Chapter 13 Commentary

SEISMICALLY ISOLATED STRUCTURE DESIGN REQUIREMENTS

13.1 GENERAL

Seismic isolation, commonly referred to as base isolation, is a design concept based on the premise that a structure can be substantially decoupled from potentially damaging earthquake motions. By substantially decoupling the structure from the ground motion, the level of response in the structure can be reduced significantly from the level that would otherwise occur in a conventional, fixed-base building.


In the mid-1980s, the initial applications identified a need to supplement existing codes with design requirements developed specifically for seismically isolated buildings. Code development work occurred throughout the late 1980s. The status of U.S. seismic isolation design requirements as of May 2003 is as follows:

1. In late 1989, the Structural Engineers Association of California (SEAOC) State Seismology Committee adopted an “Appendix to Chapter 2” of the SEAOC Blue Book entitled, “General Requirements for the Design and Construction of Seismic-Isolated Structures.” These requirements were submitted to the International Conference of Building Officials (ICBO) and were adopted by ICBO as an appendix of the 1991 Uniform Building Code (UBC). The most current version of these regulations may be found in the ASCE-7-02 (ASCE, 2003) and the 2003 International Building Code (ICC, 2003).

2. In 1991 the Federal Emergency Management Agency (FEMA) initiated a 6-year program to develop a set of nationally applicable guidelines for seismic rehabilitation of existing buildings. These guidelines (known as the NEHRP Guidelines for the Seismic Rehabilitation of Buildings) were published as FEMA 273. In 2000, FEMA 273 was republished, with minor amendments, as FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings. The design and analysis methods of the NEHRP Guidelines and the FEMA Prestandard parallel closely methods required by the NEHRP Recommended Provisions for new buildings, except that more liberal design is permitted for the superstructure of a rehabilitated building.

A general concern has long existed regarding the applicability of different types of isolation systems. Rather than addressing a specific method of base isolation, the Provisions provides general design requirements applicable to a wide range of possible seismic isolation systems.

Although remaining general, the design requirements rely on mandatory testing of isolation-system hardware to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Some systems may not be capable of demonstrating acceptability by test and, consequently, would not be permitted. In general, acceptable systems will: (1) remain stable for
required design displacements, (2) provide increasing resistance with increasing displacement, (3) not degrade under repeated cyclic load, and (4) have quantifiable engineering parameters (such as force-deflection characteristics and damping).

Conceptually, there are four basic types of isolation system force-deflection relationships. These idealized relationships are shown in Figure C13.1-1 with each idealized curve having the same design displacement, $D_D$, for the design earthquake. A linear isolation system is represented by Curve A and has the same isolated period for all earthquake load levels. In addition, the force generated in the superstructure is directly proportional to the displacement across the isolation system.

A hardening isolation system is represented by Curve B. This system is soft initially (long effective period) and then stiffens (effective period shortens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a hardening system, the superstructure is subjected to higher forces and the isolation system to lower displacements than a comparable linear system.

A softening isolation system is represented by Curve C. This system is stiff initially (short effective period) and softens (effective period lengthens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a softening system, the superstructure is subjected to lower forces and the isolation system to higher displacements than a comparable linear system.

A sliding isolation system is represented by Curve D. This system is governed by the friction force of the isolation system. Like the softening system, the effective period lengthens as the earthquake load level increases and loads on the superstructure remain constant.

The total system displacement for extreme displacement of the sliding isolation system, after repeated earthquake cycles, is highly dependent on the vibratory characteristics of the ground motion and may exceed the design displacement, $D_D$. Consequently, minimum design requirements do not adequately define peak seismic displacement for seismic isolation systems governed solely by friction forces.

