Chapter 13 Commentary

SEISMICALLY ISOLATED STRUCTURES DESIGN REQUIREMENTS

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 decoupling the structure from the ground motion, the level of response in the structure can be significantly reduced from the level that would otherwise occur in a conventional fixed-base building. Conversely, seismic isolation permits designing with a reduced level of earthquake load to achieve the same degree of seismic protection and reliability as a conventional fixed-base building.

The potential advantages of seismic isolation and the recent advancements in isolation-system products already have led to the design and construction of over 100 seismically isolated buildings and bridges in the United States. A significant amount of research, development, and application activity has occurred over the past 20 years. The following references provide a summary of some of the work that has been performed: Applied Technology Council (1986, 1993), ASCE Structures Congress (1989, 1991, 1993 and 1995), EERI Spectra (1990), Skinner, et al. (1993), U.S. Conference on Earthquake Engineering (1990 and 1994), and World Conference on Earthquake Engineering (1988, 1992 and 1996).

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 October 1996 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 isolation appendix of the *UBC* has been updated on an annual basis since that time and the most current version of these regulations may be found in the 1997 *UBC*.

2. In the late 1980s, the *building* Safety Board (BSB) of California, Office of the State Architect, adopted *An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation* based on recommendations of SEAOC. These methods were used for regulation of California hospitals until the BSB replaced them with the 1991 *UBC* appendix (with slight modification). The current version of these regulations may be found in 1995 *California Building Code*.

3. 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*) are now available as FEMA 273. The design and analysis methods of the *NEHRP Guidelines* 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.

During development of the 1994 *Provisions*, it was decided to use the latest version (1993 approved changes) of the SEAOC/*UBC* provisions as a basis for the development of the requirements included in the *Provisions*. The only significant changes involved an appropriate conversion to strength design and making the requirements applicable on a national basis. For the 1997 *Provisions*, it was decided to incorporate the latest version of the SEAOC/*UBC* provisions (1997 *UBC*). Since the 1997 *UBC* is now based on strength design, the 1997 *UBC* and the 1997 *Provisions* are almost identical, except for seismic criteria. The seismic criteria of the *Provisions* are based on the new national earthquake maps (developed by the Seismic Design Procedures Group) which can be substantially different from the seismic criteria of the 1997 *UBC*.

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 (e.g., force-deflection characteristics and damping).

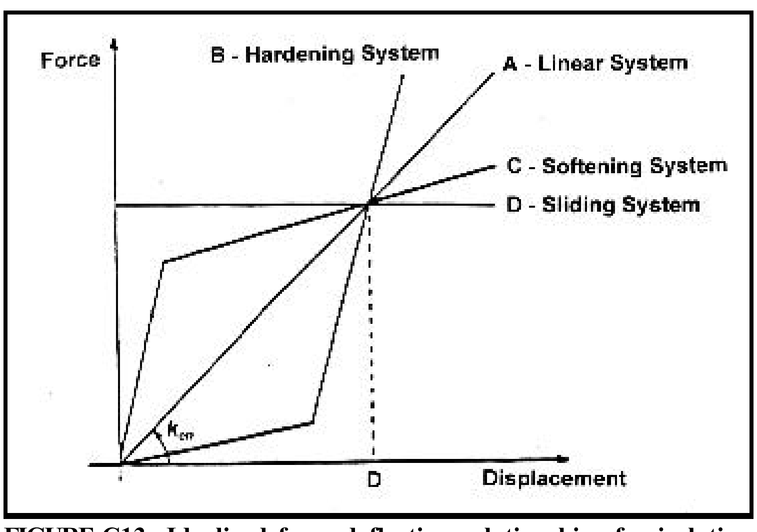

FIGURE C13 Idealized force-deflection relationships for isolation systems (stiffness effects of sacrificial wind-restraint systems not shown for clarity).

Conceptually, there are four basic types of isolation system force-deflection relationships. These idealized relationships are shown in Figure C13 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 GENERAL: The design requirements permit the use of one of three different analysis procedures for determining the design-basis 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 (i.e., 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.

Ideally, lateral displacement of an isolated structure will result, predominantly due to the deformations of the isolation system, rather than in 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-basis 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 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, 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 C13.1 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 C13.1. 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 protect the facility against loss of function.

The requirements of Chapter 13 provide isolator design displacements, structure-design-shear forces, 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.

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

Risk Category	**Earthquake Ground Motion Level**		
	Minor	**Moderate**	**Major**
Life safety[a]	F/I	F/I	F/I
Structural damage[b]	F/I	F/I	I
Nonstructural damage[c] (contents damage)	F/I	I	I

[a] Loss of life or serious injury is not expected for fixed-*base* (F) or isolated (I) *buildings*.

[b] Significant structural damage is not expected for fixed-*base* (F) or isolated (I) *buildings*.

[c] Significant nonstructural (contents) damage is not expected for fixed-*base* (F) or isolated (I) *buildings*.

