11

SEISMICALLY ISOLATED STRUCTURES

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Chapter 13 of the 2000 *NEHRP Recommended Provisions* addresses the design of buildings that incorporate a seismic isolation system. The *Provisions* provides essentially a stand alone set of design and analysis criteria for an isolation system. Chapter 13 defines load, design, and testing requirements specific to the isolation system and interfaces with the appropriate materials chapters for design of the structure above the isolation system and of the foundation and structural elements below.

A discussion of background, basic concepts, and analysis methods is followed by an example that illustrates the application of the *Provisions* to the structural design of a building with an isolation system. In this example, the building is a three-story emergency operations center (EOC) with a steel concentrically braced frame above the isolation system. Although the facility is hypothetical, it is of comparable size and configuration to actual base-isolated EOCs, and is generally representative of base-isolated buildings.

The EOC is located in San Francisco and has an isolation system that utilizes elastomeric bearings, a type of bearing commonly used for seismic isolation of buildings. The example comprehensively describes the EOC's configuration, defines appropriate criteria and design parameters, and develops a preliminary design using the equivalent lateral force (ELF) procedure of Chapter 13. It also includes a check of the preliminary design using dynamic analysis as required by the *Provisions* and specifies isolation system design and testing criteria.

Located in a region of very high seismicity, the building is subject to particularly strong ground motions. Large seismic demands pose a challenge for the design of base-isolated structures in terms of the capacity of the isolation system and the configuration of the structure above the isolation system. The isolation system must accommodate large lateral displacements (e.g., in excess of 2 ft). The structure above the isolation system should be configured to produce the smallest practical overturning loads (and uplift displacements) on the isolators. The example addresses these issues and illustrates that isolation systems can be designed to meet the requirements of the *Provisions*, even in regions of very high seismicity. Designing an isolated structure in a region of lower seismicity would follow the same approach. The isolation system displacement, overturning forces, and so forth would all be reduced, and therefore, easier to accommodate using available isolation system devices.

The isolation system for the building in the example is composed of high-damping rubber (HDR) elastomeric bearings. HDR bearings are constructed with alternating layers of rubber and steel plates all sheathed in rubber. The first base-isolated building in the United States employed this type of isolation system. Other types of isolation systems used to base isolate buildings employed lead-core elastomeric bearings (LR) and sliding isolators, such as the friction pendulum system (FPS). In regions of very high seismicity, viscous dampers have been used to supplement isolation system damping (and reduce displacement demand). Using HDR bearings in this example should not be taken as an endorsement of this particular type of isolator to the exclusion of others. The concepts of the *Provisions* apply to all types of isolations systems, and other types of isolators (and possible supplementary dampers) could have been used equally well in the example.

In addition to the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereafter, the *Provisions* and *Commentary*), the following documents are either referenced directly or are useful aids for the analysis and design of seismically isolated structures.

ATC 1996	Applied Technology Council. 1996. Seismic Evaluation and Retrofit of Buildings, ATC40.
Constantinou	Constantinou, M. C., P. Tsopelas, A. Kasalanati, and E. D. Wolff. 1999. *Property Modification Factors for Seismic Isolation Bearings,* Technical Report MCEER-99-0012. State University of New York.
CSI	Computers and Structures, Inc. (CSI). 1999. *ETABS Linear and Nonlinear Static and Dynamic Analysis and Design of Building Systems*.
FEMA 273	Federal Emergency Management Agency. 1997. *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, FEMA 273.
FEMA 222A	Federal Emergency Management Agency. 1995. *NEHRP Recommended Provisions for Seismic Regulations for New Buildings*, FEMA 222A.
91 UBC	International Conference of Building Officials. 1991. *Uniform Building Code*.
94 UBC	International Conference of Building Officials. 1994. *Uniform Building Code*.
Kircher	Kircher, C. A., G. C. Hart, and K. M. Romstad. 1989. "Development of Design Requirements for Seismically Isolated Structures" in *Seismic Engineering and Practice*, Proceedings of the ASCE Structures Congress, American Society of Civil Engineers, May 1989.
SEAOC 1999	Seismology Committee, Structural Engineers Association of California. 1999. *Recommended Lateral Force Requirements and Commentary*, 7th Ed.
SEAOC 1990	Seismology Committee, Structural Engineers Association of California. 1990. *Recommended Lateral Force Requirements and Commentary*, 5th Ed.
SEAONC Isolation	Structural Engineers Association of Northern California. 1986. *Tentative Seismic Isolation Design Requirements*.

Although the guide is based on the 2000 *Provisions* , it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantiative technical changes to the 2003 *Provisions* and its primary reference documents. While the general changes to the document are described , the deign examples and calculations have not been revised to reflect the changes to the 2003 *Provisions.*

In the 2003 edition of the *Provisions,* Chapter 13 has been restructured so that it is better integrated into the *Provisions* as a whole and is less of a stand alone set of requirements. Where they affect the design examples in this chapter, other significant changes to the 2003 *Provisions* and primary reference documents may be noted.

11.1 BACKGROUND AND BASIC CONCEPTS

Seismic isolation, commonly referred to as base isolation, is a design concept that presumes a structure can be substantially decoupled from potentially damaging earthquake ground motions. By decoupling the structure from ground shaking, isolation reduces the level of response in the structure that would otherwise occur in a conventional, fixed-base building. Conversely, base-isolated buildings may be designed with a reduced level of earthquake load to produce the same degree of seismic protection. That decoupling is achieved when the isolation scheme makes the fundamental period of the isolated structure several times greater than the period of the structure above the isolation system.

The potential advantages of seismic isolation and the advancements in isolation system products led to the design and construction of a number of isolated buildings and bridges in the early 1980s. This activity, in turn, identified a need to supplement existing seismic codes with design requirements developed specifically for such structures. These requirements assure the public that isolated buildings are safe and provide engineers with a basis for preparing designs and building officials with minimum standards for regulating construction.

Initial efforts developing design requirements for base-isolated buildings began with ad hoc groups of the Structural Engineers Association of California (SEAOC), whose Seismology Committee has a long history of contributing to codes. The northern section of SEAOC was the first to develop guidelines for the use of elastomeric bearings in hospitals. These guidelines were adopted in the late 1980s by the California Office of Statewide Health Planning and Development (OSHPD) and were used to regulate the first base-isolated hospital in California. At about the same time, the northern section of SEAOC published SEAONC Isolation, first set of general requirements to govern the design of base-isolated buildings. Most of the basic concepts for the design of seismically isolated structures found in the *Provisions* can be traced back to the initial work by the northern section of SEAOC.

By the end of the 1980s, the Seismology Committee of SEAOC recognized the need to have a more broadly based document and formed a statewide committee to develop design requirements for isolated structures Kircher. The "isolation" recommendations became an appendix to the 1990 SEAOC Blue Book. The isolation appendix was adopted with minor changes as a new appendix in the 1991 Uniform Building Code and has been updated every three years, although it remains largely the same as the original 91 UBC appendix. (SEAOC 1990 and 1999 are editions of SEAOC’s *Recommended Lateral Force Requirements and Commentary*, which is also known as the *Blue Book*.)

In the mid-1990s, the Provisions Update Committee of the Building Seismic Safety Council incorporated the isolation appendix of the 94 UBC into the 1994 *Provisions* (FEMA 222A). Differences between the *Uniform Building Code* (UBC) and the *Provisions* were intentionally minimized and subsequent editions of the *UBC* and the *Provisions* are nearly identical. Additional background may be found in the commentary to the 1999 SEAOC *Blue Book.*

The *Provisions* for designing the isolation system of a new building were used as the starting point for the isolation system requirements of the *NEHRP Guidelines for Seismic Rehabilitation of Buildings* (FEMA 273). FEMA 273 follows the philosophy that the isolation system for a rehabilitated building should be comparable to that for a new building (for comparable ground shaking criteria, etc.). The superstructure, however, could be quite different, and FEMA 273 provides more suitable design requirements for rehabilitating existing buildings using an isolation system.

11.1.1 Types of Isolation Systems

The *Provisions* requirements are intentionally broad, accommodating all types of acceptable isolation systems. To be acceptable, the *Provisions* requires the isolation system to:

1. Remain stable for maximum earthquake displacements,
2. Provide increasing resistance with increasing displacement,
3. Have limited degradation under repeated cycles of earthquake load, and
4. Have well-established and repeatable engineering properties (effective stiffness and damping).

The *Provisions* recognizes that the engineering properties of an isolation system, such as effective stiffness and damping, can change during repeated cycles of earthquake response (or otherwise have a range of values). Such changes or variability of design parameters are acceptable provided that the design is based on analyses that conservatively bound (limit) the range of possible values of design parameters.

The first seismic isolation systems used in buildings in the United States were composed of elastomeric bearings that had either a high-damping rubber compound or a lead core to provide damping to isolated modes of vibration. Other types of isolation systems now include sliding systems, such as the friction pendulum system (FPS), or some combination of elastomeric and sliding isolators. Some applications at sites with very strong ground shaking use supplementary fluid-viscous dampers in parallel with either sliding or elastomeric isolators to control displacement. While generally applicable to all types of systems, certain requirements of the *Provisions* (in particular, prototype testing criteria) were developed primarily for isolation systems with elastomeric bearings.

Isolation systems typically provide only horizontal isolation and are rigid or semi-rigid in the vertical direction. A rare exception to this rule is the full isolation (horizontal and vertical) of a building in southern California isolated by large helical coil springs and viscous dampers. While the basic concepts of the *Provisions* can be extended to full isolation systems, the requirements are only for horizontal isolation systems. The design of a full isolation system requires special analyses that explicitly include vertical ground shaking and the potential for rocking response.

Seismic isolation is commonly referred to as base isolation because the most common location of the isolation system is at or near the base of the structure. The *Provisions* does not restrict the plane of isolation to the base of the structure but does require the foundation and other structural elements below the isolation system to be designed for unreduced (R_I = 1.0) earthquake forces.

11.1.2 Definition of Elements of an Isolated Structure

The design requirements of the *Provisions* distinguish between structural elements that are either components of the isolation system or part of the structure below the isolation system (e.g., foundation) and elements of the structure above the isolation system. The isolation system is defined by the *Provisions* as:

> The collection of structural elements that includes all individual isolator units, all structural elements that transfer force between elements of the isolation system, and all connections to other structural elements. The isolation system also includes the wind-restraint system, energy-dissipation devices, and/or the displacement restraint system if such systems and devices are used to meet the design requirements of Chapter 13.

Figure 11.1-1 illustrates this definition and shows that the isolation system consists not only of the isolator units but also of the entire collection of structural elements required for the system to function properly. The isolation system typically includes segments of columns and connecting girders just above the isolator units because such elements resist moments (due to isolation system displacement) and their yielding or failure could adversely affect the stability of isolator units.

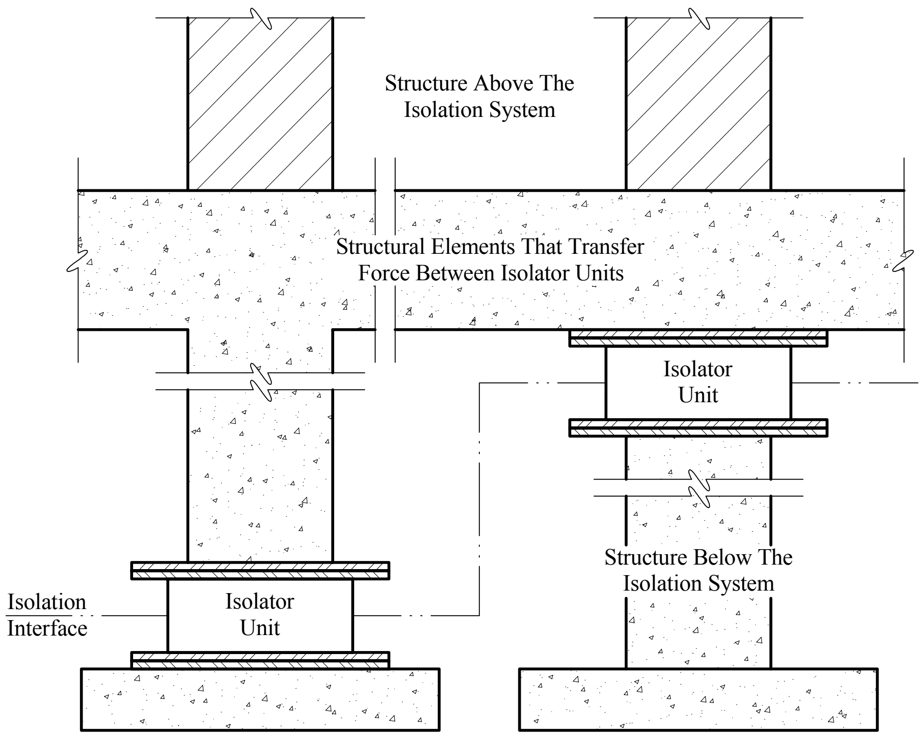


Figure 11.1-1 Isolation system terminology.

The isolation interface is an imaginary boundary between the upper portion of the structure, which is isolated, and the lower portion of the structure, which is assumed to move rigidly with the ground. Typically, the isolation interface is a horizontal plane, but it may be staggered in elevation in certain applications. The isolation interface is important for design of nonstructural components, including components of electrical and mechanical systems that cross the interface and must accommodate large relative displacements.

The wind-restraint system is typically an integral part of isolator units. Elastomeric isolator units are very stiff at very low strains and usually satisfy drift criteria for wind loads, and the static (breakaway) friction force of sliding isolator units is usually greater than the wind force.

11.1.3 Design Approach

The design of isolated structures using the *Provisions* (like the *UBC* and SEAOC's *Blue Book*) has two objectives: achieving life safety in a major earthquake and limiting damage due to ground shaking. To meet the first performance objective, the isolation system must be stable and capable of sustaining forces and displacements associated with the maximum considered earthquake and the structure above the isolation system must remain essentially elastic when subjected to the design earthquake. Limited ductility demand is considered necessary for proper functioning of the isolation system. If significant inelastic response was permitted in the structure above the isolation system, unacceptably large drifts could result due to the nature of long-period vibration. Limiting ductility demand on the superstructure has the additional benefit of meeting the second performance objective of damage control.