**13.1.1 Scope.** The requirements of Chapter 13 provide isolator design displacements, shear forces for structural design, and other specific requirements for seismically isolated structures. All other design requirements including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal shear distribution are covered by the applicable sections of the *Provisions* for conventional, fixed-base structures.
13.2 GENERAL DESIGN REQUIREMENTS

13.2.1 Occupancy importance factor. Ideally, most of the lateral displacement of an isolated structure will be accommodated by deformation of the isolation system rather than distortion of the structure above. Accordingly, the lateral-load-resisting system of the structure above the isolation system should be designed to have sufficient stiffness and strength to avoid large, inelastic displacements. For this reason, the Provisions contains criteria that limit the inelastic response of the structure above the isolation system. Although damage control for the design-level earthquake is not an explicit objective of the Provisions, an isolated structure designed to limit inelastic response of the structural system also will reduce the level of damage that would otherwise occur during an earthquake. In general, isolated structures designed in conformance with the Provisions should be able:

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

The above performance objectives for isolated structures considerably exceed the performance anticipated for fixed-base structures during moderate and major earthquakes. Table C13.2-1 provides a tabular comparison of the performance expected for isolated and fixed-base structures designed in accordance with the Provisions. Loss of function is not included in Table C13.2-1. For certain (fixed-base) facilities, loss of function would not be expected to occur until there is significant structural damage causing closure or restricted access to the building. In other cases, the facility could have only limited or no structural damage but would not be functional as a result of damage to vital nonstructural components and contents. Isolation would be expected to mitigate structural and nonstructural damage and to protect the facility against loss of function.

Table C13.2-1  Protection Provided by NEHRP Recommended Provisions for Minor, Moderate, and Major Levels of Earthquake Ground Motion

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Earthquake Ground Motion Level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minor</td>
</tr>
<tr>
<td>Life safetya</td>
<td>F, I</td>
</tr>
<tr>
<td>Structural damageb</td>
<td>F, I</td>
</tr>
<tr>
<td>Nonstructural damagec (contents damage)</td>
<td>F, I</td>
</tr>
</tbody>
</table>

a Loss of life or serious injury is not expected.
b Significant structural damage is not expected.
c Significant nonstructural (contents) damage is not expected.
F indicates fixed base; I indicates isolated.

13.2.3.1 Design spectra. Site-specific design spectra must be developed for both the design earthquake and the maximum considered earthquake if the structure is located at a site with $S_1$ greater than 0.60 or on a Class F site. All requirements for spectra are in Sec. 3.3 and 3.4.

13.2.4 Procedure selection. The design requirements permit the use of one of three different analysis procedures for determining the design-level seismic loads. The first procedure uses a simple, lateral-force formula (similar to the lateral-force coefficient now used in conventional building design) to prescribe peak lateral displacement and design force as a function of spectral acceleration and isolated-building period and damping. The second and third methods, which are required for geometrically complex or especially flexible buildings, rely on dynamic analysis procedures (either response spectrum or time history) to determine peak response of the isolated building.
The three procedures are based on the same level of seismic input and require a similar level of performance from the building. There are benefits in performing a more complex analysis in that slightly lower design forces and displacements are permitted as the level of analysis becomes more sophisticated. The design requirements for the structural system are based on the design earthquake, a severe level of earthquake ground motion defined as two-thirds of the maximum considered earthquake. The isolation system—including all connections, supporting structural elements, and the “gap”—is required to be designed (and tested) for 100 percent of maximum considered earthquake demand. Structural elements above the isolation system are not required to be designed for the full effects of the design earthquake, but may be designed for slightly reduced loads (that is, loads reduced by a factor of up to 2.0) if the structural system has sufficient ductility, etc., to respond inelastically without sustaining significant damage. A similar fixed-base structure would be designed for loads reduced by a factor of 8 rather than 2.