13.2 CRITERIA SELECTION: This section delineates the requirements for the use of the equivalent-lateral-force and dynamic methods of analysis and the conditions for developing a site-specific response spectrum. 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 the characteristics of the isolation system. In fact, the current limitations will necessitate a dynamic analysis for most isolated structures. Additionally, time-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. (i.e., very soft soil).

The restrictions placed on the use of equivalent-lateral-force design procedures effectively require dynamic analysis for virtually all isolated structures. However, 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 provides a summary of the lower-bound limits on dynamic analysis specified by the *Provisions*.

TABLE C13.2 Lower-Bound Limits on Dynamic Analysis Specified as a Percentage of Static-Analysis Design Requirements

Design Parameter	Static Analysis	Dynamic Analysis	
		Response Spectrum	Time History
Design Displacement - D_D	$D_D = (g/4B^2)(S_{D1}T_D/B_D)$	–	–
Total Design Displacement - D_T	$D_T \, \$ \, 1.1D$	$\$ \, 0.9D_T$	$\$ \, 0.9D_T$
Maximum Displacement - D_M	$D_M = (g/4B^2)(S_{M1}T_M/B_M)$	–	–
Total Maximum Displacement - D_{TM}	$D_{TM} \, \$ \, 1.1D_M$	$\$ \, 0.8D_{TM}$	$\$ \, 0.8D_{TM}$
Design Shear - V_b (at or below the *Isolation System*)	$V_b = k_{Dmax}D_D$	$\$ \, 0.9V_b$	$\$ \, 0.9V_b$
Design Shear - V_s ("Regular" Superstructure)	$V_s = k_{Dmax}D_D/R_I$	$\$ \, 0.8V_s$	$\$ \, 0.6V_s$

Design Shear - V_s ("Irregular" Superstructure)	$V_s = k_{Dmax} D_D R_I$	$\geq 1.0V_s$	$\geq 0.8V_s$
Drift (calculated using R_I for C_d)	$0.015h_{sx}$	$0.015h_{sx}$	$0.020h_{sx}$

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 0.60g or on a Class F site. Lower limits are placed on these site-specific spectra and they must not be less than 80 percent of those given in Sec. 13.4.4.

13.3 EQUIVALENT LATERAL FORCE PROCEDURE: The lateral displacement given by Equation 13.3.3.1 approximates peak design earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_D, and equivalent viscous damping, β_D, and the lateral displacement given by Equation 13.3.3.3 approximates peak maximum considered earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_M, and equivalent viscous damping, β_{DM}.

13.3.3 Minimum Lateral Displacements: Equation 13.3.3.1 is an estimate of peak displacement in the isolation system for the design earthquake. In this equation, the spectral acceleration term, S_{D1}, 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 13.3.3.1. for different values of isolation system damping, β_D (or β_M).

A comparison of values obtained from Equation 13.3.3.1 and those obtained from nonlinear time-history analyses are given in references by Kircher et al. (1988), Lashkari and Kircher (1993) 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 with the nature of the load (i.e., rate, amplitude or time dependent), the design displacements should be based on deformational characteristics of the isolation system that give the largest possible deflection (k_{Dmin}) and the design forces should be based on deformational characteristics of the isolation system that give the largest possible force (k_{Dmax}). If the true deformational characteristics of the isolation system are not stable or vary with the nature of the load (i.e., rate, amplitude or time dependent), 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 C13.3.3 defines the terminology used in the *Provisions*. Equation 13.3.3.5-1 (or Equation 13.3.3.5-2 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 $1.1D_D$, with proper justification.

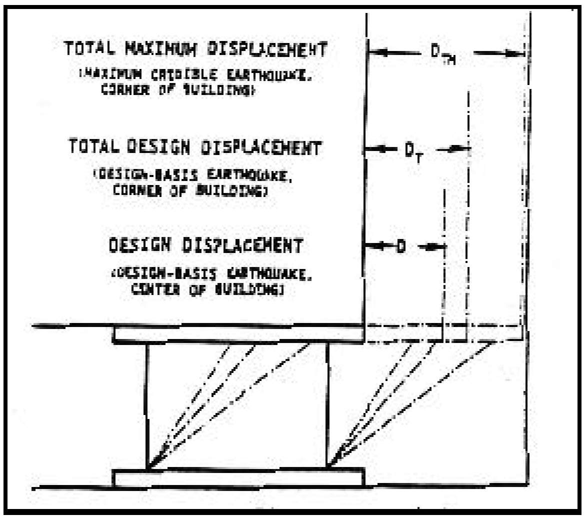

FIGURE C13.3.3 Displacement terminology.