The *Provisions* addresses the performance objectives by requiring:

1. Design of the superstructure for forces associated with the design earthquake, reduced by only a fraction of the factor permitted for design of conventional, fixed-base buildings (i.e., $R_I = 3/8\ R \leq 2.0$).

2. Design of the isolation system and elements of the structure below the isolation system (e.g., foundation) for unreduced design earthquake forces.

3. Design and prototype testing of isolator units for forces (including effects of overturning) and displacements associated with the maximum considered earthquake.

4. Provision of sufficient separation between the isolated structure and surrounding retaining walls and other fixed obstructions to allow unrestricted movement during the maximum considered earthquake.

11.1.4 Effective Stiffness and Effective Damping

The *Provisions* utilizes the concepts of effective stiffness and damping to define key parameters of inherently nonlinear, inelastic isolation systems in terms of amplitude-dependent linear properties. Effective stiffness is the secant stiffness of the isolation system at the amplitude of interest. Effective damping is the amount of equivalent viscous damping described by the hysteresis loop at the amplitude of interest. Figure 11.1-2 shows the application of these concepts to both hysteretic isolator units (e.g., friction or yielding devices) and viscous isolator units and shows the *Provisions* equations used to determine effective stiffness and damping from tests of prototypes. Ideally, the effective damping of velocity-dependent devices (including viscous isolator units) should be based on the area of hysteresis loops measured during cyclic testing of the isolation system at full-scale earthquake velocities. Tests of prototypes are usually performed at lower velocities (due to test facility limitations), resulting in hysteresis loops with less area, which produce lower (conservative) estimates of effective damping.

11.2 CRITERIA SELECTION

As specified in the *Provisions* the design of isolated structures must be based on the results of the equivalent lateral force (ELF) procedure, response spectrum analysis, or (nonlinear) time history analysis. Because isolation systems are typically nonlinear, linear methods (ELF procedure and response spectrum analysis) use effective stiffness and damping properties to model nonlinear isolation system components.

The ELF procedure is intended primarily to prescribe minimum design criteria and may be used for design of a very limited class of isolated structures (without confirmatory dynamic analyses). The simple equations of the ELF procedure are useful tools for preliminary design and provide a means of expeditious review and checking of more complex calculations. The *Provisions* also uses these equations to establish lower-bound limits on results of dynamic analysis that may be used for design. Table 11.2-1 summarizes site conditions and structure configuration criteria that influence the selection of an acceptable method of analysis for designing of isolated structures. Where none of the conditions in Table 11.2-1 applies, all three methods are permitted.

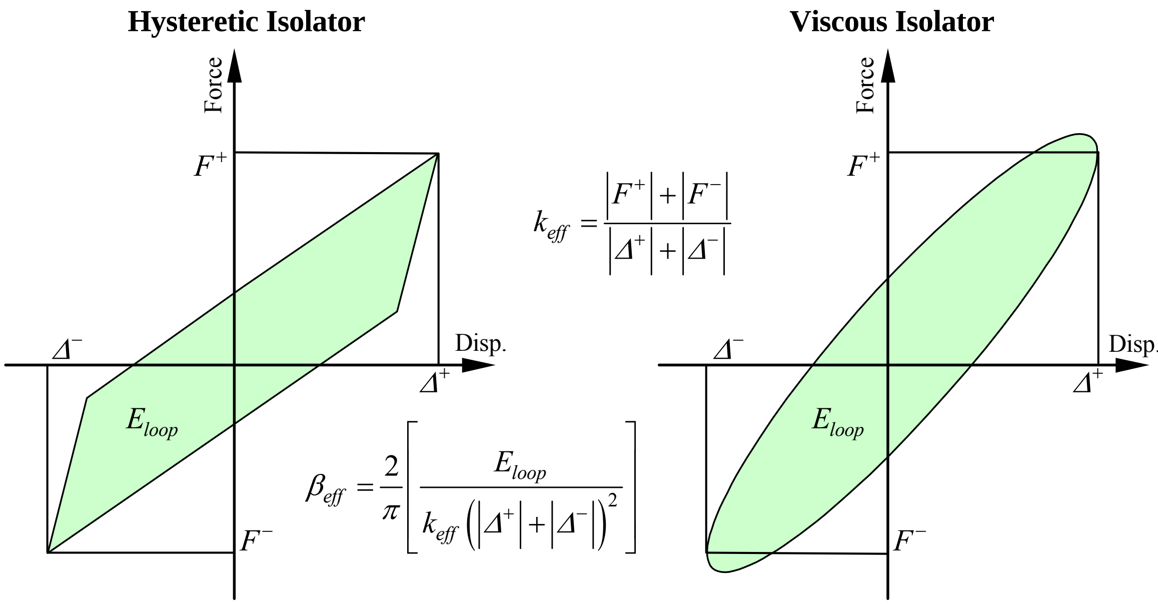


Figure 11.1-2 Effective stiffness and effective damping.

Table 11.2-1 Acceptable Methods of Analysis*

Site condition or Structure Configuration Criteria	ELF Procedure	Response Spectrum Analysis	Time History Analysis
	Site Conditions		
Near-source ($S_1 > 0.6$)	NP	P	P
Soft soil (Site Class E or F)	NP	NP	P
	Superstructure Configuration		
Flexible or irregular superstructure (height > 4 stories, height > 65 ft, or $T_M > 3.0$ sec., or $T_D \leq 3T$)	NP	P	P
Nonlinear superstructure (requiring explicit modeling of nonlinear elements; *Provisions* Sec. 13.2.5.3.1) [13.4.1.2]	NP	NP	P
	Isolation System Configuration		
Highly nonlinear isolation system or system that otherwise does not meet the criteria of *Provisions* Sec. 13.2.5.2, Item 7 [13.2.4.1, Item 7]	NP	NP	P

* P indicates permitted and NP indicates not permitted by the *Provisions*.

Seismic criteria are based on the same site and seismic coefficients as conventional, fixed-base structures (e.g., mapped value of S_1 as defined in *Provisions* Chapter 4 [3]). Additionally, site-specific design criteria are required for isolated structures located on soft soil (Site Class E of F) or near an active source such that S_1 is greater than 0.6, or when nonlinear time history analysis is used for design.

11.3 EQUIVALENT LATERAL FORCE PROCEDURE

The equivalent lateral force (ELF) procedure is a displacement-based method that uses simple equations to determine isolated structure response. The equations are based on ground shaking defined by 1 second spectral acceleration and the assumption that the shape of the design response spectrum at long periods is inversely proportional to period as shown in *Provisions* Figure 4.1.2.6 [3.3-15]. [In the 2003 edition of the *Provisions* , there is also a $1/T^2$ portion of the spectrum at periods greater than T_L. However, in most parts of the Unites States T_L is longer than the period of typical isolated structures.] Although the ELF procedure is considered a linear method of analysis, the equations incorporate amplitude-dependent values of effective stiffness and damping to implicitly account for the nonlinear properties of the isolation system. The equations are consistent with the nonlinear static procedure of FEMA 273 assuming the superstructure is rigid and lateral displacements to occur primarily in the isolation system.

11.3.1 Isolation System Displacement

The isolation system displacement for the design earthquake is determined by using *Provisions* Eq. 13.3.3.1 [13.3-1]:

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D}$$

where the damping factor ,B_D, is based on effective damping, β_D, using *Provisions* Table 13.3.3.1 [13.3-1]. This equation describes the peak (spectral) displacement of a single-degree-of-freedom (SDOF) system with period, T_D, and damping, β_D, for the design earthquake spectrum defined by the seismic coefficient, S_{D1}. S_{D1} corresponds to 5 percent damped spectral response at a period of 1 second. B_D, converts 5 percent damped response to the level of damping of the isolation system. B_D is 1.0 when effective damping, β_D, is 5 percent of critical. Figure 11.3-1 illustrates the underlying concepts of *Provisions* Eq. 13.3.3.1 [13.3-1] and the amplitude-dependent equations of the *Provisions* for effective period, T_D, and effective damping, β_D.

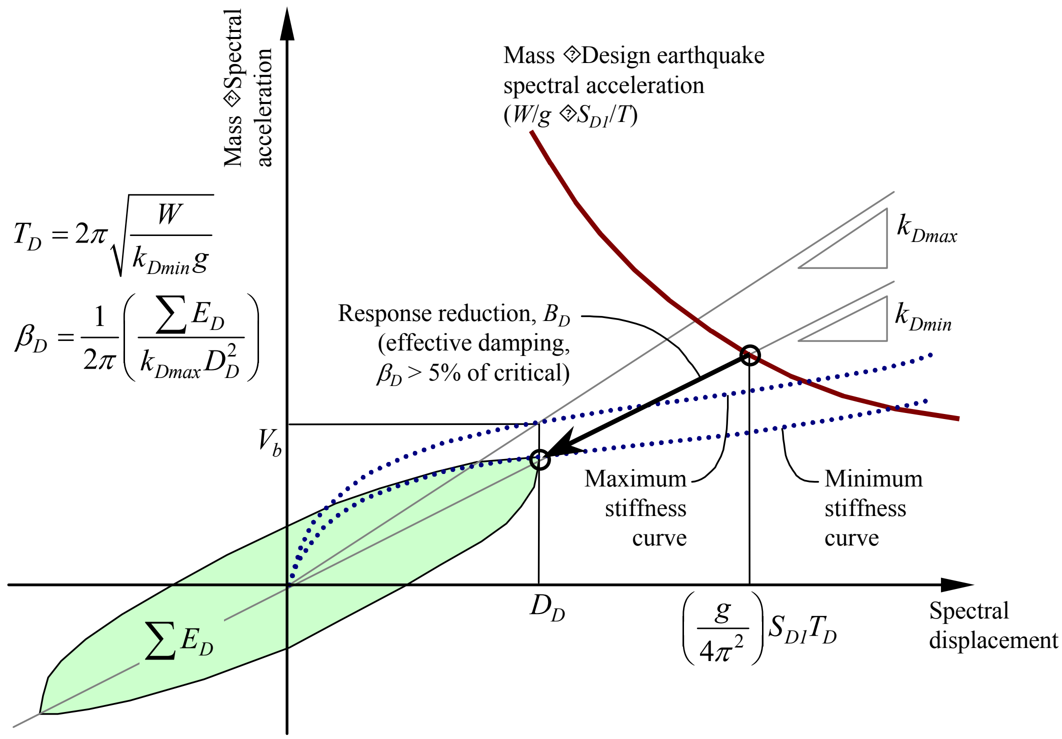


Figure 11.3-1 Isolation system capacity and earthquake demand.

The equations for maximum displacement, D_M, and design displacement, D_D, reflect differences due to the corresponding levels of ground shaking. The maximum displacement is associated with the maximum considered earthquake (characterized by S_{M1}) whereas the design displacement corresponds to the design earthquake (characterized by S_{D1}). In general, the effective period and the damping factor (T_M and B_M, respectively) used to calculate the maximum displacement are different from those used to calculate the design displacement (T_D and B_D) because the effective period tends to shift and effective damping may change with the increase in the level of ground shaking.

As shown in Figure 11.3-1, the calculation of effective period, T_D, is based on the minimum effective stiffness of the isolation system, k_{Dmin}, as determined by prototype testing of individual isolator units. Similarly, the calculation of effective damping is based on the minimum loop area, E_D, as determined by prototype testing. Use of minimum effective stiffness and damping produces larger estimates of effective period and peak displacement of the isolation system.

The design displacement, D_D, and maximum displacement, D_M, represent peak earthquake displacements at the center of mass of the building without the additional displacement, that can occur at other locations due to actual or accidental mass eccentricity. Equations for determining total displacement, including the effects of mass eccentricity as an increase in the displacement at the center of mass, are based on the plan dimensions of the building and the underlying assumption that building mass and isolation stiffness have a similar distribution in plan. The increase in displacement at corners for 5 percent mass eccentricity is about 15 percent if the building is square in plan, and as much as 30 percent if the building is long in plan. Figure 11.3-2 illustrates design displacement, D_D, and maximum displacement, D_M, at the center of mass of the building and total maximum displacement, D_{TM}, at the corners of an isolated building.

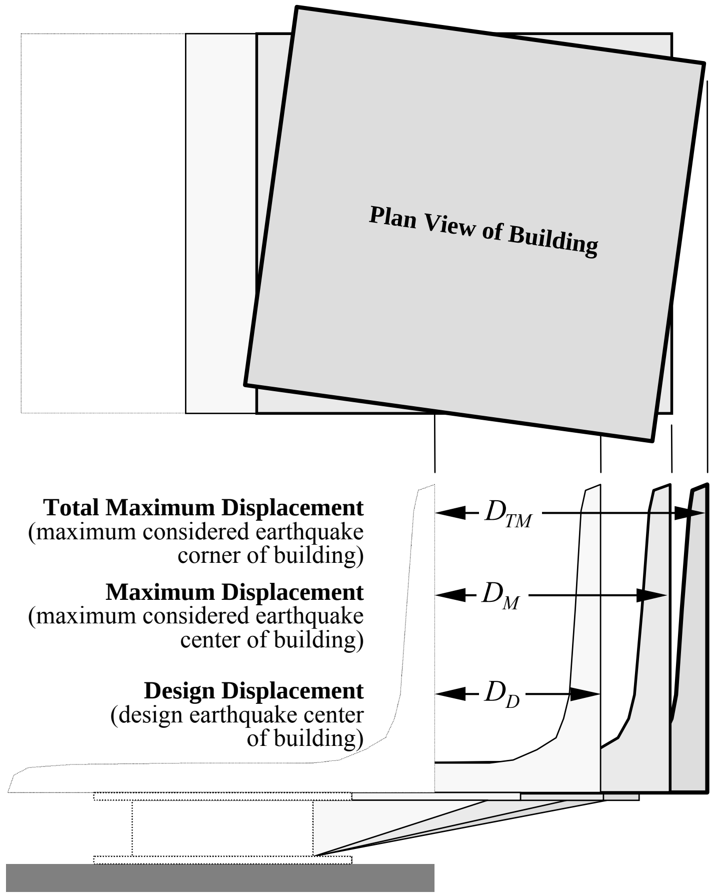


Figure 11.3-2 Design, maximum, and total maximum displacement.