This section delineates the requirements for the use of the equivalent lateral force procedure and dynamic methods of analysis. The limitations on the simplified lateral-force design procedure are quite severe at this time. Limitations cover the site location with respect to active faults; soil conditions of the site, the height, regularity and stiffness characteristics of the building; and selected characteristics of the isolation system. Response-history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a “nonlinear” isolation system including, but not limited to, isolation systems utilizing friction or sliding surfaces, isolation systems with effective damping values greater than about 30 percent of critical, isolation systems not capable of producing a significant restoring force, and isolation systems that restrain or limit extreme earthquake displacement;

2. Isolated structures with a “nonlinear” structure (above the isolation system) including, but not limited to, structures designed for forces that are less than those specified by the Provisions for “essentially-elastic” design; and

3. Isolated structures located on Class F site (that is, very soft soil).

Lower-bound limits on isolation system design displacements and structural-design forces are specified by the Provisions in Sec. 13.4 as a percentage of the values prescribed by the equivalent-lateral-force design formulas, even when dynamic analysis is used as the basis for design. These lower-bound limits on key design parameters ensure consistency in the design of isolated structures and serve as a “safety net” against gross under-design. Table C13.2-2 provides a summary of the lower-bound limits on dynamic analysis specified by the Provisions.

13.2.4.3 Variations in material properties: For analysis, the mechanical properties of seismic isolators are generally based on values provided by isolator manufacturers. The properties are evaluated by prototype testing, which often occurs shortly after the isolators have been manufactured, and checked with respect to the values assumed for design. Unlike conventional materials whose properties do not vary substantially with time, seismic isolators are composed of materials whose properties will generally vary with time. Because (a) mechanical properties can vary over the life span of a building, and (b) the testing protocol of Section 13.6 cannot account for the effects of aging, contamination, scragging (temporary degradation of mechanical properties with repeated cycling), temperature, velocity effects, and wear, the engineer-of-record must account for these effects by explicit analysis. One strategy for accommodating these effects makes use of property modification factors, which was introduced by Constantinou et al. (1999) in the AASHTO Guide Specification for Seismic Isolation Design (AASHTO, 1999). Constantinou et al. (1999) also provides information on variations in material properties for sliding isolation systems. Thompson et al. (2000) and Morgan et al. (2001) provide information on variations in material properties for elastomeric bearings.
Table C13.2-2  Lower-Bound Limits on Dynamic Procedures Specified in Relation to ELF Procedure Requirements

<table>
<thead>
<tr>
<th>Design Parameter</th>
<th>ELF Procedure</th>
<th>Dynamic Procedure</th>
<th>Response Spectrum</th>
<th>Response History</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design displacement – (D_D)</td>
<td>(D_D = (g/4\pi^2)(S_D T_D / B_D))</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Total design displacement - (D_T)</td>
<td>(D_T \geq 1.1 D)</td>
<td>(\geq 0.9 D_T)</td>
<td>(\geq 0.9 D_T)</td>
<td></td>
</tr>
<tr>
<td>Maximum displacement – (D_M)</td>
<td>(D_M = (g/4\pi^2)(S_M T_M / B_M))</td>
<td>–</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Total maximum displacement - (D_{TM})</td>
<td>(D_{TM} \geq 1.1 D_M)</td>
<td>(\geq 0.8 D_{TM})</td>
<td>(\geq 0.8 D_{TM})</td>
<td></td>
</tr>
<tr>
<td>Design shear – (V_b) (at or below the isolation system)</td>
<td>(V_b = k_{D_{max}} D_D)</td>
<td>(\geq 0.9 V_b)</td>
<td>(\geq 0.9 V_b)</td>
<td></td>
</tr>
<tr>
<td>Design shear – (V_s) (“regular” superstructure)</td>
<td>(V_s = k_{D_{max}} D_D / R_I)</td>
<td>(\geq 0.8 V_s)</td>
<td>(\geq 0.6 V_s)</td>
<td></td>
</tr>
<tr>
<td>Design shear – (V_s) (“irregular” superstructure)</td>
<td>(V_s = k_{D_{max}} D_D R_I)</td>
<td>(\geq 1.0 V_s)</td>
<td>(\geq 0.8 V_s)</td>
<td></td>
</tr>
<tr>
<td>Drift (calculated using (R_I) for (C_d))</td>
<td>(0.015 h_{sx})</td>
<td>(0.015 h_{sx})</td>
<td>(0.020 h_{sx})</td>
<td></td>
</tr>
</tbody>
</table>

13.2.5 Isolation system

13.2.5.1 Environmental conditions. Environmental conditions that may adversely affect isolation system performance should be thoroughly investigated. Significant research has been conducted on the effects of temperature, aging, etc., on isolation systems since the 1970s in Europe, New Zealand, and the United States.