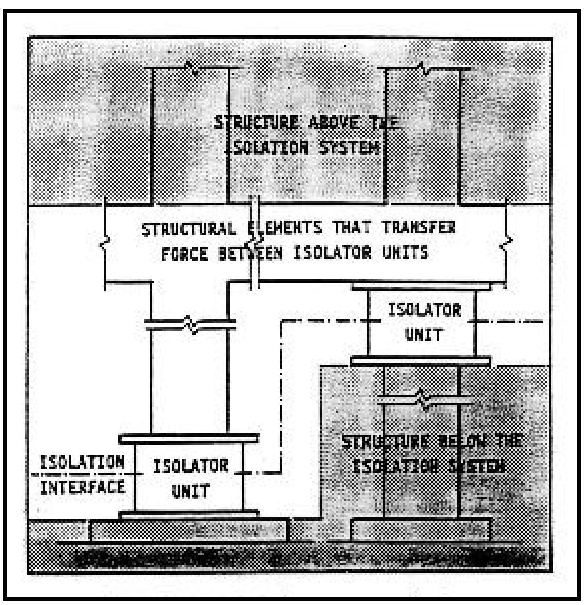

FIGURE C13.3.4 Isolation system terminology.

13.3.4 Minimum-Lateral Forces: Figure C13.3.4 defines the terminology below and above the *isolation system*. Equation 13.3.4.1 gives peak seismic shear on all structural *components* at or below the seismic interface without reduction for ductile response. Equation 13.3.4.2 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 (Applied Technology Council, 1982). Thus, a reduction factor of 2 is appropriate to ensure that the structural system remains essentially elastic for the design earthquake .

In Sec. 13.3.4.3, 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 less 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 Equation 13.3.4.2 may be reduced. Decreasing the denominator of Eq. 13.3.4.2 will lessen or eliminate inelastic response of the superstructure for the design-basis event.

13.3.5 Vertical Distribution of Force: Equation 13.3.5 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. References by Button (1993) and Constantinou et al. (1993) provide 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.

13.3.6 Drift Limits: The maximum interstory drift permitted for design of isolated structures varies depending on the method of analysis used, as summarized in Table C13.3.6. For comparison, the drift limits prescribed by the *Provisions* for fixed-base structures also are summarized in Table C13.3.6.

TABLE C13.3.6 Comparison of Drift Limits for Fixed-Base and Isolated Structures

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

Drift limits in Table C13.3.6 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 $\frac{1}{2}$ to $\frac{4}{5}$ 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 note that the drift limits for isolated structures are generally more conservative than those of conventional fixed-base structures, even when fixed-base structures are designed as Seismic Use Group III buildings.

13.4 DYNAMIC LATERAL RESPONSE PROCEDURE: This section specifies the requirements and limits of a dynamic analysis. The design displacement and force limits on a response-spectrum and time-history analysis are given in Table C13.2.

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 analysis 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 on sites with S_1 greater than 0.60g.
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, rather than velocity, dependent, since it is difficult to determine an appropriate value of equivalent viscous damping for these systems.

Additionally, time-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 SEAOC/UBC provisions for "essentially-elastic" design.
3. Isolated structures located on Class F sites (i.e., very soft soil).

When time-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 is to:

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 whose square-root-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 time-history analysis of seismically isolated structures.

13.5 LATERAL LOAD ON ELEMENTS OF STRUCTURES AND NONSTRUCTURAL COMPONENTS SUPPORTED BY BUILDINGS: 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, (i.e., stairs, elevator shafts and walls, should have details to accommodate differential motion at the isolator level without sustaining damage sufficient to threaten life safety.

13.6 DETAILED SYSTEM REQUIREMENTS: Environmental conditions that may adversely effect 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.

13.6.2.2 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.

13.6.2.3 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.

13.6.2.4 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 and be in a condition to survive aftershocks and future earthquakes.

13.6.2.5 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.

13.6.2.6 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.

13.6.2.7 Overturning: The intent of this requirement is to prevent global, structural overturning and overstress of elements due to local uplift. Uplift in a braced frame or shear wall is acceptable, provided 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 on resisting uplift forces, then component tests should be performed to demonstrate the adequacy of the system on resisting-tension forces at the design displacement.

13.6.2.8 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.

13.6.2.9 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 D4014). 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.

13.6.3 Structural System:

13.6.3.2 Building Separations: A minimum separation between the isolated structure and a rigid obstruction is required to allow free movement in all lateral directions of the superstructure 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.

13.8 DESIGN AND CONSTRUCTION REVIEW: Design 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.

13.9 REQUIRED TESTS OF THE ISOLATION SYSTEM: 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, then 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 C13.9; also included are the definitions of values used in Sec. 13.9.3.

The required sequence of tests will experimentally verify:

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.