11.3.2 Design Forces

Forces required by the *Provisions* for design of isolated structures are different for design of the superstructure and design of the isolation system and other elements of the structure below the isolation system (e.g., foundation). In both cases, however, use of the maximum effective stiffness of the isolation system is required to determine a conservative value of design force.

In order to provide appropriate overstrength, peak design earthquake response (without reduction) is used directly for design of the isolation system and the structure below. Design for unreduced design earthquake forces is considered sufficient to avoid inelastic response or failure of connections and other elements for ground shaking as strong as that associated with the maximum considered earthquake (i.e., shaking as much as 1.5 times that of the design earthquake). The design earthquake base shear, V_b, is given by *Provisions* Eq. 13.3.4.1 [13.3-7]:

$$V_b = k_{Dmax} D_{D,}$$

where k_{Dmax} is the maximum effective stiffness of the isolation system at the design displacement, D_D. Because the design displacement is conservatively based on minimum effective stiffness, *Provisions* Eq. 13.3.4.1 implicitly induces an additional conservatism of a worst case combination mixing maximum and minimum effective stiffness in the same equation. Rigorous modeling of the isolation system for dynamic analyses precludes mixing of maximum and minimum stiffness in the same analysis (although separate analyses are typically required to determine bounding values of both displacement and force).

Design earthquake response is reduced by a modest factor for design of the superstructure above the isolation interface, as given by *Provisions* Eq. 13.3.4.2 [13.3-8]:

$$V_s = \frac{V_b}{R_I} = \frac{k_{Dmax} D_D}{R_I}$$

The reduction factor, R_I, is defined as three-eighths of the R factor for the seismic-force-resisting system of the superstructure, as specified in *Provisions* Table 5.2.2 [4.3-1], with an upper-bound value of 2.0. A relatively small R_I factor is intended to keep the superstructure essentially elastic for the design earthquake (i.e., keep earthquake forces at or below the true strength of the seismic-force-resisting system). The *Provisions* also impose three limits on design forces that require the value of V_s to be at least as large as each of:

1. The shear force required for design of a conventional, fixed-base structure of period T_D.
2. The shear force required for wind design, and/or
3. A factor of 1.5 times the shear force required for activation of the isolation system.

These limits seldom govern design but reflect principles of good design. In particular, the third limit is included in the *Provisions* to ensure that isolation system displaces significantly before lateral forces reach the strength of the seismic-force-resisting system.

For designs using the ELF procedure, the lateral forces, $F_{x,}$ must be distributed to each story over the height of the structure, assuming an inverted triangular pattern of lateral load (*Provisions* Eq.13.3.5 [13.3-9]):

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i}$$

Because the lateral displacement of the isolated structure is dominated by isolation system displacement, the actual pattern of lateral force in the isolated mode of response is distributed almost uniformly over height. The *Provisions* require an inverted triangular pattern of lateral load to capture possible higher-mode effects that might be missed by not modeling superstructure flexibility. Rigorous modeling of superstructure flexibility for dynamic analysis would directly incorporate higher-mode effects in the results.

Example plots of the design displacement, D_D, total maximum displacement, D_{TM}, and design forces for the isolation system, V_b, and the superstructure, V_s ($R_I = 2$), are shown in Figure 11.3-3 as functions of the effective period of the isolation system. The figure also shows the design base shear required for a conventional building, V ($R/I = 5$). The example plots are for a building assigned to Seismic Design Category D with a one-second spectral acceleration parameter, S_{D1}, equal to 0.6, representing a stiff soil site (Site Class D) located in a region of high seismicity but not close to an active fault. In this example, the isolation system is assumed to have 20 percent effective damping (at all amplitudes of interest) and building geometry is assumed to require 25 percent additional displacement (at corners/edges) due to the requisite 5 percent accidental eccentricity.

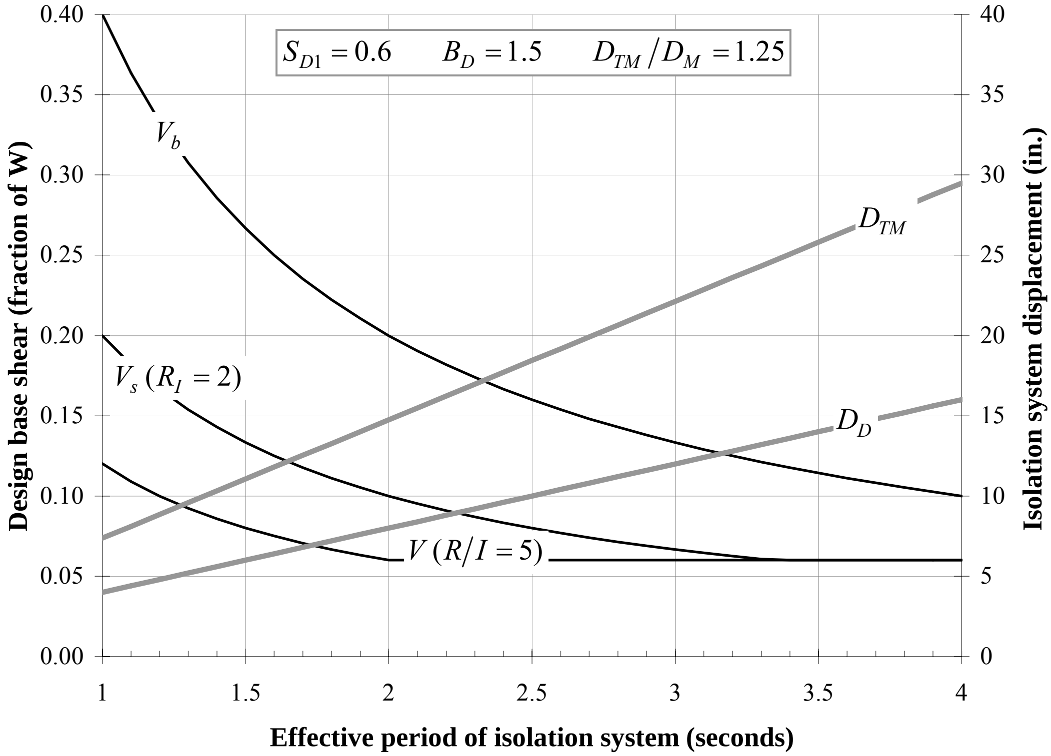


Figure 11.3-3 Isolation system displacement and shear force as function of period (1.0 in. = 25.4 mm).

The plots in Figure 11.3-3 illustrate the fundamental trade off between displacement and force as a function of isolation system displacement. As the period is increased, design forces decrease and design displacements increase linearly. Plots like those shown in Figure 11.3-3 can be constructed during conceptual design once site seismicity and soil conditions are known (or are assumed) to investigate trial values of effective stiffness and damping of the isolation system. In this particular example, an isolation system with an effective period falling between 2.5 and 3.0 seconds would not require more than 22 in. of total maximum displacement capacity (assuming $T_M \leq 3.0$ seconds). Design force on the superstructure would be less than about eight percent of the building weight (assuming $T_D \geq 2.5$ seconds and $R_I \geq 2.0$).

11.4 DYNAMIC LATERAL RESPONSE PROCEDURE

While the ELF procedure equations are useful tools for preliminary design of the isolations system, the *Provisions* requires a dynamic analysis for most isolated structures. Even where not strictly required by the *Provisions*, the use of dynamic analysis (usually time history analysis) to verify the design is common.

11.4.1 Minimum Design Criteria

The *Provisions* encourages the use of dynamic analysis but recognize that along with the benefits of more complex models and analyses also comes an increased chance of design error. To avoid possible under design, the *Provisions* establishes lower-bound limits on results of dynamic analysis used for design. The limits distinguish between response spectrum analysis (a linear, dynamic method) and time history analysis (a nonlinear, dynamic method). In all cases, the lower-bound limit on dynamic analysis is established as a percentage of the corresponding design parameter calculated using the ELF procedure equations. Table 11.4-1 summarizes the percentages that define lower-bound limits on dynamic analysis.

Table 11.4-1 Summary of Minimum Design Criteria for Dynamic Analysis

Design Parameter	Response Spectrum Analysis	Time History Analysis
Total design displacement, D_{TD}	90% D_{TD}	90% D_{TD}
Total maximum displacement, D_{TM}	80% D_{TM}	80% D_{TM}
Design force on isolation system, V_b	90% V_b	90% V_b
Design force on irregular superstructure, V_s	100% V_s	80% V_s
Design force on regular superstructure, V_s	80% V_s	60% V_s

The *Provisions* permits more liberal drift limits when the design of the superstructure is based on dynamic analysis. The ELF procedure drift limits of $0.010h_{sx}$ are increased to $0.015h_{sx}$ for response spectrum analysis and to $0.020h_{sx}$ for time history analysis (where h_{sx} is the story height at level x). Usually a stiff system (e.g., braced frames) is selected for the superstructure and drift demand is typically less than about $0.005h_{sx}$. *Provisions* Sec. 13.4.7.4 [13.4.4] requires an explicit check of superstructure stability at the maximum considered earthquake displacement if the design earthquake story drift ratio exceeds $0.010/R_I$.

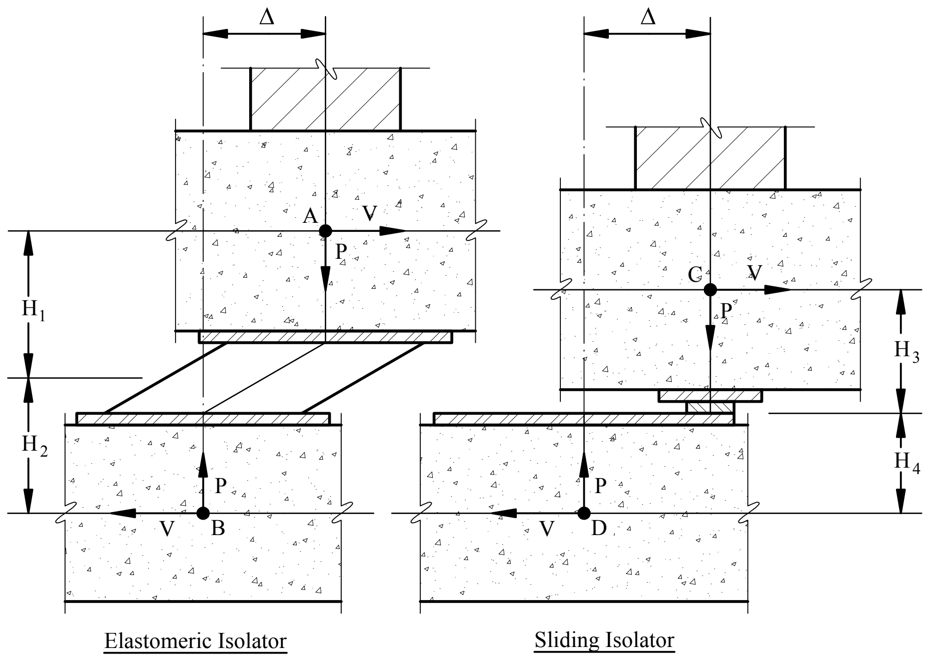
11.4.2 Modeling Requirements

As for the ELF procedure, the *Provisions* requires the isolation system to be modeled for dynamic analysis using stiffness and damping properties that are based on tests of prototype isolator units. Additionally, dynamic analysis models are required to account for:

1. Spatial distribution of individual isolator units,
2. Effects of actual and accidental mass eccentricity,
3. Overturning forces and uplift of individual isolator units, and
4. Variability of isolation system properties (due to rate of loading, etc.).

The *Provisions* requires explicit nonlinear modeling of elements if time history analysis is used to justify design loads less than those permitted for ELF or response spectrum analysis. This option is seldom exercised and the superstructure is typically modeled using linear elements and conventional methods. Special modeling concerns for isolated structures include two important and related issues: the uplift of isolator units, and the P-delta effects on the isolated structure. Isolator units tend to have little or no ability to resist tension forces and can uplift when earthquake overturning (upward) loads exceed factored gravity (downward) loads. Local uplift of individual elements is permitted (*Provisions* Sec. 13.6.2.7 [13.2.5.7]) provided the resulting deflections do not cause overstress or instability. To calculate uplift effects, gap elements may be used in nonlinear models or tension may be released manually in linear models.

The effects of P-delta loads on the isolation system and adjacent elements of the structure can be quite significant. The compression load, P, can be large due to earthquake overturning (and factored gravity loads) at the same time that large displacements occur in the isolation system. Computer analysis programs (most of which are based on small-deflection theory) may not correctly calculate P-delta moments at the isolator level in the structure above or in the foundation below. Figure 11.4-1 illustrates moments due to P-delta effects (and horizontal shear loads) for an elastomeric bearing isolation system and a sliding isolation system. For the elastomeric system, the P-delta moment is split one-half up and one-half down. For the sliding system, the full P-delta moment is applied to the foundation below (due to the orientation of the sliding surface). A reverse (upside down) orientation would apply the full P-delta moment on the structure above.



$$M_A = \frac{P\Delta}{2} + VH_1$$
$$M_B = \frac{P\Delta}{2} + VH_2$$
$$M_C = VH_3$$
$$M_D = P\Delta + VH_4$$

Figure 11.4-1 Moments due to horizontal shear and P-delta effects.

For time history analysis, nonlinear force-deflection characteristics of isolator units are explicitly modeled (rather than using effective stiffness and damping). Force-deflection properties of isolator units are typically approximated by a bilinear, hysteretic curve whose properties can be accommodated by commercially available nonlinear structural analysis programs. Such bilinear hysteretic curves should have approximately the same effective stiffness and damping at amplitudes of interest as the true force-deflection characteristics of isolator units (as determined by prototype testing).

Figure 11.4-2 shows a bilinear idealization of the response of a typical nonlinear isolator unit. Figure 11.4-2 also includes simple equations defining the yield point (D_y, F_y) and end point (D, F) of a bilinear approximation that has the same effective stiffness and damping as the true curve (at a displacement, D).

