "base slab" above the isolators is not included in the vertical distribution of forces.

The proposed revision to the vertical force distribution is based on recent analytical studies (York and Ryan 2008 in collaboration with Structural Engineers Association of Northern California's Protective Systems Subcommittee PSSC). Linear theory of base isolation predicts that base shear is uniformly distributed over the height of the building, while the equivalent lateral force procedure of ASCE 7-10 prescribes a distribution of lateral forces that increase linearly with increasing height. The uniform distribution is consistent with the first mode shape of an isolated building, and the linear distribution is consistent with the first mode shape of a fixed-base building. However, a linear distribution may be overly conservative for an isolated building structure, especially for one- or two-story buildings with heavy base mass relative to the roof.

The principle established in the York and Ryan (2008) study was to develop two independent equations: one to predict the superstructure base shear $V_{st}$ relative to the base shear across the isolators $V_b$, and a second to distribute $V_{st}$ over the height of the building. Considering a reduction in $V_{st}$ relative to $V_b$ allowed for the often significant inertial forces at the base level, which can be amplified because of disproportionate mass at the base level, to be accounted for in design. The study also assumed that the superstructure base shear was distributed over the height using a '$k$' distribution (i.e., lateral force $\propto w_x h_x^k$ where $w_x$ is the weight and $h_x$ the height to level $x$), where $k = 0$ is a uniform distribution and $k = 1$ is a linear distribution. In the study, representative base-isolated multistory single-bay frame models were developed, and response history analysis was performed with a suite of 20 motions scaled to a target spectrum corresponding to the effective isolation system parameters. Regression analysis was performed to develop a best fit (relative to median results from response history analysis) of the superstructure to base shear ratio and $k$ factor as a function of system parameters. The equations recommended in York and Ryan (2008) provided the best "goodness of fit" among several considered, with $R^2$ values exceeding 0.95. Note that Eqs. (17.5-8) and (17.5-11) in the code change are the same as Eqs. (15) and (17) in York and Ryan (2008), with one modification: the coefficient for $k$ in Eq. (17.5-11) has been modified to reflect that the reference plane for determining height should be taken as the plane of isolation, which is below the isolated base slab.

It is difficult to confirm in advance whether the upper bound or lower bound isolation system response will govern the design of the isolation system and structure. It is possible, and even likely, that the distribution corresponding to upper bound isolation system properties will govern the design of one portion of the structure, and the lower bound distribution will govern another. For example, lower bound isolation system response may produce a higher displacement, $D_M$, a lower damping, $\beta_M$, but also a higher base shear, $V_b$. This difference could result in a vertical force distribution that governs for the lower stories of the building. The corresponding upper bound case, with lower displacement, $D_M$, but higher damping, $\beta_M$, might govern design of the upper part of the structure, even though the base shear, $V_b$, is lower.

The proposal to adopt the approach in York and Ryan (2008) is part of an overall revamp that will permit the equivalent static force method to be extended to a wider class of buildings. In York and Ryan (2008), the current method was shown to be quite conservative for systems with low to medium levels of damping combined with stiff superstructures but unconservative for highly damped systems or systems with relatively flexible superstructures.

The proposal has undergone a high level of scrutiny by the code committee. First, regression analysis was performed using the original York and Ryan (2008) response history data set to fit several alternative distributions suggested by code committee members that were intuitively more appealing. In all cases, the equations recommended in York and Ryan (2008) were shown to best fit the data. Second, a few code committee members appropriately attempted to validate the equations using independently generated response history analysis data sets. Much discussion ensued following the discovery that the equations were unconservative for a class of one- and two-story buildings with long isolation periods and high levels of effective damping in the isolation system. This was most noticeable for one- and two-story buildings, i.e., with relatively low $W_{st}/W$ ratios, predominantly single-mode fixed-base response, and where $T_{fb}$ aligned with the period based on the initial stiffness of the isolation system, $T_{k1}$. The York and Ryan (2008) data set was confirmed to contain similar cases to those generated independently, and the unconservatism was rationalized as a natural outcome of the regression approach. In an attempt to remove the unconservatism, equations were fit to the 84th percentile (median $+1\sigma$) vertical force distributions based on the original York and Ryan (2008) data set. However, the resulting distributions were unacceptably conservative and thus rejected.

The York and Ryan (2008) data set was subsequently expanded to broaden the range of fixed-base periods for low-rise structures and to provide additional confirmation of the independent data set. In addition, isolation system hysteresis loop shape was identified as the most significant factor in the degree of higher mode participation, resulting in increased $V_{st}/V_b$ ratio and $k$ factor. The provisions now identify this variable as needing a more conservative $k$ factor.