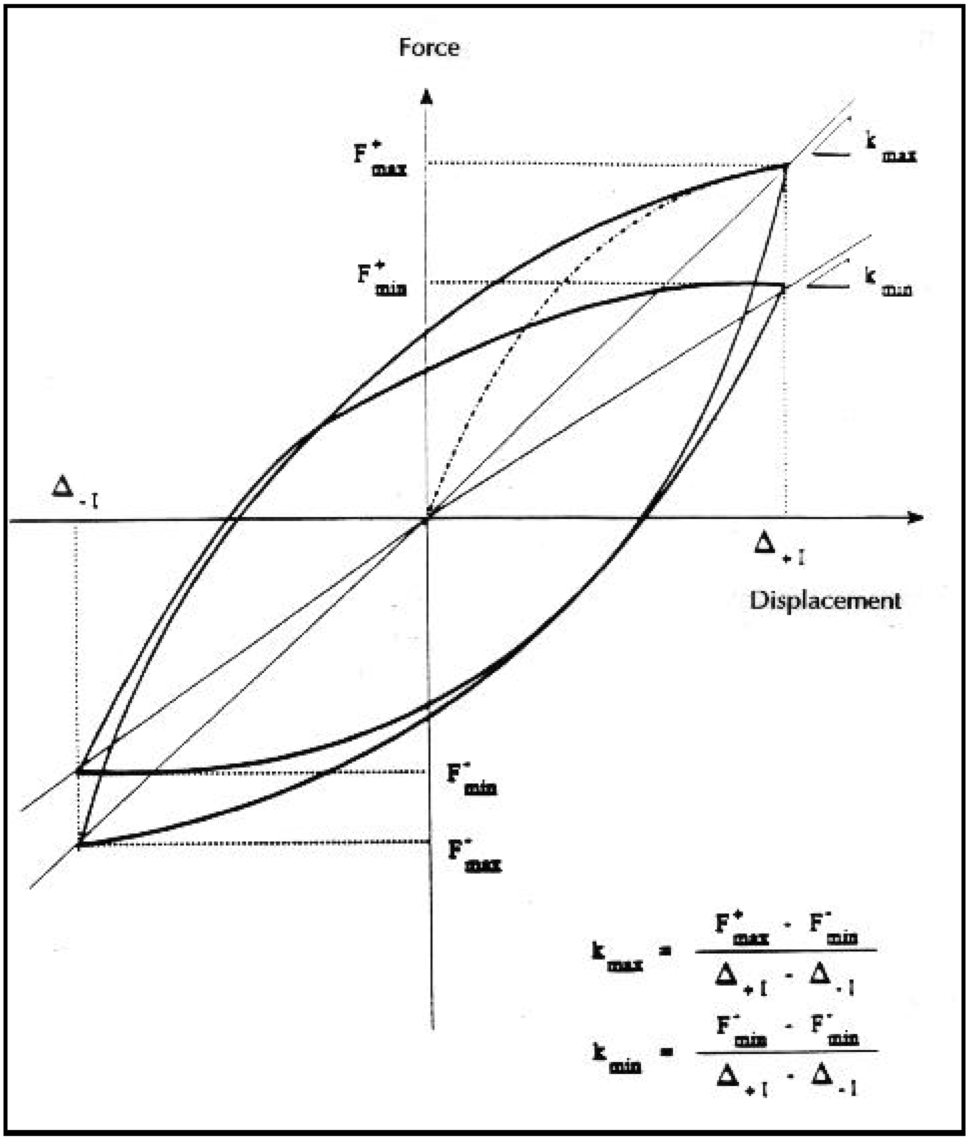

FIGURE 13.9 The effect of stiffness on an isolation bearing.

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

Variations in effective stiffness greater than ±15 percent over 3 cycles of loading at a given amplitude, or ±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.

13.9.5 Design Properties of the Isolated System:

13.9.5.1 Maximum and Minimum Effective Stiffness: The effective stiffness is determined from the hysteresis loops shown in Figure C13.9). Stiffness may vary considerably as the test amplitude increases but should be reasonably stable (±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.

13.9.5.2 Effective Damping: The determination of equivalent viscous damping is reasonably reliable for systems whose damping characteristics are velocity dependent. For systems that have amplitude-dependent, 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 (e.g., pure-sliding systems) is greater than 30 percent, then the design-basis force and displacement should be determined by the time-history-analysis method, as specified in Sec. C13.2.

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Appendix to Chapter 13 *Commentary*

STRUCTURES WITH DAMPING SYSTEMS

Appendix A13 is an entirely new addition to the 2000 *Provisions* that does not include a detailed commentary at this time. A detailed commentary will be developed during the next update cycle when it is expected that the appendix will be incorporated into the main body of the *Provisions.*

The balance of this section provides background on the underlying philosophy used by TS-12 to develop the appendix, the definition of the damping system, the concept of effective damping, and the calculation of earthquake response using linear analysis methods.