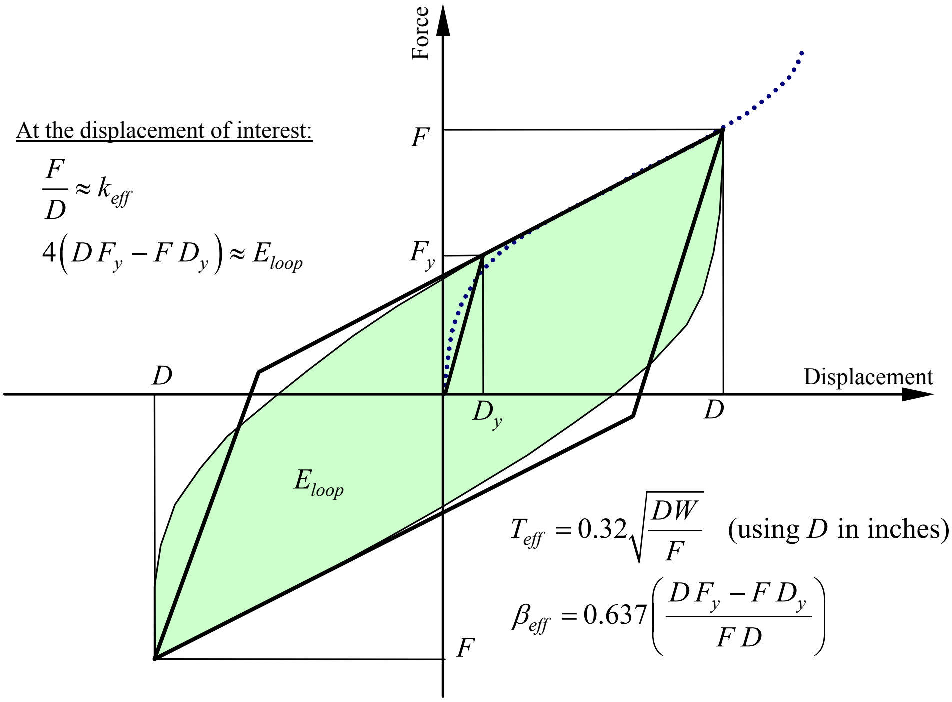


Figure 11.4-2 Bilinear idealization of isolator unit behavior.

11.4.3 Response Spectrum Analysis

Response spectrum analysis requires that isolator units be modeled using amplitude-dependent values of effective stiffness and damping that are the same as those for the ELF procedure. The effective damping of the isolated modes of response is limited to 30 percent of critical. Higher modes of response are usually assumed to have five percent damping—a value of damping appropriate for the superstructure, which remains essentially elastic. As previously noted, maximum and minimum values of effective stiffness are typically used to individually capture maximum displacement of the isolation system and maximum forces in the superstructure. Horizontal loads are applied in the two orthogonal directions, and peak response of the isolation system and other structural elements is determined using the 100 percent plus 30 percent combination method.

11.4.4 Time History Analysis

Time history analysis with explicit modeling of nonlinear isolator units is commonly used for the evaluation of isolated structures. Where at least seven pairs of time history components are employed, the values used in design for each response parameter of interest may be the average of the corresponding analysis maxima. Where fewer pairs are used (with three pairs of time history components being the minimum number permitted), the maximum value of each parameter of interest must be used for design.

The time history method is not a particularly useful design tool due to the complexity of results, the number of analyses required (e.g., to account for different locations of eccentric mass), the need to combine different types of response at each point in time, etc. It should be noted that while *Provisions* Chapter 5 does not require consideration of accidental torsion for either the linear or nonlinear response history procedures, Chapter 13 does require explicit consideration of accidental torsion, regardless of the analysis method employed. Time history analysis is most useful when used to verify a design by checking a few key design parameters, such as: isolation displacement, overturning loads and uplift, and story shear force.

11.5 EMERGENCY OPERATIONS CENTER USING ELASTOMERIC BEARINGS, SAN FRANCISCO, CALIFORNIA

This example features the seismic isolation of a hypothetical emergency operations center (EOC), located in the center of San Francisco, California, an area of very high seismicity. Using high-damping rubber bearings, other types of isolators could be designed to have comparable response properties. Isolation is an appropriate design strategy for EOCs and other buildings where the goal is to limit earthquake damage and protect facility function. The example illustrates the following design topics:

1. Determination of seismic design parameters,
2. Preliminary design of superstructure and isolation systems (using the ELF procedure),
3. Dynamic analysis of seismically isolated structures, and
4. Specification of isolation system design and testing criteria.

While the example includes development of the entire structural system, the primary focus is on the design and analysis of the isolation system. Examples in other chapters may be referred to for more in-depth descriptions of the provisions governing detailed design of the superstructure (i.e., the structure above the isolation system) and the foundation.

11.5.1 System Description

This EOC is a three-story, steel-braced frame structure with a large, centrally located mechanical penthouse. Story heights of 15 ft at all floors accommodate computer access floors and other architectural and mechanical systems. The roof and penthouse roof decks are designed for significant live load to accommodate a helicopter-landing pad and meet other functional requirements of the EOC. Figure 11.5-1 shows the three-dimensional model of the structural system.

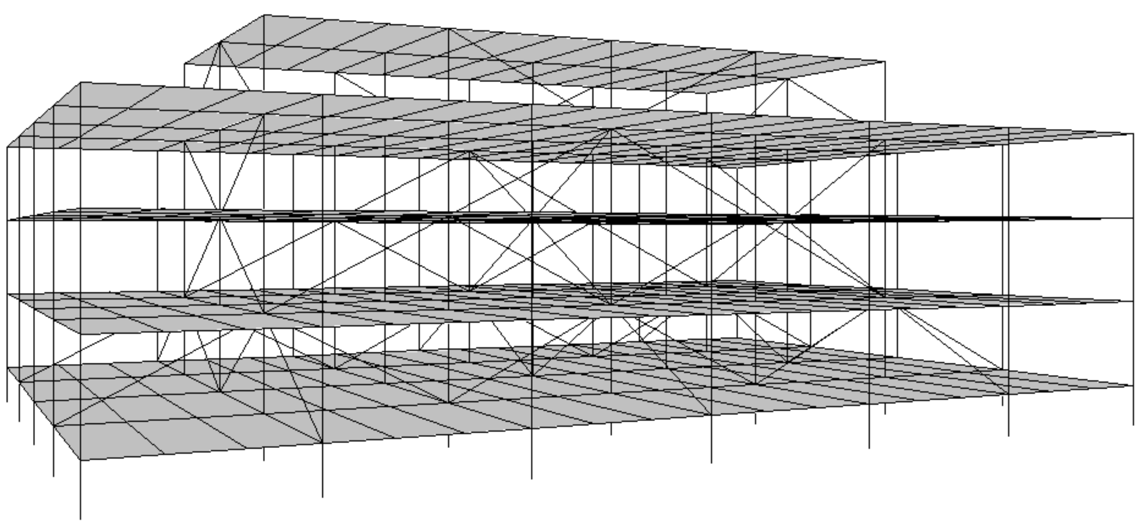


Figure 11.5-1 Three-dimensional model of the structural system.

The structure (which is regular in configuration) has plan dimensions of 120 ft. by 180 ft. at all floors except for the penthouse, which is approximately 60 ft by 120 ft in plan. Columns are spaced at 30 ft in both directions. This EOC's relatively large column spacing (bay size) is used to reduce the number of isolator units for design economy and to increase gravity loads on isolator units for improved earthquake performance. Figures 11.5-2 and 11.5-3, are framing plans for the typical floor levels (1, 2, 3, and roof) and the penthouse roof.

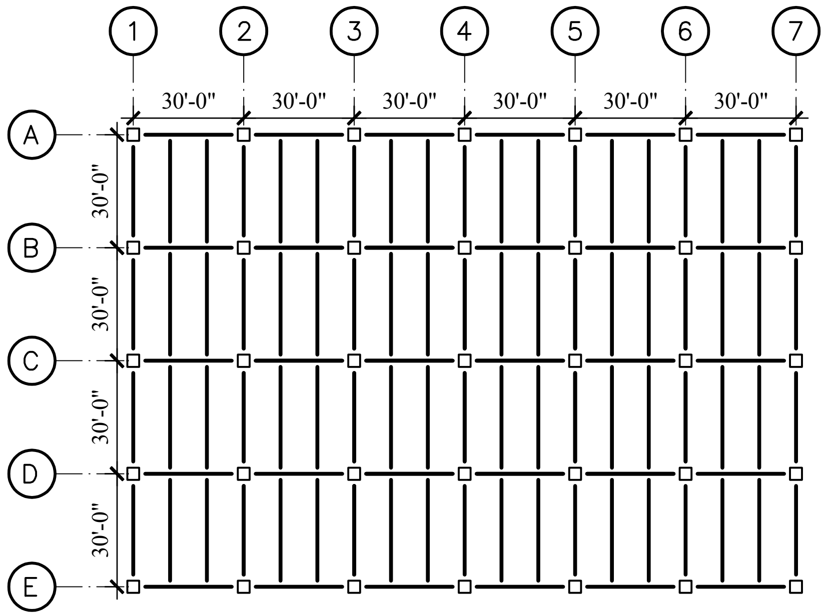


Figure 11.5-2 Typical floor framing plan (1.0 ft = 0.3048 m).

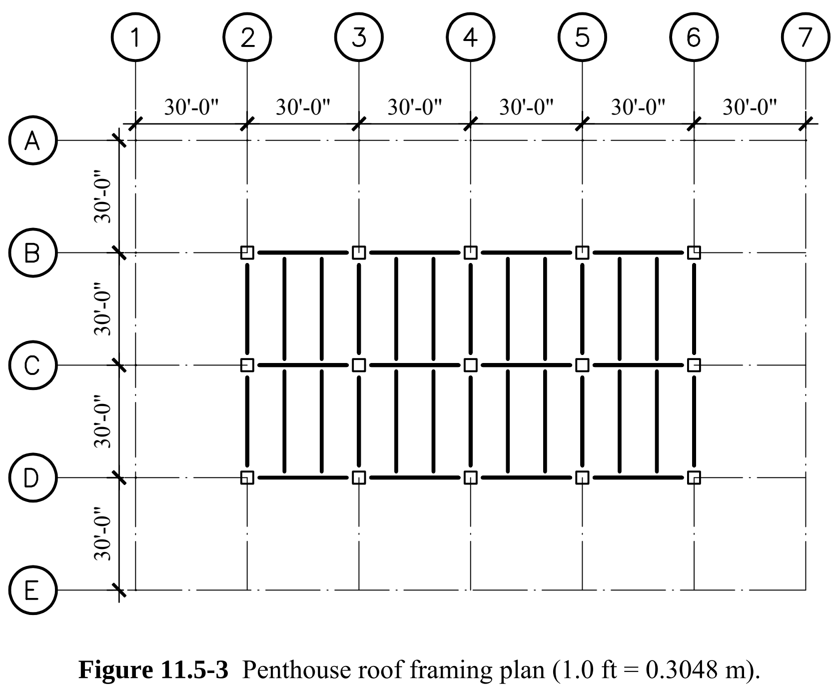


Figure 11.5-3 Penthouse roof framing plan (1.0 ft = 0.3048 m).

The vertical load carrying system consists of concrete fill on steel deck floors, roofs supported by steel beams at 10 ft on center, and steel girders at column lines. Isolator units support the columns below the first floor. The foundation is a heavy mat (although the spread footings or piles could be used depending on the soil type, depth to the water table, and other site conditions).

The lateral system consists of a symmetrical pattern of concentrically braced frames. These frames are located on Column Lines B and D in the longitudinal direction, and on Column Lines 2, 4 and 6 in the transverse direction. Figures 11.5-4 and 11.5-5, respectively, show the longitudinal and transverse elevations. Braces are specifically configured to reduce the concentration of earthquake overturning, and uplift loads on isolator units by:

1. Increasing the continuous length of (number of) braced bays at lower stories,
2. Locating braces at interior (rather than perimeter) column lines (more hold-down weight), and
3. Avoiding common end columns for transverse and longitudinal bays with braces.

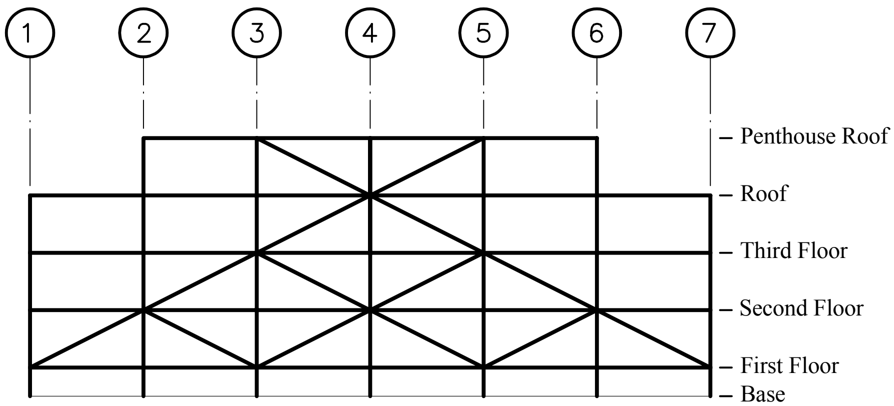


Figure 11.5-4 Longitudinal bracing elevation (Column Lines B and D).

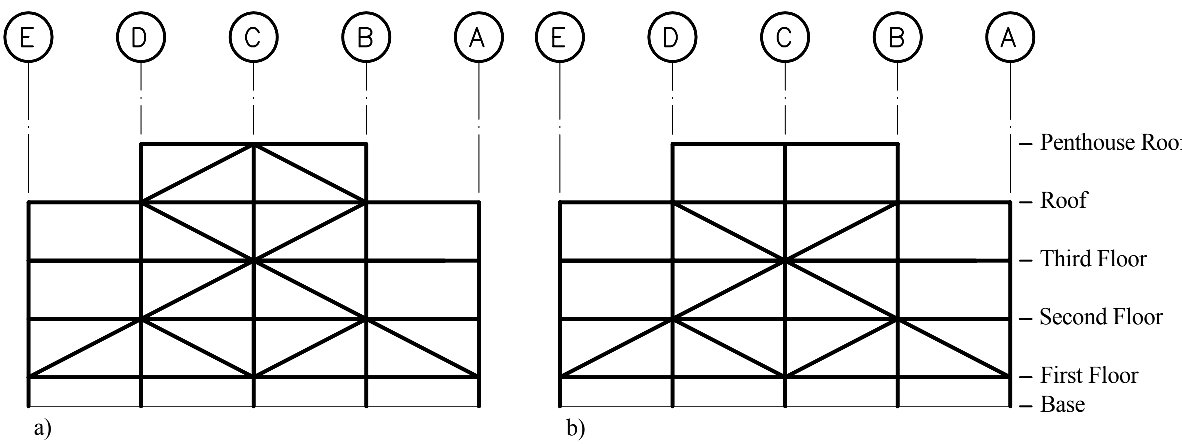


Figure 11.5-5 Transverse bracing elevations: (a) on Column Lines 2 and 6 and (b) on Column Line 4.