When computing the vertical force distribution using the equivalent linear force procedure, the provisions now divide isolation systems into two broad categories according to the shape of the hysteresis loop. Systems that have an abrupt transition between preyield and postyield response (or preslip and postslip for friction systems) are described as “strongly bilinear” and have been found to typically have higher superstructure accelerations and forces. Systems with a gradual or multistage transition between pre- and postyield response are described as “weakly bilinear” and were observed to have relatively lower superstructure accelerations and forces, at least for systems that fall within the historically adopted range of system strength/friction values (nominal isolation system force at zero displacement, $F_o = 0.03 \times W$ to $0.07 \times W$).

This limitation is acceptable because isolation systems with strength levels that fall significantly outside the upper end of this range are likely to have upper bound properties that do not meet the limitations of Section 17.4.1, unless the postyield stiffness or hazard level is high. Care should also be taken when using the equations to assess the performance of isolation systems at lower hazard levels because the equivalent damping can increase beyond the range of applicability of the original work.

Additional description of the two hysteresis loop types are provided in Table C17.5-2. An example of a theoretical loop for each system type is shown in Fig. C17.5-4.

Capturing this acceleration and force increase in the equivalent linear force procedure requires an increase in the $V_{st}/V_b$ ratio (Eq. (17.5-7) and the vertical force distribution $k$ factor (Eq. (17.5-11)). Consequently, the provisions require a different exponent to be used in Eq. (17.5-7) for a system that exhibits “strongly bilinear” behavior. Similar differences were observed in the $k$ factor (Eq. (17.5-11)), but these findings were judged to be insufficiently well developed to include in the provisions at this time, and the more conservative value for “strongly bilinear” systems was adopted for both system types.

The exception in Section 17.5.5 is a tool to address the issue identified in the one- and two-story buildings on a project-specific basis and to simplify the design of seismically isolated structures by eliminating the need to perform time-consuming and complex response history analysis of complete 3D building models each time the design is changed. At the beginning of the project, a response history analysis of a simplified building model (e.g., a stick model on isolators) is used to establish a custom inertia force distribution for the project. The analysis of the 3D building model can then be accomplished using simple static analysis techniques.

The limitations on use of the equivalent linear force procedure (Section 17.4.1) and on the response spectrum analysis procedure (Section 17.4.2.1) provide some additional limits. Item 7a in Section 17.4.1 requires a minimum restoring force, which effectively limits postyield stiffness to $K_d > F_o/D_M$ and also limits effective damping to 32% for a bilinear system.

Items 2 and 3 in Section 17.4.1 limit the effective period, $T_M \le 4.5$ s and effective damping, $\beta_M \le 30\%$ explicitly.

**C17.5.6 Drift Limits.** Drift limits are divided by $C_d/R$ for fixed-base structures since displacements calculated for lateral loads reduced by $R$ are multiplied by $C_d$ before checking drift. The $C_d$ term is used throughout the standard for fixed-base structures to approximate the ratio of actual earthquake response to response calculated for reduced forces. Generally, $C_d$ is $1/2$ to $4/5$ the value of $R$. For isolated structures, the $R_I$ factor is used both to reduce lateral loads and to increase displacements (calculated for reduced lateral loads) before checking drift. Equivalency would be obtained if the drift

**Table C17.5-2 Comparison of “Strongly Bilinear” and “Weakly Bilinear” Isolation Systems**

| System Type and Equation Term[a] | Pre- to Postyield Transition Characteristics | Cyclic Behavior Below Bilinear Yield/Slip Deformation | Example of Hysteresis Loop Shape | Example Systems[b] |
|---|---|---|---|---|
| “Strongly bilinear” ($1–3.5\beta_M$) | Abrupt transition from preyield or preslip to postyield or postslip | Essentially linear elastic, with little energy dissipation | Fig. C17.5-4a | • Flat sliding isolators with rigid backing<br>• Single-concave FPS<br>• Double-concave FPS with same friction coefficients top and bottom |
| “Weakly bilinear” ($1–2.5\beta_M$) | Smooth or multistage transition from preyield or preslip to postyield or postslip | Exhibits energy dissipation caused by yielding or initial low-level friction stage slip | Fig. C17.5-4b | • Elastomeric and viscous dampers<br>• Triple-concave FPS<br>• High-damping rubber<br>• Lead-rubber<br>• Elastomeric-backed sliders |

[a]Equation term refers to the exponent in Eq. (17.5-11).
[b]FPS is friction pendulum system.