13.2.5.2 Wind forces. Lateral displacement over the depth of the isolator zone resulting from wind loads should be limited to a value similar to that required for other story heights.

13.2.5.3 Fire resistance. In the event of a fire, the isolation system should be capable of supporting the weight of the building, as required for other vertical-load-supporting elements of the structure, but may have diminished functionality for lateral (earthquake) load.

13.2.5.4 Lateral-restoring force. The isolation system should be configured with a lateral-restoring force sufficient to avoid significant residual displacement as a result of an earthquake, such that the isolated structure will not have a stability problem so as to be in a condition to survive aftershocks and future earthquakes.

13.2.5.5 Displacement restraint. The use of a displacement restraint is not encouraged by the Provisions. Should a displacement restraint system be implemented, explicit analysis of the isolated structure for maximum considered earthquake is required to account for the effects of engaging the displacement restraint.

13.2.5.6 Vertical-load stability. The vertical loads to be used in checking the stability of any given isolator should be calculated using bounding values of dead load and live load and the peak earthquake demand of the maximum considered earthquake. Since earthquake loads are reversible in nature, peak earthquake load should be combined with bounding values of dead and live load in a manner which 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 maximum considered earthquake displacement of the isolation system.
13.2.5.7 **Overturning.** The intent of this requirement is to prevent both global structural overturning and overstress of elements due to local uplift. Uplift in a braced frame or shear wall is acceptable so long as the isolation system does not disengage from its horizontal-resisting connection detail. The connection details used in some isolation systems are such that tension is not permitted on the system. If the tension capacity of an isolation system is to be utilized to resist uplift forces, then component tests should be performed to demonstrate the adequacy of the system to resist tension forces at the design displacement.

13.2.5.8 **Inspection and replacement.** Although most isolation systems will not need to be replaced after an earthquake, it is good practice to provide for inspection and replacement. After an earthquake, the building should be inspected and any damaged elements should be replaced or repaired. It is advised that periodic inspections be made of the isolation system.

13.2.5.9 **Quality control.** A test and inspection program is necessary for both fabrication and installation of the isolation system. Because base isolation is a developing technology, it may be difficult to reference standards for testing and inspection. Reference can be made to standards for some materials such as elastomeric bearings (ASTM D 4014). Similar standards are required for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality should be developed for each project. The requirements will vary with the type of isolation system used.

13.2.6 **Structural system**

13.2.6.1 **Horizontal distribution of force**

13.2.6.2 **Building separations.** A minimum separation between the isolated structure and a rigid obstruction is required to allow free movement of the superstructure in all lateral directions during an earthquake. Provision should be made for lateral motion greater than the design displacement, since the exact upper limit of displacement cannot be precisely determined.

13.2.7 **Elements of structures and nonstructural components.** To accommodate the differential movement between the isolated building and the ground, provision for flexible utility connections should be made. In addition, rigid structures crossing the interface (such as stairs, elevator shafts and walls) should have details to accommodate differential motion at the isolator level without sustaining damage sufficient to threaten life safety.

13.3 **EQUIVALENT LATERAL FORCE PROCEDURE**

13.3.2 **Minimum lateral displacements.** The lateral displacement given by Eq. 13.3-1 approximates peak design earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, $T_D$, and equivalent viscous damping, $\beta_D$, and the lateral displacement given by Eq. 13.3-3 approximates peak maximum considered earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, $T_M$, and equivalent viscous damping, $\beta_M$.