The isolation system is composed of 35 identical elastomeric isolator units, located below columns. The first floor is just above grade, and the isolator units are approximately 3 ft below grade to provide clearance below the first floor, for construction and maintenance personnel. A short retaining wall borders the perimeter of the facility and provides 30 in. of "moat" clearance for lateral displacement of the isolated structure. Access to the EOC is provided at the entrances by segments of the first floor slab, which cantilever over the moat.

Girders at the first-floor column lines are much heavier than the girders at other floor levels, and have moment-resisting connections to columns. These girders stabilize the isolator units by resisting moments due to vertical (P-delta effect) and horizontal (shear) loads. Column extensions from the first floor to the top plates of the isolator units are stiffened in both horizontal directions, to resist these moments and to serve as stabilizing haunches for the beam-column moment connections.

11.5.2 Basic Requirements

11.5.2.1 Specifications

General: 1997 *Uniform Building Code* (*UBC*)

Seismic: *NEHRP Recommended Provisions*

11.5.2.2 Materials

Concrete:

Strength	$f_c' =$ 3 ksi
Weight (normal)	γ =150 pcf

Steel:

Columns	F_y = 50 ksi
Primary first-floor girders (at column lines)	F_y = 50 ksi
Other girders and floor beams	F_y = 36 ksi
Braces	F_y = 46 ksi

Steel deck: 3-in.-deep, 20-gauge deck

Seismic isolator units (high-damping rubber):

Maximum long-term-load (1.2D + 1.6L) face pressure, σ_{LT}	1,400 psi
Maximum short-term-load (1.5D + 1.0L + Q_{MCE}) face pressure, σ_{ST}	2,800 psi
Minimum bearing diameter (excluding protective cover)	1.25D_{TM}
Minimum rubber shear strain capacity (isolator unit), γ_{max}	300 percent
Minimum effective horizontal shear modulus, G_{min} Third cycle at γ = 150 percent (after scragging/recovery)	65 - 110 psi
Maximum effective horizontal shear modulus, G_{max} First cycle at γ = 150 percent (after scragging/recovery)	1.3 × G_{min}
Minimum effective damping at 150 percent rubber shear strain, β_{eff}	15 percent

11.5.2.3 Gravity Loads

Dead loads:

Main structural elements (slab, deck, and framing)	self weight

Miscellaneous structural elements (and slab allowance)	10 psf
Architectural facades (all exterior walls)	750 plf
Roof parapets	150 plf
Partitions (all enclosed areas)	20 psf
Suspended MEP/ceiling systems and supported flooring	15 psf
Mechanical equipment (penthouse floor)	50 psf
Roofing	10 psf

Reducible live loads:

Floors (1-3)	100 psf
Roof decks and penthouse floor	50 psf

Live load reduction:

The 1997 *UBC* permits area-based live load reduction, of not more than 40 percent for elements with live loads from a single story (e.g., girders), and not more than 60 percent for elements with live loads from multiple stories (e.g., axial component of live load on columns at lower levels and isolator units).

EOC weight (dead load) and live load:

Penthouse roof	W_{PR}	= 965 kips
Roof (penthouse)	W_R	= 3,500 kips
Third floor	W_3	= 3,400 kips
Second floor	W_2	= 3,425 kips
First floor	W_1	= 3,425 kips
Total EOC weight	W	= 14,715 kips

(See Chapter 1 for a discussion of live load contributions to the seismic weight.)

Live load (L) above isolation system	L = 7,954 kips
Reduced live load (RL) above isolation system	RL = 3,977 kips

Table 11.5-1 Gravity Loads on Isolator Units*

Column line	Dead/live loads (kips)			
	1	2	3	4
A	182/73	349/175	303/153	345/173
B	336/166	570/329	606/346	539/309
C	295/149	520/307	621/356	639/358

1.0 kip = 4.45 kN.
* Loads at Column Lines 5, 6 and 7 (not shown) are the same as those at Column Lines 3, 2, and 1, respectively; loads at Column Lines D and E (not shown) are the same as those at Column Lines B and A, respectively.

11.5.2.4 *Provisions* Parameters

11.5.2.4.1 Performance Criteria (Provisions Sec. 1.3 [1.2 and 1.3])

Designated Emergency Operation Center	Seismic Use Group III
Occupancy Importance Factor	I = 1.5 (conventional)
Occupancy Importance Factor (*Provisions* Chapter 13)	I = 1.0 (isolated)

Chapter 13 does not require use of the occupancy importance factor to determine the design loads on the structural system of an isolated building (that is, I = 1.0), but the component importance factor is required by Chapter 6, to determine seismic forces on components (I_p = 1.5 for some facilities).

11.5.2.4.2 Ground Motion (Provisions Chapter 4 [3])

Site soil type (assumed)	Site Class D
Maximum considered earthquake (MCE) spectral acceleration at short periods (*Provisions* Map 7)	S_S = 1.50

[The 2003 Provisions have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package.]

Site coefficient (*Provisions* Table 4.1.2.4b [3.3-1])	F_a = 1.0
MCE spectral acceleration adjusted for site class (F_aS_S)	S_{MS} = 1.50
Design earthquake (DE) spectral acceleration ($2/3S_{MS}$)	S_{DS} = 1.0
MCE spectral acceleration at a period of 1 second (*Provisions* Map 8)	S_1 = 0.9

On Map 8, the contour line of S_1 = 0.9 runs approximately through the center of the San Francisco peninsula, with other San Francisco peninsula contour lines ranging from 0.6 (greatest distance from San Andreas Fault), to about 1.6 (directly on top of the San Andreas Fault).

Site coefficient (*Provisions* Table 4.1.2.4b [3.3-2])	F_v = 1.5
MCE spectral acceleration adjusted for site class (F_vS_1)	S_{M1} = 1.35
Design earthquake (DE) spectral acceleration ($2/3S_{M1}$)	S_{D1} = 0.9
Seismic Design Category (*Provisions* Table 4.2.1b [1.4-2])	Seismic Design Category F

[In the 2003 edition of the *Provisions*, the ground motion trigger for Seismic Design Categories E and F have been changed to $S_1 \geq 0.60$. No change would result for this example.]

11.5.2.4.3 Design Spectra (Provisions Sec. 13.4.4.1)

Figure 11.5-6 plots design earthquake and maximum considered earthquake response spectra as constructed in accordance with the procedure of *Provisions* Sec. 13.4.4.1 [3.4] using the spectrum shape defined by *Provisions* Figure 4.1.2.6 [3.3-15]. [In the 2003 edition of *Provisions*, the shape of the design spectrum changes at period beyond T_L, but no change would result for this example.] *Provisions* Sec. 13.4.4.1[13.2.3.1] requires site-specific design spectra to be calculated for sites with S_1 greater than 0.6 (e.g., sites near active sources). Site-specific design spectra may be taken as less than 100 percent but not less than 80 percent of the default design spectra of *Provisions* Figure 4.1.2.6 [3.3-15].

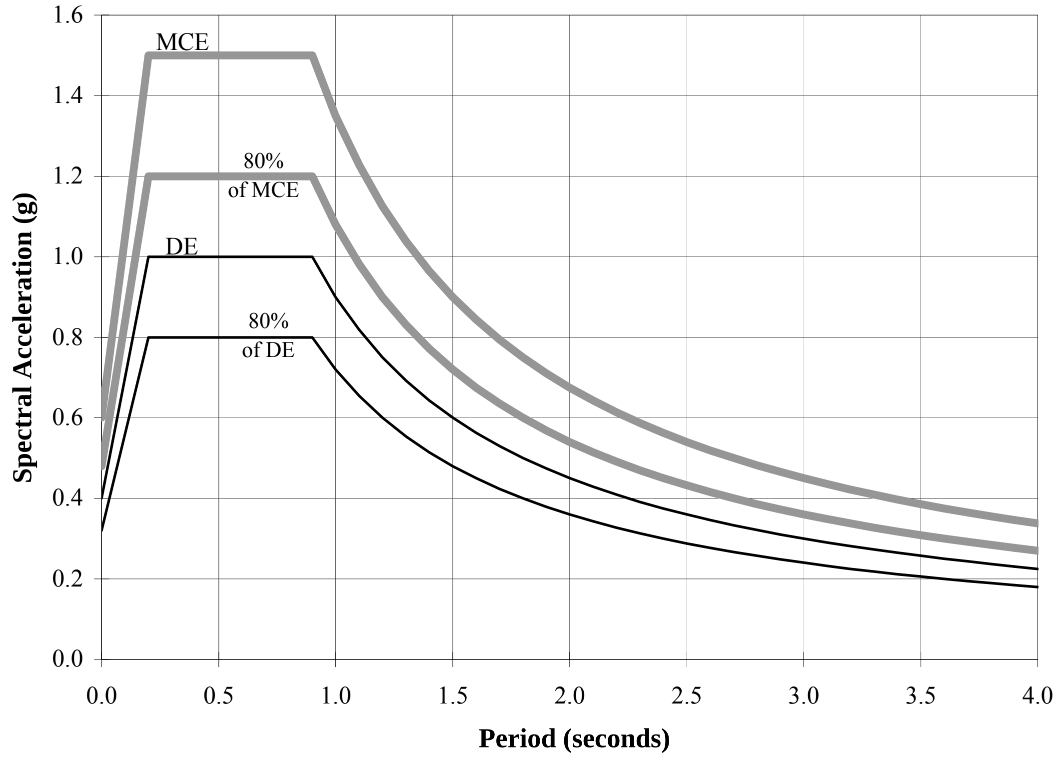


Figure 11.5-6 Example design spectra (5 percent damping).

For this example, site-specific spectra for the design earthquake and the maximum considered earthquake, were assumed to be 80 percent (at long periods) and 100 percent (at short periods) of the respective spectra shown in Figure 11.5-6. The 80 percent factor reflects a reduction in demand that could be achieved through a detailed geotechnical investigation of site soil conditions. In general, site-specific spectra for regions of high seismicity, with well defined fault systems (like those of the San Francisco Bay Area), would be expected to be similar to the default design spectra of *Provisions* Figure 4.1.2.6 [3.3-15].

11.5.2.4.4 Design Time Histories (Provisions Sec. 13.4.4.2 [13.2.3.2])

For time history analysis, *Provisions* Sec. 13.4.4.7 [13.4.2.3] requires no less than three pairs of horizontal ground motion time history components to be selected from actual earthquake records, and scaled to match either the design earthquake (DE) or the maximum considered earthquake (MCE) spectrum. The selection and scaling of appropriate time histories is usually done by an earth scientist, or a geotechnical engineer experienced in the seismology of the region; and takes the earthquake magnitudes, fault distances, and source mechanisms that influence hazards at the building site into consideration.

For this example, records from the El Centro Array Station No. 6, the Newhall Fire Station, and the Sylmar Hospital are selected from those recommended by *ATC-40* [ATC, 1996], for analysis of buildings at stiff soil sites with ground shaking of 0.2g or greater. These records, and the scaling factors required to match the design spectra are summarized in Table 11.5-2. These three records represent strong ground shaking recorded relatively close to fault rupture; and contain long-period pulses in the direction of strongest shaking, which can cause large displacement of the isolated structures. MCE scaling factors for the El Centro No. 6 and Sylmar records are 1.0, indicating that these records are used without modification for MCE time history analysis of the EOC.

Provisions Sec. 13.4.4.2 [13.2.3.2] provides criteria for scaling earthquake records to match a target spectrum over the period range of interest, defined as $0.5T_D$ to $1.25T_M$. In this example, T_D and T_M are both assumed to be 2.5 seconds, so the period range of interest is from 1.25 seconds to 3.125 seconds. For each period in this range, the average of the square-root-of-the-sum-of-the-squares (SRSS) combination of each pair of horizontal components of scaled ground motion, may not fall below the target spectrum by more 10 percent. The target spectrum is defined as 1.3 times the design spectrum of interest (either the DE or the MCE spectrum).

Figure 11.5-7 compares the spectra of scaled time history components with the spectrum for the design earthquake. Rather than comparing the average of the SRSS spectra with 1.3 times the design spectrum, as indicated in the *Provisions*, Figure 11.5-7 shows the (unmodified) design spectrum and the average of the SRSS spectra divided by 1.3. The effect is the same, but the method employed eliminates one level of obscurity and permits direct comparison of the calculated spectra without additional manipulation. A comparison of maximum, considered earthquake spectra would look similar (with values that were 1.5 times larger). The fit is very good at long periods - the period range of interest for isolated structures. At 2.5 seconds, the spectrum of the scaled time histories has the same value as the target spectrum. Between 1.25 seconds and 3.125 seconds, the spectrum of the scaled time histories is never less than the target spectrum by more than 10 percent. At short periods, the spectrum of scaled time history motions are somewhat greater than the target spectrum - a common result of scaling real time histories to match the long-period portion of the design spectrum.

Figure 11.5-7 also includes plots of upper-bound (maximum demand) and lower-bound (minimum demand) envelopes of the spectra of the six individual components of scaled time histories. The maximum demand envelope illustrates that the strongest direction (component) of shaking of at least one of the three scaled time histories is typically about twice the site-specific spectrum in the period range of the isolated structure - consistent with strong ground shaking recorded near sources in the fault normal direction.