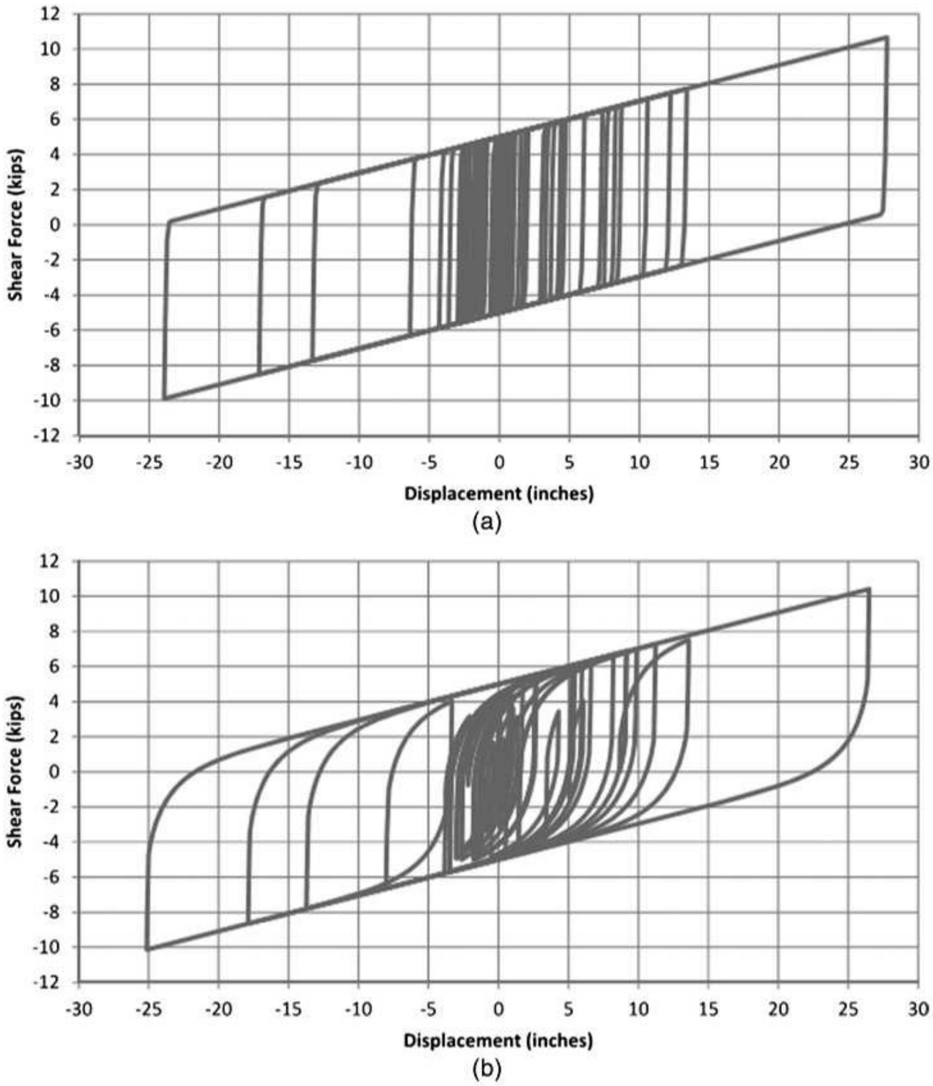


**FIGURE C17.5-4 Example Isolation System Example Loops**

limits for both fixed-base and isolated structures were based on their respective $R$ factors. It may be noted that the drift limits for isolated structures generally are more conservative than those for conventional, fixed-base structures, even where fixed-base structures are assigned to Risk Category IV. The maximum story drift permitted for design of isolated structures is constant for all risk categories.

## C17.6 DYNAMIC ANALYSIS PROCEDURES

This section specifies the requirements and limits for dynamic procedures.

A more detailed or refined study can be performed in accordance with the analysis procedures described in this section, compatible with the minimum requirements of Section 17.5. Reasons for performing a more refined study include

1. The importance of the building.
2. The need to analyze possible structure-isolation system interaction where the fixed-base period of the building is greater than one-third of the isolated period.
3. The need to explicitly model the deformational characteristics of the lateral force-resisting system where the structure above the isolation system is irregular.
4. The desirability of using site-specific ground motion data, especially for very soft or liquefiable soils (Site Class F) or for structures located where $S_1$ is greater than 0.60.
5. The desirability of explicitly modeling the deformational characteristics of the isolation system. This point is especially important for systems that have damping characteristics that are amplitude-dependent, rather than velocity-dependent, because it is difficult to determine an appropriate value of equivalent viscous damping for these systems.

Where response history analysis is used as the basis for design, the design displacement of the isolation system and design forces in elements of the structure above are computed from the average of seven pairs of ground motion, each selected and scaled in accordance with Section 17.3.2.