The basic approach taken by TS-12 in developing the appendix for *structures* with *damping systems* is based on the following concepts:

1. Appendix is applicable to all types of *damping systems*, including both *displacement-dependent damping devices* of hysteretic or friction systems and *velocity-dependent damping devices* of viscous or visco elastic systems.
2. Appendix provides minimum design criteria with performance objectives comparable to those of a *structure* with a conventional *seismic-force-resisting system* (but also permits design criteria that will achieve higher performance levels).
3. Appendix requires *structures* with a *damping system* to have a *seismic-force-resisting system* that provides a complete load path. The *seismic-force-resisting system* must comply with the requirements of the *Provisions*, except that the *damping system* may be used to meet drift limits.
4. Appendix requires design of *damping devices* and prototype testing of damper units for displacements, velocities and forces corresponding to those of the *maximum earthquake* (same approach as that used for *structures* with an *isolation system*).
5. Appendix provides "simple" linear static or response spectrum analysis methods for design of most *structures* that meet certain configuration and other limiting criteria (e.g., at least two *damping devices* at each story configured to resist torsion). *Appendix requires additional nonlinear time history analysis to confirm peak response of structures not meeting the criteria for linear analysis (and for structures close to faults).*

Damping System: The appendix defines the *damping system* as:

> The collection of structural elements that includes all individual *damping devices*, all structural elements or bracing required to transfer forces from damping devices to the *base* of the

structure and all structural elements required to transfer forces from *damping devices* to the *seismic-force-resisting system*.

The *damping system* is defined separately from the *seismic-force-resisting system*, although the two systems may have common elements. As illustrated in Figure CA13-1, the *damping system* may be external or internal to the *structure* and may have no shared elements, some shared elements, or all elements in common with the *seismic-force-resisting system*. Elements common to the *damping system* and the *seismic-force-resisting system* must be designed for combined loads of the two loads of the two systems.

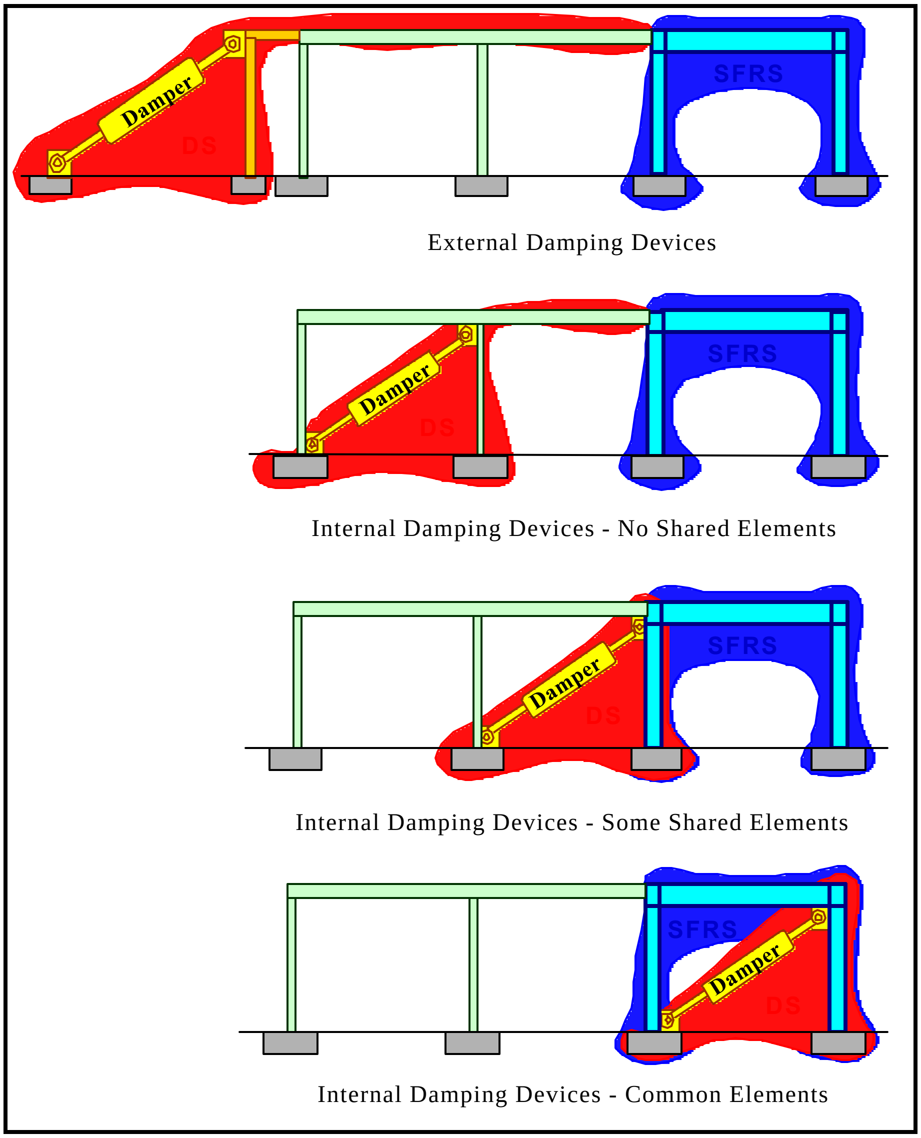

FIGURE C13A-1 Damping System (DS) and Seismic-Force-Resisting System (SRFS) Configurations