Table 11.5-2 Earthquake Time History Records and Scaling Factors

Record No.	Earthquake Source and Recording Station			Scaling factor	
	Year	Earthquake	Station (owner)	DE	MCE
1	1979	Imperial Valley, CA	El Centro Array Station 6 (USGS)	0.67	1.0
2	1994	Northridge, CA	Newhall Fire Station (CDMG)	1.0	1.5
3	1994	Northridge, CA	Sylmar Hospital (CDMG)	0.67	1.0

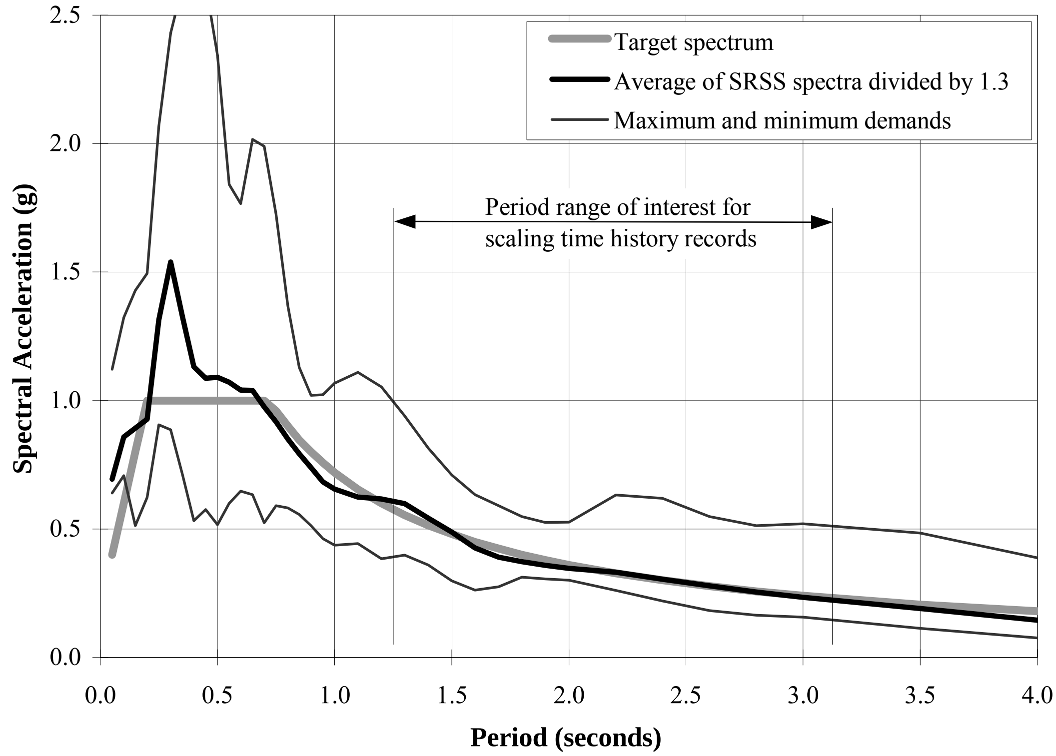


Figure 11.5-7 Comparison of design earthquake spectra.

11.5.2.5 Structural Design Criteria

11.5.2.5.1 Design Basis (Provisions Sec. 5.2 and 13.2 [4.3 and 13.2])

Seismic-force-resisting system.	Special steel concentrically braced frames (height < 100 ft)
Response modification factor, R (*Provisions* Table 5.2.2 [4.3-1]	6 (conventional)
Response modification factor for design of the superstructure, R_I (*Provisions* Sec. 13.3.4.2 [13.3.3.2], $3/8R \leq 2$)	2 (isolated)
Plan irregularity (of superstructure) - (*Provisions* Table 5.2.3.2)	None
Vertical irregularity (of superstructure) - from Table 5.2.3.3	None
Lateral response procedure (Provisions Sec. 13.2.5.2 [13.2.4.1], $S_1 > 0.6$)	Dynamic analysis
Redundancy/reliability factor - (*Provisions* Sec. 5.2.4.2 [4.3.3])	$\rho > 1.0$ (conventional) $\rho = 1.0$ (isolated)

Provisions Sec. 5.2.4.2 [4.3.3] requires the use of a calculated ρ value, which would be greater than 1.0 for a conventional structure with a brace configuration similar to the superstructure of the base-isolated EOC. However, in the author's opinion, the use of R_I equal to 2.0 (rather than R equal to 6) as required by *Provisions* Sec. 13.3.4.2 precludes the need to further increase superstructure design forces for redundancy/reliability. Future editions of the *Provisions* should address this issue.

[The redundancy factor is changed substantially in the 2003 Provisions. However, the rationale set forth above by the author still holds; use of $\rho = 1.0$ is reasonable since $R_I = 2.0$.]

11.5.2.5.2 Horizontal Earthquake Loads and Effects (Provisions Chapters 5 and 13)

Design earthquake (acting in either the X or Y direction)	DE (site specific)
Maximum considered earthquake (acting in either the X or Y direction)	MCE (site specific)
Mass eccentricity - actual plus accidental	$0.05b$ = 6 ft ($\perp$ to X axis) $0.05d$ = 9 ft ($\perp$ to Y axis)

The superstructure is symmetric about both principal axes, however, the placement of the braced frames results in a ratio of maximum corner displacement to average displacement of 1.25, exceeding the threshold of 1.2 per the definition of the *Provisions*. If the building were not on isolators, the accidental torsional moment would need to be increased from 5 percent to 5.4 percent of the building dimension. The input to the superstructure is controlled by the isolation system, and it is the author's opinion that the amplification of accidental torsion is not necessary for such otherwise regular structures. Future editions of the *Provisions* should address this issue. Also refer to the discussion of analytical modeling of accidental eccentricities in *Guide* Chapter 1.

Superstructure design (reduced DE response)	$Q_E = Q_{\mathrm{DE/2}}$ = DE/2.0
Isolation system and foundation design (unreduced DE response)	$Q_E = Q_{\mathrm{DE}}$ = DE/1.0
Check of isolation system stability (unreduced MCE response)	$Q_E = Q_{\mathrm{MCE}}$ = MCE/1.0

11.5.2.5.3 Combination of Horizontal Earthquake Load Effects (Provisions Sec. 5.25.2.3 [4.4.2.3] and 13.4.6.3 [13.4.2.2])

Response due earthquake loading in the X and Y directions	Q_E = Max $(1.0Q_{EX}+0.3Q_{EY},$ $0.3Q_{EX}+1.0Q_{EY})$

In general, the horizontal earthquake load effect, Q_E, on the response parameter of interest is influenced by only one direction of horizontal earthquake load, and $Q_E = Q_{EX}$ or $Q_E = Q_{EY}$. Exceptions include vertical load on isolator units due to earthquake overturning forces.

11.5.2.5.4 Combination of Horizontal and Vertical Earthquake Load Effects (Provisions Sec. 5.2.7 [4.2.2.1])

Design earthquake ($\rho Q_E \pm 0.2S_{DS}D$)	$E = Q_E \pm 0.2D$
Maximum considered earthquake ($\rho Q_E \pm 0.2S_{MS}D$)	$E = Q_E \pm 0.3D$

11.5.2.5.5 Superstructure Design Load Combinations (UBC Sec. 1612.2, using R_I = 2)

Gravity loads (dead load and reduced live load)	$1.2D + 1.6L$
Gravity and earthquake loads ($1.2D + 0.5L + 1.0E$)	$1.4D + 0.5L + Q_{\mathrm{DE/2}}$

Gravity and earthquake loads (0.9D - 1.0E)	0.7D - $Q_{DE/2}$

11.5.2.5.6 Isolation System and Foundation Design Load Combinations (UBC Sec. 1612.2)

Gravity loads (for example, long term load on isolator units)	1.2D + 1.6L
Gravity and earthquake loads (1.2D + 0.5L + 1.0E)	1.4D + 0.5L + Q_{DE}
Gravity and earthquake loads (0.9D - 1.0E)	0.7D - Q_{DE}

11.5.2.5.7 Isolation System Stability Load Combinations (Provisions Sec. 13.6.2.6 [13.2.5.6])

Maximum short term load on isolator units (1.2D + 1.0L + $\|E\|$)	1.5D + 1.0L + Q_{MCE}
Minimum short term load on isolator units (0.8D - $\|E\|$)	0.8D - Q_{MCE}

Note that in the above combinations, the vertical earthquake load (0.2$S_{MS}D$) component of $|E|$ is included in the maximum (downward) load combination, but excluded from the minimum (uplift) load combination. It is the author's opinion that vertical earthquake ground shaking is of a dynamic nature, changing direction too rapidly to affect appreciably uplift of isolator units and need not be used with the load combinations of *Provisions* Sec. 13.6.2.6 [13.2.5.6] for determining minimum (uplift) vertical loads on isolator units due to the MCE. Future editions of the *Provisions* should explicitly address this issue.

11.5.3 Seismic Force Analysis

11.5.3.1 Basic Approach to Modeling

To expedite calculation of loads on isolator units, and other elements of the seismic-force-resisting system, a 3-dimensional model of the EOC is developed and analyzed using the ETABS computer program (CSI, 1999). While there are a number of programs to choose from, ETABS (version 7.0) is selected for this example since it permits the automated release of tension in isolator units subject to uplift and has built-in elements for modeling other nonlinear properties of isolator units. Arguably, all of the analyses performed by the ETABS program could be done by hand, or by spreadsheet calculation (except for confirmatory time history analyses).

The ETABS model is used to perform the following types of analyses and calculations:

1. Dead load weight and live loads for the building--calculate maximum long-term load on isolator units (*Guide* Table 11.5-1)

2. ELF procedure for gravity and reduced design earthquake loads--design superstructure (ignoring uplift of isolator units)

3. Nonlinear static analysis with ELF loads for gravity and unreduced design earthquake loads--design isolation system and foundation (considering uplift of isolator units)

4. Nonlinear static analysis with ELF loads - gravity and unreduced design earthquake loads--calculate maximum short-term load (downward force) on isolator units (*Guide* Table 11.5-5) and calculate minimum short-term load (downward force) of isolator units (*Guide* Table 11.5-6)

5. Nonlinear static analysis with ELF loads for gravity and unreduced MCE loads--calculate maximum short-term load (downward force) on isolator units (*Guide* Table 11.5-7) and calculate minimum short-term load (uplift displacement) of isolator units (*Guide* Table 11.5-8)

6. Nonlinear time history analysis - gravity and scaled DE or MCE time histories--verify DE displacement of isolator units and design story shear (*Guide* Table 11.5-10), verify MCE displacement of isolator units (*Guide* Table 11.5-11), verify MCE short-term load (downward force) on isolator units (*Guide* Table 11.5-12), and verify MCE short-term load (uplift displacement) of isolator units (*Guide* Table 11.5-13).

The *Provisions* requires a response spectrum analysis for the EOC (see *Guide* Table 11.2-1). In general, the response spectrum method of dynamic analysis is considered sufficient for facilities that are located at a stiff soil site, which have an isolation system meeting the criteria of *Provisions* Sec. 13.2.5.2.7, Item 7 [13.2.4.1, Item 7]. However, nonlinear static analysis is used for the design of the EOC, in lieu of response spectrum analysis, to permit explicit modeling of uplift of isolator units. For similar reasons, nonlinear time history analysis is used to verify design parameters.

11.5.3.2 Detailed Modeling Considerations

Although a complete description of the ETABS model is not possible, key assumptions and methods used to model elements of the isolation system and superstructure are described below.

11.5.3.2.1 Mass Eccentricity

Provisions Sec. 13.4.5.2 [13.4.1.1] requires consideration of mass eccentricity. Because the building in the example is doubly symmetric, there is no actual eccentricity of building mass (but such would be modeled if the building were not symmetric). Modeling of accidental mass eccentricity would require several analyses, each with the building mass located at different eccentric locations (for example, four quadrant locations in plan). This is problematic, particularly for dynamic analysis using multiple time history inputs. In this example, only a single (actual) location of mass eccentricity is considered, and calculated demands are increased moderately for the design of the seismic-force-resisting system, and isolation system (for example, peak displacements calculated by dynamic analysis are increased by 10 percent for design of the isolation system).

11.5.3.2.2 P-delta Effects

P-delta moments in the foundation and the first floor girders just above isolator units due to the large lateral displacement of the superstructure are explicitly modeled. The model distributes half of the P-delta moment to the structure above, and half of the *P*-delta moment to the foundation below the isolator units. ETABS (version 7.0) permits explicit modeling of the P-delta moment, but most computer programs (including older versions of ETABS) do not. Therefore, the designer must separately calculate these moments and add them to other forces for the design of affected elements. The P-delta moments are quite significant, particularly at isolator units that resist large earthquake overturning loads along lines of lateral bracing.

11.5.3.2.5 Isolator Unit Uplift

Provisions Sec. 13.6.2.7 [13.2.5.7] permits local uplift of isolator units provided the resulting deflections do not cause overstress of isolator units or other structural elements. Uplift of some isolator units is likely (for unreduced earthquake loads) due to the high seismic demand associated with the site. Accordingly, isolator units are modeled with gap elements that permit uplift when the tension load exceeds the tensile capacity of an isolator unit. Although most elastomeric bearings can resist some tension stress before yielding (typically about 150 psi), isolator units are assumed to yield as soon as they are loaded in tension, producing larger estimates of uplift displacement and overturning loads on isolator units that are in compression.

The assumption that isolator units have no tension capacity is not conservative, however, for design of the connections of isolator units to the structure above and the foundation below. The design of anchor bolts and other connection elements, must include the effects of tension in isolator units (typically based on a maximum stress of 150 psi).

11.5.3.2.4 Bounding Values of Bilinear Stiffness of Isolator Units

The design of elements of the seismic-force-resisting system is usually based on a linear, elastic model of the superstructure. When such models are used, *Provisions* Sec. 13.4.5.3.2 [13.4.1.2] requires that the stiffness properties of nonlinear isolation system components be based on the maximum effective stiffness of the isolation system (since this assumption produces larger earthquake forces in the superstructure). In contrast, the *Provisions* require that calculation of isolation system displacements be based on the minimum effective stiffness of the isolation system (since this assumption produces larger isolation system displacements).