The provisions permit a 10% reduction of $V_b$ below the isolation system and 20% reduction of $V_b$ for the structure above the isolators if the structure is of regular configuration. The displacement reduction should not be greater than 20% if a dynamic analysis is performed.

In order to avoid the need to perform a large number of nonlinear response history analyses that include the suites of ground motions, the upper and lower bound isolator properties, and five or more locations of the center of mass, this provision allows the center-of-mass analysis results to be scaled and used to account for the effects of mass eccentricity in different building quadrants.

The following is a recommended method of developing appropriate amplification factors for deformations and forces for use with center-of-mass nonlinear response history analyses (NRHAs) which account for the effects of accidental torsion. The use of other rationally developed amplification factors is permitted.

The most critical directions for shifting the calculated center of mass are such that the accidental eccentricity adds to the inherent eccentricity in each orthogonal direction at each level. For each of these two eccentric mass positions, and with lower bound isolator properties, the suite of NRHA analyses should be run and the results processed in accordance with Section 17.6.3.4. The analysis cases are defined in Table C17.6-1.

The results from Cases IIa and IIb are then compared in turn to those from Case I. The following amplification factors (ratio of Case IIa or IIb response to Case I response) are computed:

1. The amplification of isolator displacement at the plan location with the largest isolator displacement;
2. The amplification of story drift in the structure at the plan location with the highest drift, enveloped over all stories; and
3. The amplification of frame-line shear forces at each story for the frame subjected to the maximum drift.

The larger of the two resulting scalars on isolator displacement should be used as the displacement amplification factor; the larger of the two resulting scalars on drift should be used as the deformation amplification factor; and the larger of the two resulting scalars on force should be used as the force amplification factor. Once the amplification factors are established, the effects of accidental eccentricity should be considered as follows.

The nonlinear response history analysis procedure should be carried out for the inherent mass eccentricity case only, considering both upper and lower bound isolator properties. For each isolator property variation, response quantities should be computed in accordance with Section 17.6.3.4. All resulting isolator displacements should be increased by the displacement amplification factor, all resulting deformation response quantities should be increased by the deformation amplification factor, and all resulting force quantities should be increased by the force amplification before being used for evaluation or design.

The procedure for scaling of dynamic analysis results to the ELF-based minima described in Section 17.6.4.3 is slightly different for response spectrum versus response history analysis. The reason for this difference is that it is necessary to create a consistent basis of comparison between the dynamic response quantities and the ELF-based minima (which are based on the maximum direction). When response spectrum analysis is performed, the isolator displacement, base shear, and story shear at any level used for comparison with the ELF-based minima already correspond to a single, maximum direction of excitation.

**Table C17.6-1 Analysis Cases for Establishing Amplification Factors**

| Case | Isolator Properties | Accidental Eccentricity |
|---|---|---|
| I | Lower bound | No |
| IIa | Lower bound | Yes, $X$ direction |
| IIb | Lower bound | Yes, $Y$ direction |

Thus, the vector sum of the 100%/30% directional combination rule (as described in Section 17.6.3.3) need not be used. Note, however, that while the 100%/30% directional combination rule is not required in scaling response spectrum analysis results to the ELF-based minima of Section 17.6.4.3, the 100%/30% directional combination rule is still required for design of the superstructure by response spectrum analysis, per Section 17.6.3.3. When nonlinear response history analysis is performed, the isolator displacement and base shear for each ground motion is calculated as the maximum of the vector sum of the two orthogonal components (of displacement or base shear) at each time step. The average of the maxima over all ground motions of these displacement and base shear vector-sum values is then used for comparison with the ELF-based minimum displacement and base shear per Section 17.6.4.3.

**C17.6.2 Modeling.** Capturing the vertical response of a building structure with a high degree of confidence may be a challenging task. Nonetheless, when the effects of vertical shaking are to be included in the analysis and/or design process of an isolated building structure, the following modeling recommendations are provided:

1. Vertical mass: All beams, columns, shear walls, and slabs should be included in the model, and the vertical mass should be distributed appropriately across the footprint of each floor.
2. Foundation properties: A range of soil properties and foundation damping should be considered in the analysis procedure since horizontal and vertical ground motion excitation can significantly affect building response.
3. Soil–foundation–structure interaction effects: Foundation damping, embedment, and base slab averaging may alter the vertical motions imparted on the structure as compared to the free-field motions.
4. Degrees of freedom: Additional degrees of freedom (e.g., nodes along the span of a beam or slab) will need to be added to the model to capture vertical effects.
5. Reduced time step: Since vertical ground motion excitation and building response often occur at higher frequencies than lateral excitation and response, a finer analysis time step might be required when vertical motions are included.