The *seismic-force-resisting system* may be thought of as a collection of lateral-force resisting elements of the *structure* if the *damping system* was not functional (e.g., *damping devices* were disconnected). This system is required to be designed for not less than 75 percent of the *base shear* of a conventional *structure* (not less than 100 percent, if the *structure* is highly irregular), using an *R* factor as defined in Table 5.2.2. This system provides both a safety net against damping system malfunction as well as the stiffness and strength necessary for the balanced lateral displacement of the damped *structure.*

The appendix requires the *damping system* to be designed for the actual (non-reduced) earthquake forces (e.g., peak force occurring in *damping devices*). For certain elements of the *damping system*, other than *damping devices*, limited yielding is permitted provided such behavior does not affect *damping system* function or exceed the amount permitted by the *Provisions* for elements of conventional *structures*.

The *damping devices* include damper units and all pins, bolts, gusset plates, brace extensions and other *components* required to connect damping devices to other elements of the structure. Following the same approach as that used for design of seismic isolators, *damping devices* must be designed for *maximum earthquake* displacements, velocities and forces. Likewise, prototype damper units must be fully tested to demonstrate adequacy for *maximum earthquake* loads and to establish design properties (e.g., effective damping).

Effective Damping

The appendix reduces the response of a *structure* with a *damping system* by the damping coefficient, *B*, based on the effective damping, $\$$, of the mode of interest. This is the same approach as that used by the *Provisions* for isolated structures. Values of the *B* coefficient recommended for design of damped *structures* are same as those in the *Provisions* for isolated *structures* at damping levels up to 30 percent, but now extend to higher damping levels based on a recent MCEER study by Constantinou, et al. Like isolation, effective damping of the fundamental-mode of a damped structure is based on the nonlinear force-deflection properties of the *structure*. For use with linear analysis methods, nonlinear properties of the *structure* are inferred from overstrength, S_O, and other terms of the *Provisions*. For nonlinear analysis methods, properties of the *structure* would be based on explicit modeling of the post-yield behavior of elements.

Figure CA13-2 illustrates reduction in design earthquake response of the fundamental mode due to effective damping coefficient, B_{1D}. The capacity curve is a plot of the nonlinear behavior of the fundamental mode in spectral acceleration/displacement coordinates. Damping reduction is applied at the effective (secant stiffness) period of the fundamental mode of vibration.