The concept of bounding values, as discussed above, applies to all of the available analysis methods. For the ELF procedure, the *Provisions* equations are based directly on maximum effective stiffness, k_{Dmax} and k_{Mmax}, are used for calculating design forces; and on minimum effective stiffness, k_{Dmin} and k_{Dmax}, are used for calculating design displacements. Where (nonlinear) time history analysis is used, isolators are explicitly modeled as bilinear hysteretic elements with upper or lower-bound stiffness curves. Upper-bound stiffness curves are used to verify the forces used for the design of the superstructure and lower-bound stiffness curves are used to verify design displacements of the isolation system.

11.5.4 Preliminary Design Based on the ELF Procedure

11.5.4.1 Calculation of Design Values

11.5.4.1.1 Design Displacements

Preliminary design begins with the engineer's selection of the effective period (and damping) of the isolated structure, and the calculation of the design displacement, D_D. In this example, the effective period of the EOC facility (at the design earthquake displacement) is T_D = 2.5 seconds; and the design displacement is calculated as follows, using *Provisions* Eq. 13.3.3.1:

$$D_D = \left(\frac{g}{4\pi^2}\right)\frac{S_{D1}T_D}{B_D} = (9.8)\frac{0.9(2.5)}{1.35} = 16.3 \text{ in.}$$

The 1.35 value of the damping coefficient, B_D, is given in *Provisions* Table 13.3.3.1 [13.3-1] assuming 15 percent effective damping at 16.3 in. of isolation system displacement. Effective periods of 2 to 3 seconds and effective damping values of 10 to 15 percent are typical of high-damping rubber (and other types of) bearings.

Stability of the isolation system must be checked for the maximum displacement, D_M, which is calculated using *Provisions* Eq. 13.3.3.3 as follows:

$$D_M = \left(\frac{g}{4\pi^2}\right)\frac{S_{M1}T_M}{B_M} = (9.8)\frac{1.35(2.5)}{1.35} = 24.5 \text{ in.}$$

For preliminary design, the effective period and effective damping at maximum displacement are assumed to be the same as the values at the design displacement. While both the effective period and damping values may reduce slightly at larger rubber strains, the ratio of the two parameters tend to be relatively consistent.

The total displacement of specific isolator units (considering the effects of torsion) is calculated based on the plan dimensions of the building, the total torsion (due to actual, plus accidental eccentricity), and the distance from the center of resistance of the building to the isolator unit of interest. Using *Provisions* Eq. 13.3.3.5-1 and 13.3.3.5-2 [13.3-5 and 13.3-6], the total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of isolator units located on Column Lines 1 and 7 are calculated for the critical (transverse) direction of earthquake load as follows:

$$D_{TD} = D_D\left[1 + y\left(\frac{12e}{b^2+d^2}\right)\right] = 16.3\left[1 + 90\left(\frac{12(0.05)(180)}{120^2+180^2}\right)\right] = 16.3(1.21) = 19.7 \text{ in.}$$

$$D_{TM} = D_M\left[1 + y\left(\frac{12e}{b^2+d^2}\right)\right] = 24.5\left[1 + 90\left(\frac{12(0.05)(180)}{120^2+180^2}\right)\right] = 24.5(1.21) = 29.6 \text{ in.}$$

The equations above assume that mass is distributed in plan in proportion to isolation system stiffness and shifted by 5percent, providing no special resistance to rotation of the building on the isolation system. In fact, building mass is considerably greater toward the center of the building, as shown by the schedule of gravity loads in Table 11.5-1. The stiffness of the isolation system is uniform in plan (since all isolators are of the same size) providing significant resistance to dynamic earthquake rotation of the building. While the *Provisions* permit a reduction in the total displacements calculated using the ELF procedure (with proper substantiation of resistance to torsion), in this example the 21 percent increase is considered to be conservative for use in preliminary design and for establishing lower-bound limits on dynamic analysis results.

11.5.4.1.2 Minimum and Maximum Effective Stiffness

Provisions Eq. 13.3.3.2 [13.3-2] expresses the effective period at the design displacement in terms of building weight (dead load) and the minimum effective stiffness of the isolation system, k_{Dmin}. Rearranging terms and solving for minimum effective stiffness:

$$k_{Dmin} = \left(\frac{4\pi^2}{g}\right)\frac{W}{T_D^2} = \left(\frac{1}{9.8}\right)\frac{14{,}715}{2.5^2} = 240 \text{ kips/in.}$$

This stiffness is about 6.9 kips/in. for each of 35 identical isolator units. The effective stiffness can vary substantially from one isolator unit to another and from one cycle of prototype test to another. Typically, an isolator unit's effective stiffness is defined by a range of values for judging acceptability of prototype (and production) bearings. The minimum value of the stiffness range, k_{Dmin}, is used to calculate isolation system design displacements; the maximum value of the stiffness, k_{Dmax}, is used to define design forces.

The variation in effective stiffness depends on the specific type of isolator, elastomeric compound, loading history, etc., but must, in all cases, be broad enough to comply with the *Provisions* Sec. 13.9.5.1 [13.6.4.1]requirements that define maximum and minimum values of effective stiffness based on testing of isolator unit prototypes. Over the three required cycles of test at D_D, the maximum value of effective stiffness (for example, at the first cycle) should not be more than about 30 percent greater than the minimum value of effective stiffness (for example, at the third cycle) to comply with *Provisions* Sec.

13.9.4 [13.6.3]. On this basis, the maximum effective stiffness, k_{Dmax}, of the isolation system in this example, is limited to 312 kips/in. (that is, 1.3 × 240 kips/in.).

The range of effective stiffness defined by k_{Dmin} and k_{Dmax} (as based on cyclic tests of prototype isolator units) does not necessarily bound all the possible variations in the effective stiffness of elastomeric bearings. Other possible sources of variation include: stiffness reduction, due to post-fabrication "scragging" of bearings (by the manufacturer), and partial recovery of this stiffness over time. Temperature and aging effects of the rubber material, and other changes in properties that can also occur over the design life of isolator units. (Elastomeric bearings are typically "scragged" immediately following molding and curing to loosen up the rubber molecules by application of vertical load.) *Provisions* Sec. 13.6.2.1 [13.2.5.1] requires that such variations in isolator unit properties be considered in design, but does not provide specific criteria. A report - *Property Modification Factors for Seismic Isolation Bearings* (Constantinou, 1999) - provides guidance for establishing a range of effective stiffness (and effective damping) properties that captures all sources of variation over the design life of the isolator units. The full range of effective stiffness has a corresponding range of effective periods (with different levels of spectral demand). The longest effective period (corresponding to the minimum effective stiffness) of the range would be used to define isolation system design displacements; and the shortest effective period (corresponding to the maximum effective stiffness) of the range would be used to define the design forces on the superstructure.

11.5.4.1.3 Lateral Design Forces

The lateral force required for the design of the isolation system, foundation, and other structural elements below the isolation system, is given by *Provisions* Eq. 13.3.4.1 [13.3-7]:

$$V_b = k_{Dmax} D_D = 312(16.3) = 5{,}100 \text{ kips}$$

The lateral force required for checking stability and ultimate capacity of elements of the isolation system, may be calculated as follows:

$$V_{MCE} = k_{Dmax} D_M = 312(24.5) = 7{,}650 \text{ kips}$$

The (unreduced) base shear of the design earthquake is about 35 percent of the weight of the EOC, and the (unreduced) base shear of the MCE is just over 50 percent of the weight. In order to design the structure above the isolation system, the design earthquake base shear is reduced by the R_I factor in *Provisions* Eq. 13.3.4.2 [13.3-8]:

$$V_s = \frac{k_{Dmax} D_D}{R_I} = \frac{312(16.3)}{2.0} = 2{,}550 \text{ kips}$$

This force is about 17 percent of the dead load weight of the EOC, which is somewhat less than, but comparable to, the force that would be required for the design of a conventional, fixed-base building of the same size and height, seismic-force-resisting system, and site seismic conditions. Story shear forces on the superstructure are distributed vertically over the height of the structure in accordance with *Provisions* Eq. 13.3.5 [13.3-9], as shown in Table 11.5-3.

Table 11.5-3 Vertical Distribution of Reduced Design Earthquake Forces (DE/2)

Story level, x	Weight, w_x (kips)	Height above isolation system, h_x (ft)	Force, $F_x = \frac{w_x h_x V_s}{\sum_i w_i h_i}$ (kips)	Cumulative force (kips)	Force divided by weight, $\frac{F_x}{w_x}$
Penthouse roof	965	64	370	370	0.38
Roof	3,500	49	1,020	1,390	0.29
Third floor	3,400	34	690	2,080	0.20
Second floor	3,425	19	390	2,470	0.11
First floor	3,425	4	80	2,550	0.023

1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN.

Provisions Eq. 13.3.5 distributes lateral seismic design forces (DE/2) over the height of the building in an inverted, triangular, pattern as indicated by the ratio of F_x/w_x, shown in Table 11.5-3. Because the superstructure is much stiffer laterally, than the isolation system, it tends to move as a rigid body in the first mode with a pattern of lateral seismic forces that is more uniformly distributed over the height of the building. The use of a triangular load pattern for design is intended to account for higher-mode response that may be excited due to flexibility of the superstructure. *Provisions* Eq. 13.3.5 [13.3-9] is also used to distribute forces over the height of the building for unreduced DE and MCE forces, as summarized in Table 11.5-4.

Table 11.5-4 Vertical Distribution of Unreduced DE and MCE Forces

Story level, x	Design earthquake (DE)			Maximum considered earthquake (MCE)		
	Force (kips)	Cumulative force (kips)	Force divided by weight	Force (kips)	Cumulative force (kips)	Force divided by weight
Penthouse roof	740	740	0.76	1,110	1,110	1.14
Roof	2,040	2,780	0.59	3,060	4,170	0.88
Third floor	1,380	4,160	0.41	2,070	6,240	0.61
Second floor	780	4,940	0.23	1,170	7,410	0.34
First floor	160	5,100	0.047	240	7,650	0.071

1.0 kip = 4.45 kN.

11.5.4.1.4 Design Earthquake Forces for Isolator Units

Tables 11.5-5 and 11.5-6 show the maximum and minimum downward forces for design of the isolator units. These forces are a result from the simultaneous application of unreduced design earthquake story forces, summarized in Table 11.5-4 and appropriate gravity loads to the model of the EOC. (See *Guide* Sec. 11.5.2.5 for the design load combinations.) As described in *Guide* Sec. 11.5.2.5, loads are applied simultaneously in two horizontal directions. The tables report the results for both of the orientations: 100 percent in the X direction, plus 30 percent in the Y direction, and 30 percent in the X direction, plus 100 percent in the Y direction. Where the analyses indicate that certain isolator units could uplift during peak DE response (as indicated by zero downward force), the amount of uplift is small and would not appreciably affect the distribution of earthquake forces in the superstructure.

Table 11.5-5 Maximum Downward Force (kips) for Isolator Design ($1.4D + 0.5L + Q_{DE}$)*

Column line	Maximum downward force (kips) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	347/348	661/892	522/557	652/856
B	833/634	1,335/**1,519**	1,418/1,278	984/1,186
C	537/502	939/890	1,053/1,046	1,074/1,070

1.0 kip = 4.45 kN

.*Forces at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; loads at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

Table 11.5-6 Minimum Downward Force (kips) for Isolator Design ($0.7D - Q_{DE}$)*

Column line	Maximum downward force (kips) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	87/84	165/**0**	200/169	163/**0**
B	**0**/123	11/**0**	23/69	280/59
C	169/196	299/243	428/427	447/442

1.0 kip = 4.45 kN

* Forces at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; loads at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

11.5.4.1.5 Maximum Considered Earthquake Forces and Displacements for Isolator Units

Simultaneous application of the unreduced MCE story forces, as summarized in Table 11.5-4 and appropriate gravity loads to the model of the EOC, result in the maximum downward forces on isolator units shown in *Guide* Table 11.5-7, and the maximum uplift displacements shown in Table 11.5-8. The load orientations and MCE load combinations, are described in *Guide* Sec. 11.5.2.5. The tables report the results for both of the load orientations: 100 percent in the X direction, plus 30 percent in the Y direction, and 30 percent in the X direction, plus 100 percent in the Y direction. Since the nonlinear model assumes that the isolators have no tension capacity, the values given in *Guide* Table 11.5-8 are upper bounds on uplift displacements.

Table 11.5-7 Maximum Downward Force (kips) on Isolator Units ($1.5D + 1.0L + Q_{MCE}$)*

Column line	Maximum downward force (kips) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	444/445	829/1,202	645/710	819/1,146
B	1,112/794	1,739/**2,006**	1,848/1,635	1,219/1,521
C	680/618	1,171/1,091	1,298/1,282	1,316/1,307

1.0 kip = 4.45 kN.
* Forces at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; loads at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

Table 11.5-8 Maximum Uplift Displacement (in.) of Isolator Units ($0.8D - Q_{MCE}$)*

Column line	Maximum uplift displacement (in.) 100%(X) ± 30%(Y) / 30%(X) ± 100%(Y)			
	1	2	3	4
A	No uplift	0.00/**0.94**	No uplift	0.00/0.50
B	0.54/0.00	0.19/0.45	0.13/0.00	0.00/0.14
C	No uplift	No uplift	No uplift	No uplift

1.0 in. = 25.4 mm.
* Displacements at column lines 5, 6 and 7 (not shown) are the same as those at column lines 3, 2, and 1, respectively; displacements at column lines D and E (not shown) are the same as those at column lines B and A, respectively.

11.5.4.1.6 Limits on Dynamic Analysis

The displacements and forces determined by the ELF procedure provide a basis for expeditious assessment of size and capacity of isolator units and the required strength of the superstructure. The results of the ELF procedure also establish limits on design parameters when dynamic analysis is used as the basis for design. Specifically, the total design displacement, D_{TD}, and the total maximum displacement of the isolation system, D_{TM}, determined by dynamic analysis cannot be less than 90 percent and 80 percent, respectively, of the corresponding ELF procedure values:

$$D_{TD,\,dynamic} \geq 0.9 D_{TD,\,\mathrm{ELF}} = 0.9(19.7) = 17.7 \text{ in.}$$

$$D_{TM,\,dynamic} \geq 0.8 D_{TM,\,\mathrm{ELF}} = 0.8(29.6) = 23.7 \text{ in.}$$

The superstructure, if regular, can also be designed for less base shear, but not less than 80 percent of the base shear from the ELF procedure:

$$V_{s,\,dynamic} \geq 0.8 V_{s,\,\mathrm{ELF}} = 0.8(2{,}550) = 2{,}040 \text{ kips } (= 0.14W)$$

As an exception to the above, design forces less than 80 percent of the ELF results are permitted if justified by time history analysis (which is seldom, if ever, the case).