**C17.6.3.4 Response History Analysis Procedure.** For sites identified as near-fault, each pair of horizontal ground motion components shall be rotated to the fault-normal and fault-parallel directions of the causative faults and applied to the building in such orientation.

For all other sites, each pair of horizontal ground motion components shall be applied to the building at orthogonal orientations such that the mean of the component response spectra for the records applied in each direction is approximately equal ($\pm 10\%$) to the mean of the component response spectra of all records applied for the period range specified in Section 17.3.3. Peer review would be the judge of "approximately equal."

## C17.7 DESIGN REVIEW

The provisions allow for a single peer reviewer to evaluate the isolation system design. The reviewer should be a registered design professional (RDP), and if the engineer of record (EOR) is required to be a structural engineer (SE), the owner may consider ensuring that there is one SE on the peer review team. On more significant structures, it is likely that the design review panel may include two or three individuals, but for many isolated structures, a single, well-qualified peer reviewer is sufficient. If a manufacturer with unknown experience in the United States is selected as the supplier, the building owner may require the design reviewer to attend prototype tests.

The standard requires peer review to be performed by registered design professionals who are independent of the design team and other project contractors. The reviewer or review panel should include individuals with special expertise in one or more aspects of the design, analysis, and implementation of seismic isolation systems.

The peer reviewer or review panel should be identified before the development of design criteria (including site-specific ground-shaking criteria) and isolation system design options. Furthermore, the review panel should have full access to all pertinent information and the cooperation of the general design team and regulatory agencies involved in the project.

## C17.8 TESTING

The design displacements and forces determined using the standard assume that the deformational characteristics of the isolation system have been defined previously by comprehensive testing. If comprehensive test data are not available for a system, major design alterations in the structure may be necessary after the tests are complete. This change would result from variations in the isolation system properties assumed for design and those obtained by test. Therefore, it is advisable that prototype tests of systems be conducted during the early phases of design if sufficient prototype test data are not available from a given manufacturer.

The design displacements and forces determined using the standard are based on the assumption that the deformational characteristics of the isolation system have been defined previously by comprehensive qualification and prototype testing. Variations in isolator properties are addressed by the use of property variation factors that account for expected variation in isolator and isolation system properties from the assumed nominal values. In practice, past prototype test data are very likely to have been used to develop the estimated nominal values and associated lambda factors used in the design process, as described in Section 17.2.8.4.

When prototype testing is performed in accordance with Section 17.8.2, it serves to validate and check the assumed nominal properties and property variation factors used in the design. Where project-specific prototype testing is not performed, it is possible to perform a subset of the checks described below on the isolator unit and isolation system test properties using data from the quality control test program, described in Section 17.8.5.

**C17.8.2.2 Sequence and Cycles.** Section 17.2.8.4 describes the method by which minimum and maximum isolator properties for design and analysis are established using property variation or lambda (λ) factors to account for effects such as specification tolerance, cyclic degradation, and aging. The structural analysis is therefore performed twice, and the resulting demands are enveloped for design. For force-based design parameters and procedures, this requirement is relatively straightforward, as typically one case or the other governs, primarily, but not always, the upper bound. However, for components dependent on both force and deformation, e.g., the isolators, there exist two sets of axial load and displacement values for each required test. Lower bound properties typically result in larger displacements and smaller axial loads, whereas upper bound properties typically result in smaller displacements and larger axial loads. To avoid requiring that a complete set of duplicate tests be performed for the lower and upper bound conditions, Section 17.8.2.2 requires the results to be enveloped, combining the larger axial demands from one case with the larger displacements from the other. Strictly, these demands and displacement do not occur simultaneously, but the enveloping process is conservative.

The enveloping process typically results in test axial loads that correspond to the maximum properties and displacements that correspond to minimum properties. Hence, the test results determined using the enveloped demands may not directly relate to the design properties or analysis results determined for maximum and minimum properties separately. However, since the test demands envelop the performance range for the project, the registered design professional is able to use them to determine appropriate properties for both linear and nonlinear analysis using the same philosophy as provided here.

Two alternate testing protocols are included in Section 17.8.2.2. The traditional three-cycle tests are preserved in Item 2(a) for consistency with past provisions. These tests can be performed dynamically but have often been performed at slow speed consistent with the capability of manufacturers' testing equipment. The alternate test sequence provided in Item 2(b) is more suited to full-scale dynamic cyclic testing.

The Item (3) test displacement has been changed from $D_D$ to $D_M$, reflecting the focus of the provisions on only the $MCE_R$ event. Since this test is common to both test sequences 2(a) and 2 (b), it becomes important for property determination. This is the only test required to be repeated at different axial loads when isolators are also axial load-carrying elements, which is typically the case. This change was made to counter the criticism that the total test sequence of past provisions represented the equivalent energy input of many $MCE_R$ events back to back and that prototype test programs could not be completed in a reasonable time if any provision for isolator cooling and recovery was included.