Equation 13.3-1 is an estimate of peak displacement in the isolation system for the design earthquake. In this equation, the spectral acceleration term, $S_{DD}$, is the same as that required for design of a conventional fixed-base structure of period, $T_D$. A damping term, $B_D$, is used to decrease (or increase) the computed displacement when the equivalent damping coefficient of the isolation system is greater (or smaller) than 5 percent of critical damping. Values of coefficient $B_D$ (or $B_M$ for the maximum considered earthquake) are given in Table 13.3-1 for different values of isolation system damping, $\beta_D$ (or $\beta_M$).

A comparison of values obtained from Eq. 13.3-1 and those obtained from nonlinear time-history analyses are given in Kircher et al. (1988) and Constantinou et al. (1993).

Consideration should be given to possible differences in the properties of the isolation system used for design and the properties of isolation system actually installed in the building. Similarly, consideration should be given to possible changes in isolation system properties due to different design conditions or load combinations. If the true deformational characteristics of the isolation system are not stable or vary...
Seismically Isolated Structure Design Requirements

with the nature of the load (being rate-, amplitude-, or time-dependent), the design displacements should be based on deformational characteristics of the isolation system that give the largest possible deflection \( (kD_{\text{min}}) \), the design forces should be based on deformational characteristics of the isolation system that give the largest possible force \( (kD_{\text{max}}) \), and the damping level used to determine design displacements and forces should be based on deformational characteristics of the isolation system that represent the minimum amount of energy dissipated during cyclic response at the design level.

The configuration of the isolation system for a seismically isolated building or structure should be selected in such a way as to minimize any eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system. In this way, the effect of torsion on the displacement of isolation elements will be reduced. As for conventional structures, allowance for accidental eccentricity in both horizontal directions must be considered. Figure C13.3-1 defines the terminology used in the Provisions. Equation 13.3-5 (or Eq. 13.3-6 for the maximum considered earthquake) provides a simplified formulae for estimating the response due to torsion in lieu of a more refined analysis. The additional component of displacement due to torsion increases the design displacement at the corner of the structure by about 15 percent (for a perfectly square building in plan) to about 30 percent (for a very long, rectangular building) if the eccentricity is 5 percent of the maximum plan dimension. Such additional displacement, due to torsion, is appropriate for buildings with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the building or certain sliding systems that minimize the effects of mass eccentricity will have reduced displacements due to torsion. The Provisions permits values of \( D_T \) as small as \( 1.1D_0 \), with proper justification.
13.3.3 Minimum lateral forces. Figure C13.3-2 defines the terminology below and above the isolation system. Equation 13.3-7 gives peak seismic shear on all structural components at or below the seismic interface without reduction for ductile response. Equation 13.3-8 specifies the peak seismic shear for design of structural systems above the seismic interface. For structures that have appreciable inelastic-deformation capability, this equation includes an effective reduction factor of up to 2 for response beyond the strength-design level.

The basis for the reduction factor is that the design of the structural system is based on strength-design procedures. A factor of at least 2 is assumed to exist between the design-force level and the true-yield level of the structural system. An investigation of 10 specific buildings indicated that this factor varied between 2 and 5 (ATC, 1982). Thus, a reduction factor of 2 is appropriate to ensure that the structural system remains essentially elastic for the design earthquake.

In Sec. 13.3.3.2, the limitations given on $V_S$ ensure that there is at least a factor of 1.5 between the nominal yield level of the superstructure and (1) the yield level of the isolation system, (2) the ultimate capacity of a sacrificial wind-restraint system which is intended to fail and release the superstructure during significant lateral load, or (3) the break-away friction level of a sliding system.

These limitations are essential to ensure that the superstructure will not yield prematurely before the isolation system has been activated and significantly displaced.