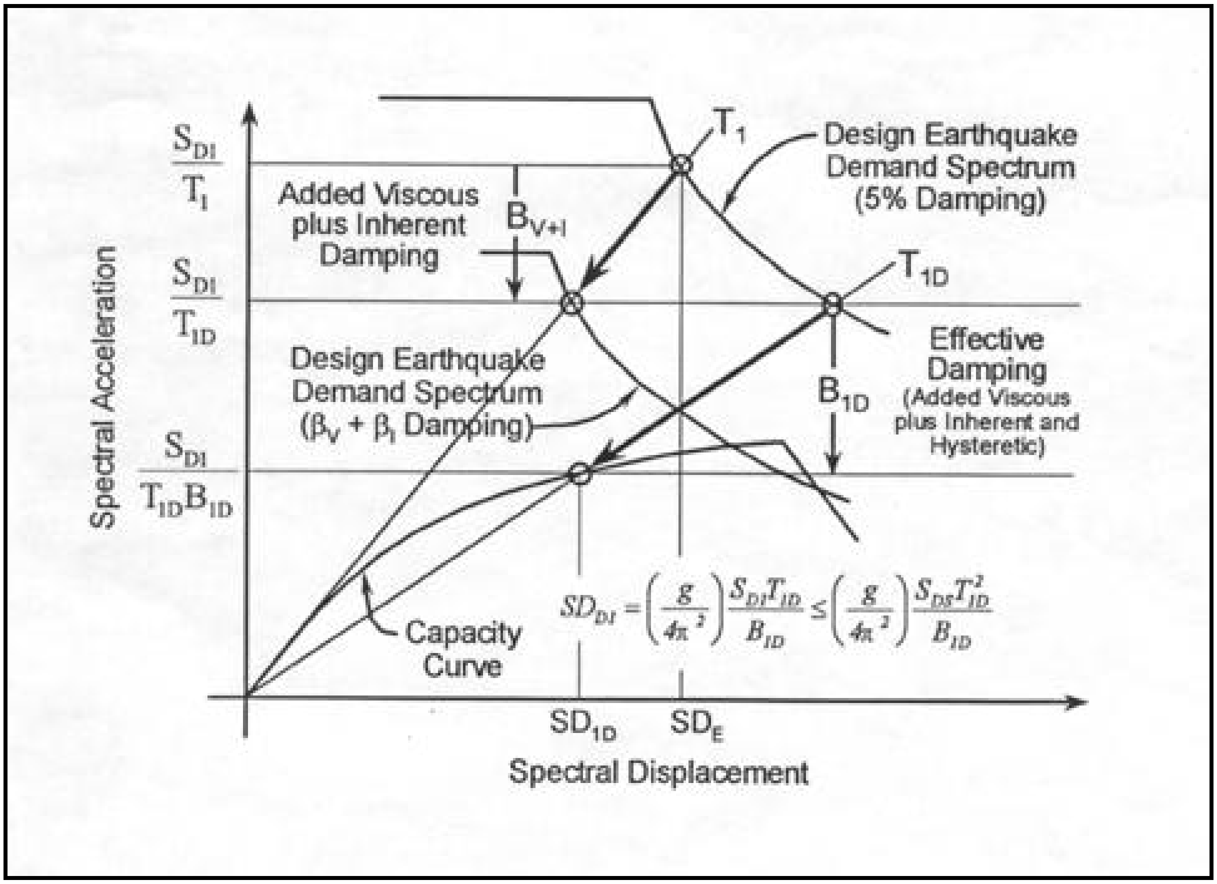

FIGURE C13A-2. Effective Damping Reduction of Design Demand

In general, effective damping is a combination of three components:

1. Inherent Damping $ Inherent damping of *structure* at or just below yield, excluding added viscous damping (e.g., typically assumed to be 5 percent of critical for structural systems without dampers).

2. Hysteretic Damping $ Post-yield hysteretic damping of the *seismic-force-resisting system* at the amplitude of interest (i.e., taken as 0 percent of critical at or below yield).

3. Added Viscous Damping $ Viscous component of the damping system (i.e., taken as 0 percent for hysteretic or friction-based damping systems).

Both hysteretic damping and the effects of added viscous damping are amplitude dependent and the relative contributions to total effective damping changes with the amount of post-yield response of the *structure*. For example, adding dampers to a structure decreases post-yield displacement of the *structure* and hence decreases the amount of hysteretic damping dissipated by the *seismic-force-resisting system*. If the displacements were reduced to the point of yield, the hysteretic component of effective damping would be zero and the effective damping would be equal to inherent damping plus added viscous damping. If there were no *damping system* (i.e., conventional *structure*), then effective damping would simply be equal to inherent damping (e.g., typically assumed to be 5 percent of critical for most conventional *structures*).

Design Earthquake Response Linear Analysis Methods

The appendix specifies *design earthquake* displacements, velocities and forces in terms of *design earthquake* spectral acceleration and modal properties. For linear static analysis, response is defined by two modes: (1) the fundamental mode, and (2) the residual mode. The residual mode is a new concept used to approximate the combined effects of higher modes. While typically of secondary importance to inter-story drift, higher modes can be a significant contributor to inter-story velocity and hence are important for design of *velocity-dependent damping devices*. For response spectrum analysis, higher modes are explicitly evaluated.

For either linear static or response spectrum analysis, response in the fundamental mode in the direction of interest is based on assumed nonlinear (pushover) properties of the *structure*. Nonlinear (pushover) properties, expressed in terms of *base shear* and roof displacement, are related to building capacity, expressed in terms of spectral coordinates, using mass participation and other fundamental-mode factors shown in Figure CA13-3. The conversion concepts and factors shown in Figure CA13-3 are the same as those defined in Chapter 9 of *NEHRP Guidelines* (FEMA 273) for seismic rehabilitation of a *structure* with *damping devices*.

When using linear analysis methods, the shape of the fundamental-mode pushover curve is not known and an idealized elasto-plastic shape is assumed, as shown in Figure CA13-4. The idealized pushover curve shares a common point with the actual pushover curve at the *design earthquake* displacement, D_{1D}. The idealized curve permits defining global ductility demand due to the *design earthquake*, μ_D, as the ratio of design displacement, D_{1D}, to the yield displacement, D_Y. This ductility factor is used in the calculation of various design factors and to set limits on the building ductility demand, μ_{max}, that are consistent with conventional building response limits. Design examples using linear analysis methods have been developed and found to compare well with the results of nonlinear time history analysis (Ramirez et al., 2000).

The appendix requires elements of the *damping system* to be designed for actual fundamental-mode *design earthquake* forces corresponding to a *base shear* value of V_Y (except *damping devices* are designed and prototype tested for *maximum earthquake* forces). Elements of the *seismic-force-resisting system* are designed for reduced fundamental-mode *base shear*, V_1, where force reduction is based on system overstrength, Ω_O, conservatively decreased by the ratio, C_d/R, for elastic analysis (when actual pushover strength is not known).