The current test program is therefore more reflective of code-minimum required testing. The RDP and/or the isolator manufacturer may wish to perform additional testing to more accurately characterize the isolator for a wider range of axial loads and displacements than is provided here. For example, this might include performing the Item 2(b) dynamic test at additional axial loads once the code-required sequence is complete.

Heat effects for some systems may become significant, and misleading, if insufficient cooling time is included between adjacent tests. As a consequence, in test sequence 4 only five cycles of continuous dynamic testing are required as this is a limit of most test equipment. The first-cycle or scragging effects observed in some isolators may recover with time, so back-to-back testing may result in an underestimation of these effects. Refer to Constantinou et al. (2007) and Kalpakidis and Constantinou (2008) for additional information. The impact of this behavior may be mitigated by basing cyclic lambda factors on tests performed relatively early in the sequence before these effects become significant.

**C17.8.2.3 Dynamic Testing.** Section 17.8.2.3 clarifies when dynamic testing is required. Many common isolator types exhibit velocity dependence, however, this testing can be expensive and can only be performed by a limited number of test facilities. The intent is not that dynamic testing of isolators be performed for every project. Sufficient dynamic test data must be available to characterize the cyclic performance of the isolator, in particular the change in isolator properties during the test, i.e., with respect to the test average value. Dynamic testing must therefore be used to establish the $\lambda_{(\text{test, min})}$ and $\lambda_{(\text{test, max})}$ values used in Section 17.2.8.4, since these values are typically underestimated from slow-speed test data. If project prototype or production testing is to be performed at slow speeds, this testing would also be used to establish factors that account for the effect of velocity and heating on the test average values of $k_{\text{eff}}$, $k_d$, and $E_{\text{loop}}$. These factors can either be thought of as a separate set of velocity-correction factors to be applied on test average values, or they can be incorporated into the $\lambda_{(\text{test, min})}$ and $\lambda_{(\text{test, max})}$ values themselves.

It may also be possible to modify the isolator mathematical model, for example, to capture some or all of the isolator velocity dependence, e.g., the change in yield level of the lead core in a lead rubber bearing (LRB).

If project-specific prototype testing is undertaken, it may be necessary to adjust the test sequence in recognition of the capacity limitations of the test equipment, and this notion is now explicitly recognized in Section 17.8.2.2. For example, tests that simultaneously combine maximum velocity and maximum displacement may exceed the capacity of the test equipment and may also not be reflective of earthquake shaking characteristics. A more detailed examination of analysis results may be required to determine the maximum expected velocity corresponding to the various test deformation levels and to establish appropriate values for tests.

Refer to Constantinou et al. (2007) for additional information.

**C17.8.2.4 Units Dependent on Bilateral Load.** All types of isolators have bilateral load dependence to some degree. The mathematical models used in the structural analysis may include some or all of the bilateral load characteristics for the particular isolator type under consideration. If not, it may be necessary to examine prototype test data to establish the impact on the isolator force-deformation response as a result of the expected bilateral loading demands. A bounding approach using lambda ($\lambda$) factors is one method of addressing bilateral load effects that cannot be readily incorporated in the isolator mathematical model.

Bilateral isolator testing is complex, and only a few test facilities are capable of performing these tests. Project-specific bilateral load testing has not typically been performed for isolation projects completed to date. In lieu of performing project-specific testing, less restrictive similarity requirements may be considered by the registered design professional compared to those required for test data submitted to satisfy similarity for Sections 17.8.2.2 and 17.8.2.5. Refer to Constantinou et al. (2007) for additional information.

**C17.8.2.5 Maximum and Minimum Vertical Load.** The exception to Section 17.8.2.5 permits that the tests may be performed twice, once with demands resulting from upper bound properties and once with lower bound properties. This option may be preferable for these isolator tests performed at $D_{TM}$ since the isolator will be closer to its ultimate capacity.

**C17.8.2.7 Testing Similar Units.** Section 17.8.2.7 now provides specific limits related to the acceptability of data from testing of similar isolators. A wider range of acceptability is permitted for dynamic test data.