The design shear force, $V_S$, specified by the requirements of this section ensures that the structural system of an isolated building will be subjected to significantly lower inelastic demands than a conventionally designed structure. Further reduction in $V_S$, such that the inelastic demand on a seismically isolated structure would be the same as the inelastic demand on a conventionally designed structure, was not considered during development of these requirements but may be considered in the future.

If the level of performance of the isolated structure is desired to be greater than that implicit in these requirements, then the denominator of Eq. 13.3-8 may be reduced. Decreasing the denominator of Eq. 13.3-8 will lessen or eliminate inelastic response of the superstructure for the design-basis event.

13.3.4 Vertical distribution of forces. Equation 13.3-9 describes the vertical distribution of lateral force based on an assumed triangular distribution of seismic acceleration over the height of the structure.
above the isolation interface. Constantinou et al. (1993) provides a good summary of recent work which demonstrates that this vertical distribution of force will always provide a conservative estimate of the distributions obtained from more detailed, nonlinear analysis studies.

13.3.5 Drift limits. The maximum story drift permitted for design of isolated structures varies depending on the method of analysis used, as summarized in Table C13.3-1. For comparison, the drift limits prescribed by the Provisions for fixed-base structures also are summarized in Table C13.3-1.

<table>
<thead>
<tr>
<th>Structure</th>
<th>Seismic Use Group</th>
<th>Fixed-Base</th>
<th>Isolated</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buildings (other than masonry) four stories or less in height with component drift design</td>
<td>I</td>
<td>$0.025h_{sx}/(C_d/R)$</td>
<td>$0.015h_{sx}$</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>$0.020h_{sx}/(C_d/R)$</td>
<td>$0.015h_{sx}$</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>$0.015h_{sx}/(C_d/R)$</td>
<td>$0.015h_{sx}$</td>
</tr>
<tr>
<td>Other (non-masonry) buildings</td>
<td>I</td>
<td>$0.020h_{sx}/C_d/R$</td>
<td>$0.015h_{sx}$</td>
</tr>
<tr>
<td></td>
<td>II</td>
<td>$0.015h_{sx}/C_d/R$</td>
<td>$0.015h_{sx}$</td>
</tr>
<tr>
<td></td>
<td>III</td>
<td>$0.010h_{sx}/C_d/R$</td>
<td>$0.015h_{sx}$</td>
</tr>
</tbody>
</table>

Drift limits in Table C13.3-1 are divided by $C_d/R$ for fixed-base structures since displacements calculated for lateral loads reduced by $R$ are factored by $C_d$ before checking drift. The $C_d$ term is used throughout the Provisions 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 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 are generally more conservative than those for conventional, fixed-base structures, even when fixed-base structures are designed as Seismic Use Group III buildings.

13.4 DYNAMIC PROCEDURES

This section specifies the requirements and limits for dynamic procedures. The design displacement and force limits on response spectrum and response history procedures are given in Table C13.2-1. A more-detailed or refined study can be performed in accordance with the analysis procedures described in this section. The intent of this section is to provide procedures which are compatible with the minimum requirements of Sec. 13.3. Reasons for performing a more refined study include:

1. The importance of the building.
2. The need to analyze possible structure/isolation-system interaction when 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 when the structure above the isolation system is irregular.
4. The desirability of using site-specific ground-motion data, especially for soft soil types (Site Class F) or for structures located where $S_I$ is greater than 0.60.
5. The desirability of explicitly modeling the deformational characteristics of the base-isolation system. This is especially important for systems that have damping characteristics that are amplitude-dependent, rather than velocity-dependent, since it is difficult to determine an appropriate value of equivalent viscous damping for these systems.
Sec. 13.2.4 of this commentary discusses other conditions which require use of the response history procedure.

When 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 to be based on the maximum of the results of not less than three separate analyses, each using a different pair of horizontal time histories. Each pair of horizontal time histories should:

1. Be of a duration consistent with the design earthquake or the maximum considered earthquake,
2. Incorporate near-field phenomena, as appropriate, and
3. Have response spectra for which the square-root-of-the-sum-of-the-squares combination of the two horizontal components equals or exceeds 1.3 times the “target” spectrum at each spectral ordinate.