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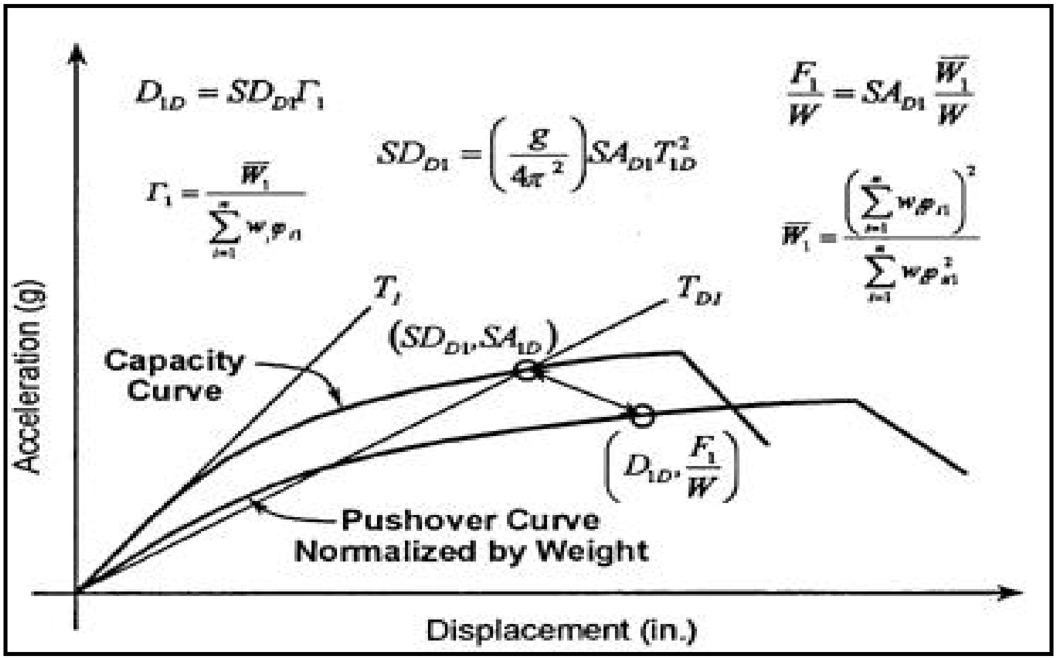

FIGURE C13A-3. Pushover and Capacity Curves

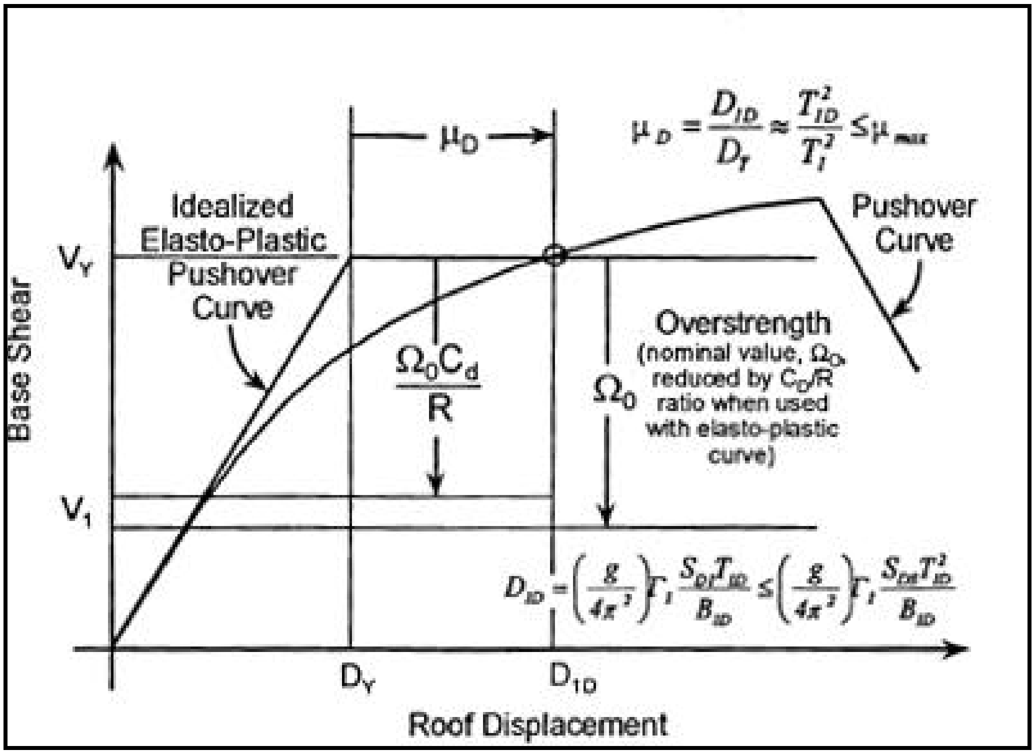

FIGURE C13A-4. Idealized Elasto-Plastic Pushover Curve Used for Linear Analysis