1. The submitted test data should demonstrate the manufacturers' ability to successfully produce isolators that are comparable in size to the project prototypes, for the relevant dimensional parameters, and to test them under force and displacement demands equal to or comparable to those required for the project.
2. It is preferred that the submitted test data necessary to satisfy the registered design professional and design review be for as few different isolator types and test programs as possible. Nonetheless, it may be necessary to consider data for isolator A to satisfy one aspect of the required project prototype test program, and data from isolator B for another.
3. For more complex types of testing, it may be necessary to accept a wider variation of isolator dimension or test demands than for tests that more fundamentally establish the isolator nominal operating characteristics, e.g., the testing required to characterize the isolator for loading rate dependence (Section 17.8.2.3) and bilateral load dependence (Section 17.8.2.4).
4. The registered design professional is not expected to examine quality control procedures in detail to determine whether the proposed isolators were manufactured using sufficiently similar methods and materials. Rather, it is the responsibility of the manufacturer to document the specific differences, if any, preferably via traceable quality control documentation and to substantiate that any variations are not significant.
5. In some cases, the manufacturer may not wish to divulge proprietary information regarding methods of isolator fabrication, materials, or quality control procedures. These concerns may or may not be alleviated by confidentiality agreements or other means to limit the distribution and publication of sensitive material. Regardless, the final acceptability of the test information of similar units is at the sole discretion of the registered design professional and the design review, and not the manufacturer.
6. Similarity can be especially problematic in a competitive bid situation, when successful selection may hinge on the success of one supplier in eliminating the need to fabricate and test project-specific prototype isolators. This requirement can be addressed by determining acceptability of similarity data before bid or by including more detailed similarity acceptance provisions in the bid documentation than have been provided herein.

Refer to Constantinou et al. (2007) and Shenton (1996) for additional information.

**C17.8.3 Determination of Force-Deflection Characteristics.** The method of determining the isolator effective stiffness and effective damping ratio is specified in Eqs. (17.8-1) and (17.8-2). Explicit direction is provided for establishment of effective stiffness and effective damping ratio for each cycle of test. A procedure is also provided for fitting a bilinear loop to a given test cycle, or to an average test loop to determine the postyield stiffness, $k_d$. This process can be performed several different ways; however, the fitted bilinear loop should also match effective stiffness and energy dissipated per cycle from the test. Once $k_d$ is established, the other properties of the bilinear loop (e.g., $f_y$, $f_o$) all follow from the bilinear model.

Depending on the isolator type and the degree of sophistication of the isolator hysteresis loop adopted in the analysis, additional parameters may also be calculated, such as different friction coefficients, tangent stiffness values, or trilinear loop properties.

These parameters are used to develop a mathematical model of the isolator test hysteresis that replicates, as near as possible, the observed test response for a given test cycle. The model should result in a very close match to the effective stiffness and effective damping ratio and should result in a good visual fit to the hysteresis loop with respect to the additional parameters. The mathematic loop model must, at a minimum, match the effective stiffness and loop area from the test within the degree of variation adopted within the $\lambda_{(\text{spec, min})}$ to $\lambda_{(\text{spec, max})}$ range.

Data from the first cycle (or half cycle) of testing is not usually representative of full-cycle behavior and is typically discarded by manufacturers during data processing. An additional cycle (or half cycle) is added at the end to provide the required number of test cycles from which data can be extracted. However, the first cycle of a test is often important when establishing upper bound isolator properties and should be included when determining the $\lambda_{(\text{test, min})}$ and $\lambda_{(\text{test, max})}$ factors. The form of the test loop, however, is different to that of a full-scale loop, particularly for multistage isolator systems such as the double- or triple-concave friction pendulum system. This form may require different hysteresis parameters to be considered than the ones described by the bilinear model in Fig. 17.8-1. The provisions permit the use of different methods for fitting the loop, such as a straight-line fit of $k_d$ directly to the hysteresis curve extending to $D_M$ and then determining $k_1$ to match $E_{\text{loop}}$, or an alternate is defining $D_y$ and $F_y$ by visual fit and then determining $k_d$ to match $E_{\text{loop}}$.

The effective stiffness and effective damping ratio are required in linear static and linear response spectrum analysis. However, even if a nonlinear response history analysis is performed, these parameters are still required to check the required minimum lateral displacements and lateral forces of Sections 17.5.3 and 17.5.4, respectively.

**C17.8.4 Test Specimen Adequacy.** For each isolator type, the effective stiffness and effective damping ratio for a given test axial load, test displacement, and cycle of test are determined in accordance with Section 17.8.3. For the dynamic test sequence in Item 2(a) in Section 17.8.2.2, there are two cycles at each increment of test displacement; for the traditional slow-speed sequence, there are three.

However, as part of a seismic isolation system, the axial load on a given isolator varies during a single complete cycle of loading. The required range of variation is assumed to be defined by the test load combinations required in Section 17.2.4.6, and the appropriate properties for analysis are assumed to be the average of the properties at the three axial loads. The test performed for Item (3) in Section 17.8.2.2 is critical to this evaluation since it is the three-cycle test performed at all three axial loads common to both the dynamic and slow-speed sequence.