The average value of seven time histories is a standard required by the nuclear industry and is considered appropriate for nonlinear response history analysis of seismically isolated structures.

13.5 DESIGN REVIEW

Review of the design and analysis of the isolation system and design review of the isolator testing program is mandated by the Provisions for two key reasons:

1. The consequences of isolator failure could be catastrophic.
2. Isolator design and fabrication technology is evolving rapidly and may be based on technologies unfamiliar to many design professionals.

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

The review team should be formed prior to the development of design criteria (including site-specific ground shaking criteria) and isolation system design options. Further, the review team should have full access to all pertinent information and the cooperation of the design team and regulatory agencies involved with the project.

13.6 TESTING

The design displacements and forces developed from the Provisions are predicated on the basis that the deformational characteristics of the base isolation system have been previously defined by a comprehensive set of tests. If a comprehensive amount of test data are not available on a system, major design alterations in the building may be necessary after the tests are complete. This would result from variations in the isolation-system properties assumed for design and those obtained by test. Therefore, it is advisable that prototype systems be tested during the early phases of design, if sufficient test data is not available on an isolation system.

Typical force-deflection (or hysteresis) loops are shown in Figure C13.6-1; also included are the definitions of values used in Sec. 13.6.2.
The required sequence of tests will verify experimentally:

1. The assumed stiffness and capacity of the wind-restraining mechanism;
2. The variation in the isolator's deformational characteristics with amplitude (and with vertical load, if it is a vertical load-carrying member);
3. The variation in the isolator's deformational characteristics for a realistic number of cycles of loading at the design displacement; and
4. The ability of the system to carry its maximum and minimum vertical loads at the maximum displacement.

Force-deflection tests are not required if similarly sized components have been tested previously using the specified sequence of tests.

Variations in effective stiffness greater than 15 percent over 3 cycles of loading at a given amplitude, or greater than 20 percent over the larger number of cycles at the design displacement, would be cause for rejection. The variations in the vertical loads required for tests of isolators which carry vertical, as well as lateral, load are necessary to determine possible variations in the system properties with variations in overturning force. The appropriate dead loads and overturning forces for the tests are defined as the average loads on a given type and size of isolator for determining design properties and are the absolute maximum and minimum loads for the stability tests.

### 13.6.4 Design properties of the isolation system

#### 13.6.4.1 Maximum and minimum effective stiffness.

The effective stiffness is determined from the hysteresis loops shown in Figure C13.6-1. Stiffness may vary considerably as the test amplitude.
increases but should be reasonably stable (within 15 percent) for more than 3 cycles at a given amplitude.

The intent of these requirements is to ensure that the deformational properties used in design result in the maximum design forces and displacements. For determining design displacement, this means using the lowest damping and effective-stiffness values. For determining design forces, this means using the lowest damping value and the greatest stiffness value.

**13.6.4.2 Effective damping.** The determination of equivalent viscous damping is reasonably reliable for systems whose damping characteristics are velocity dependent. For systems that have amplitudedependent, energy-dissipating mechanisms, significant problems arise in determining an equivalent viscous-damping value. Since it is difficult to relate velocity and amplitude-dependent phenomena, it is recommended that when the equivalent-viscous damping assumed for the design of amplitude-dependent, energy-dissipating mechanisms (such as pure-sliding systems) is greater than 30 percent, then the design-basis force and displacement should be determined using the response history procedure, as discussed in Commentary Sec. 13.2.4.

**REFERENCES**


Morgan, T., A. S. Whittaker, and A. C. Thompson. 2001. “Cyclic behavior of high-damping rubber bearings,” Proceedings, Fifth World Congress on Joints, Bearings and Seismic Systems for Concrete Structures, American Concrete Institute, Rome, Italy.