In addition, since all isolators must undergo the same total horizontal cyclic loading as part of the same system, it is therefore assumed to be appropriate to assemble the total seismic isolation system properties using the following sequence:

1. Average the test results for a given isolator and cycle of loading across the three test axial loads. Also compute corresponding test lambda factors for each isolator type.
2. Sum up the total isolation system properties for each cycle of loading according to the number of isolators of each type.
3. Determine the maximum and minimum values of total system effective stiffness over the required three cycles of testing and the corresponding values of the effective damping ratio. Also compute the test lambda factors for the overall isolation system.

Two sets of test lambda factors emerge from this process, those applicable to individual isolators determined in (1) and those applicable to the overall isolation system properties determined in (3). In general, the test lambda factors for individual isolator tests are similar to those for each isolator type, which are similar to that for the overall isolation system. If this is the case, it may be more convenient to simplify the lambda factors assumed during design to reflect reasonable envelope values to be applied to all isolator types.

However, if the test lambda factors that emerge from project-specific prototype testing differ significantly from those assumed during design, it may be helpful to build up the system properties as described above, since the unexpectedly high test lambda factors for one isolator type may be offset by test lambda factors for another isolator type that were lower than the assumed values. In this circumstance, the prototype test results may be considered acceptable, provided that the torsional behavior of the system is not significantly affected and that the isolator connection and adjacent members can accommodate any resulting increase in local force demands.

Also, note that a subset of the isolation system properties can be determined from quality assessment and quality control (production) testing. This testing is typically performed at an axial load corresponding to the average $D + 0.5L$ axial load for the isolator type and to a displacement equal to $2/3(D_M)$. Keep in mind that isolator properties with target nominal three-cycle values estimated to match the average test value across three axial loads may not exactly match the values from production testing at the average dead load.

This result is most commonly observed with effective stiffness and effective damping ratio values for friction-based isolators since the average of the three test axial loads required in Section 17.8.2.2 does not exactly match that present in the isolator during the lateral analysis (the seismic weight, typically $1.0 \times$ Dead Load). In this case, some additional adjustment of properties may be required. Once the test effective stiffness and effective damping ratio of the isolation system have been established, these are compared to the values assumed for design in Section 17.2.8.4, defined by the nominal values and the values of $\lambda_{(\text{test, max})}$ and $\lambda_{(\text{test, min})}$.

In practice, instead of performing prototype tests for direct use in analysis, it may be simpler to use prototype test data or data from acceptable past testing of similar units (see Section 17.8.2.7) to establish isolator property dependence relationships for such things as axial load or velocity. If relationships are established for applicable hysteresis-loop parameters, such as yield force, friction ratio, initial stiffness, and postyield stiffness, these can be used to generate the required isolator unit and isolation system effective stiffness and effective damping ratios for the project over the required operating range.

**C17.8.5 Production Tests.** The number of production isolation units to be tested in combined compression and shear is 100%. Both quasi-static and dynamic tests are acceptable for all types of isolators. If a quasi-static test is used, it must have been performed as a part of the prototype tests. The registered design professional (RDP) is responsible for defining in the project specifications the scope of the manufacturing quality control test program. The RDP decides on the acceptable range of variations in the measured properties of the production isolation units. All (100%) of the isolators of a given type and size are tested in combined compression and shear, and the allowable variation of the mean should be within the specified tolerance of Section 17.2.8.4 (typically $\pm 10\%$ or $\pm 15\%$). Individual isolators may be permitted a wider variation ($\pm 15\%$ or $\pm 20\%$) from the nominal design properties. For example, the mean of the characteristic strength, $Q$, for all tested isolators might be permitted to vary no more than $\pm 10\%$ from the specified value of $Q$, but the characteristic strength for any individual isolation unit might be permitted to vary no more than $\pm 15\%$ from the specified value of $Q$.

Another commonly specified allowable range of deviation from specified properties is ±15% for the mean value of all tested isolation units, and ±20% for any single isolation unit.

The combined compression and shear testing of the isolators reveals the most relevant characteristics of the completed isolation unit and permits the RDP to verify that the production isolation units provide load-deflection behavior that is consistent with the structural design assumptions. Although vertical load-deflection tests have sometimes been specified in quality control testing programs, these test data are typically of little value. Consideration should be given to the overall cost and schedule effects of performing multiple types of quality control tests, and only those tests that are directly relevant to verifying the design properties of the isolation units should be specified.

Where project-specific prototype testing in accordance with Section 17.8.2 is not performed, the production test program should evaluate the performance of each isolator unit type for the property variation effects from Section 17.2.8.4.

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