11.5.4.2 Design of the Superstructure

The lateral forces, developed in the previous section, in combination with gravity loads, provide a basis for the design of the superstructure, using methods similar to those used for a conventional building. In this example, selection of member sizes were made based on the results of ETABS model calculations. Detailed descriptions of the design calculations are omitted, since the focus of this section is on design aspects unique to isolated structures (i.e., design of the isolation system, which is described in the next section).

Figures 11.5-8 and 11.5-9 are elevation views at Column Lines 2 and B, respectively. Figure 11.5-10 is a plan view of the building that shows the framing at the first floor level.

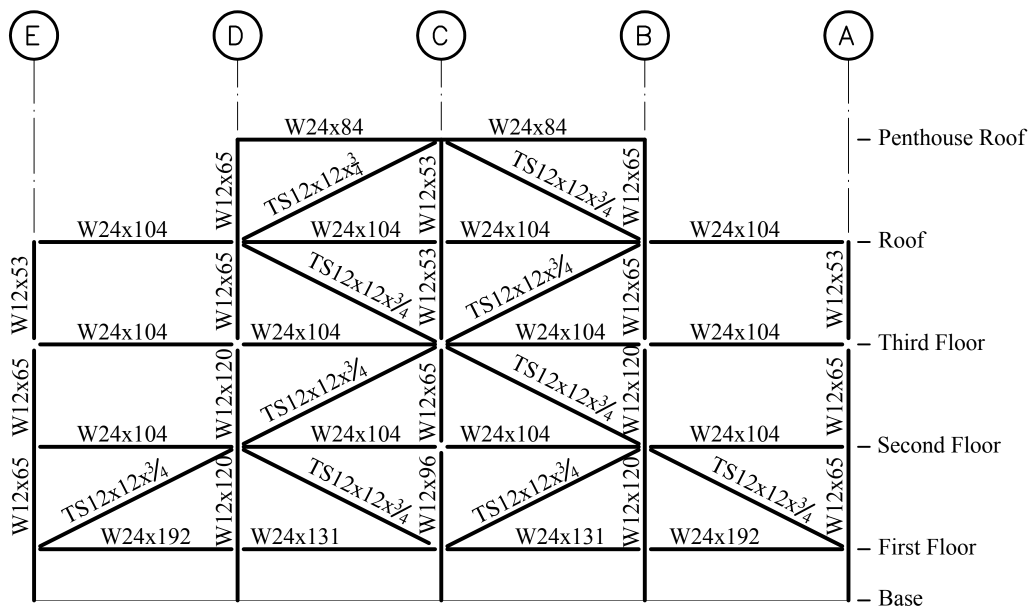


Figure 11.5-8 Elevation of framing on Column Line 2 (Column Line 6 is similar).

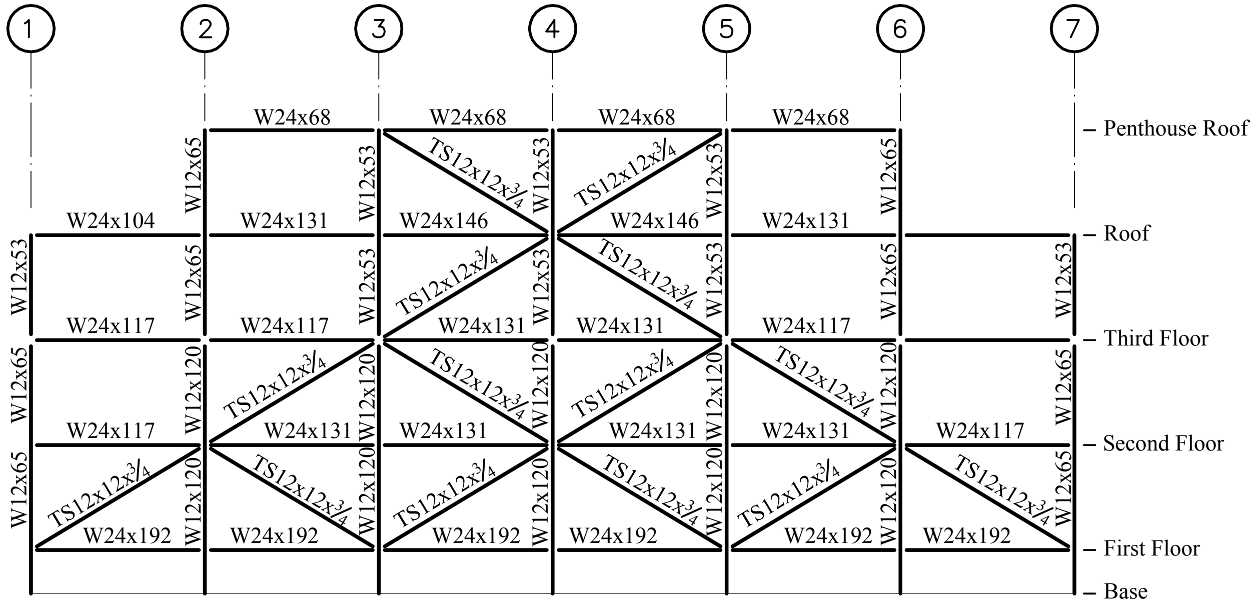


Figure 11.5-9 Elevation of framing on Column Line B (Column Line D is similar).

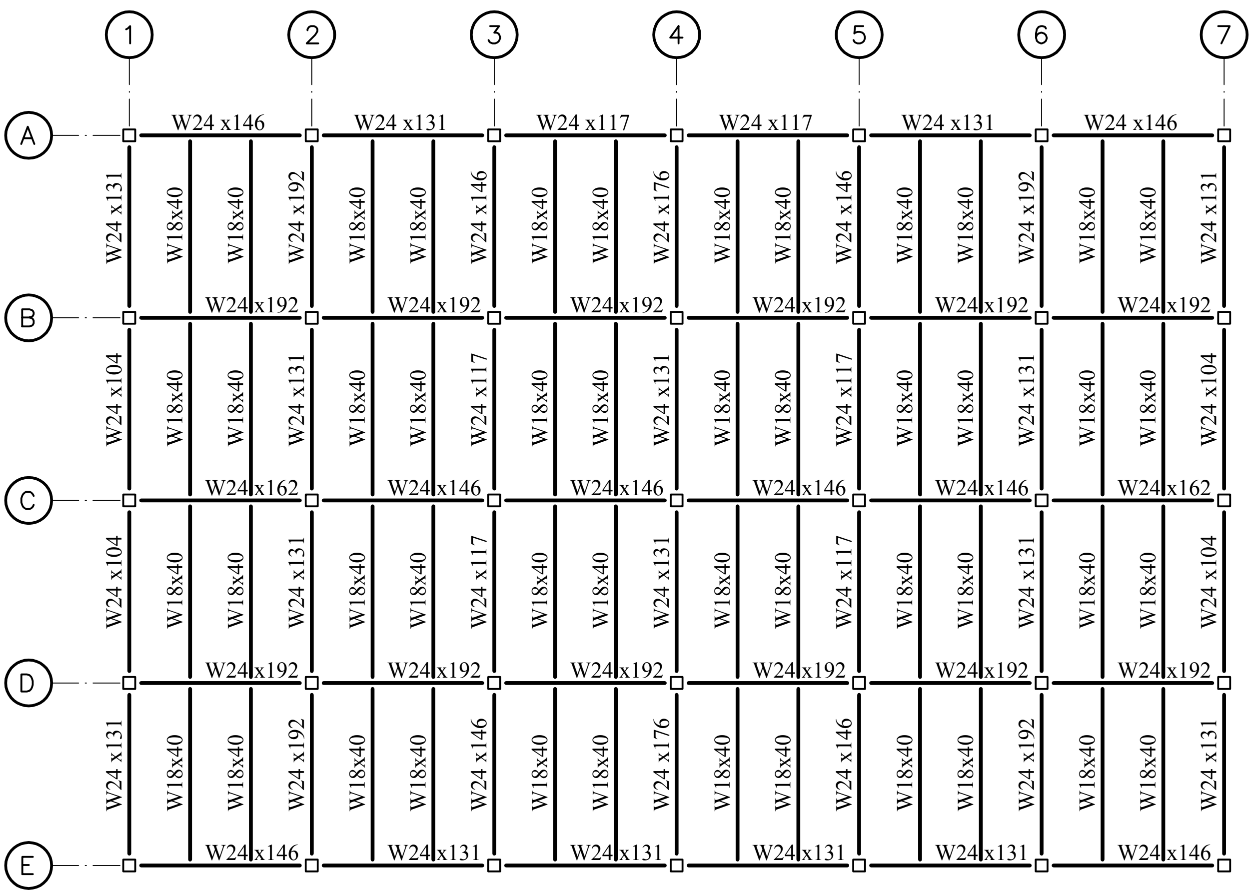


Figure 11.5-10 First floor framing plan.

As shown in the elevations (Figures 11.5-8 and 11.5-9), fairly large (12×12×¾ in.) tubes are consistently used throughout the structure for diagonal bracing. A quick check of these braces indicates that stresses will be at, or below yield for design earthquake loads. The six braces at the third floor, on lines 2, 4, and 6 (critical floor and direction of bracing) carry a reduced design earthquake force of about 400 kips each (= 2,080 kips/6 braces × cos(30°)). The corresponding stress is about 12.5 ksi for reduced design earthquake forces, or about 25 ksi for unreduced design earthquake forces; indicating that the structure is expected to remain elastic during the design earthquake.

As shown in Figure 11.5-10, the first floor framing has heavy, W24 girders along lines of bracing (lines B, D, 2, 4, and 6). These girders resist P-delta moments, as well as other forces. A quick check of these girders indicates that only limited yielding is likely, even for the MCE loads (up to about 2 ft of MCE displacement). Girders on Line 2 that frame into the column at Line B (critical columns and direction of framing), resist a P-delta moment due to the MCE of about 1,000 kip-ft (2,000 kips/2 girders × 2 ft/2). Moment in these girders due to MCE shear force in isolators is about 450 kip-ft (8.9 kips/in. × 24 in. × 4 ft/2 girders). Considering additional moment due to gravity loads, the plastic capacity of the first floor girders (1,680 kip-ft) would not be reached until isolation system displacements exceed about 2 ft. Even beyond this displacement, post-yield deformation would be limited (due to the limited extent and duration of MCE displacements beyond 2 ft), and the first floor girders would remain capable of stabilizing the isolator units.

11.5.4.3 Design of the Isolation System

The displacements and forces calculated in *Guide* Sec. 11.5.4.1 provide a basis to either:

1. Develop a detailed design of the isolator units or
2. Include appropriate design properties in performance-based specifications.

Developing a detailed design of an elastomeric bearing, requires a familiarity with rubber bearing technology, that is usually beyond the expertise of most structural designers; and often varies based on the materials used by different manufacturers. This example, like most recent isolation projects, will define design properties for isolator units that are appropriate for incorporation into a performance specification (and can be bid by more than one bearing manufacturer). Even though the specifications will place the responsibility for meeting performance standards with the supplier, the designer must still be knowledgeable of available products and potential suppliers, to ensure success of the design.

11.5.4.3.1 Size of Isolator Units

The design properties of the seismic isolator units are established based on the calculations of ELF demand, recognizing that dynamic analysis is required to verify these properties (and will likely justify slightly more lenient properties). The key parameters influencing size are:

1. Peak displacement of isolator units,
2. Average long-term (gravity) load on all isolator units,
3. Maximum long-term (gravity) load on individual isolator units, and
4. Maximum short-term load on individual isolator units (gravity plus MCE loads) including maximum uplift displacement.

These parameters are summarized in *Guide* Table 11.5-9. Loads or displacements on individual isolator units are taken as the maximum load or displacement on all isolator units (since the design is based on only one size of isolator unit). Reduced live load is used for determining long-term loads on isolators.

Table 11.5-9 Summary of Key Design Parameters [isolator unit location]

Key Design Parameter	ELF Procedure or Gravity Analysis	Dynamic Analysis Limit
DE displacement at corner of building (D_{TD})	19.7 in. [A1]	> 17.7 in. [A1]
MCE displacement at corner of building (D_{TM})	29.6 in. [A1]	> 23.7 in. [A1]
Average long-term load ($1.0D + 0.5L$)	477 kips [ALL]	N/A
Maximum long-term load ($1.2D + 1.6L$)	1,053 kips [C4]	N/A
DE max short-term load ($1.4D + 0.5L + Q_{DE}$)	1,519 kips [B2]	N/A
DE minimum short-term load ($0.7D - Q_{DE}$)	150 kips uplift [A2]	N/A
MCE max short-term load ($1.5D + 1.0L + Q_{MCE}$)	2,006 kips [B2]	N/A
MCE minimum short-term load ($0.8D - Q_{MCE}$)	0.94 in. uplift [A2]	N/A

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

As rule of thumb, elastomeric isolators should have a diameter, excluding the protective layer of cover, of no less than 1.25 times maximum earthquake displacement demand. In this case, the full displacement determined by the ELF procedure would require an isolator diameter of:

$$\varnothing_{ISO} \geq 1.25(29.6) = 37 \text{ in. } (0.95 \text{ m})$$

If justified by dynamic analysis, then 80 percent of the ELF procedure displacement would require an isolator diameter of:

$$\varnothing_{ISO} \geq 1.25(23.7) = 30 \text{ in. } (0.75 \text{ m})$$

For this example, a single size of isolator unit is selected with a nominal diameter of no less than 35.4 in. (0.90 m). Although the maximum vertical loads vary enough to suggest smaller diameter of isolator units at certain locations (such as at building corners), all of the isolator units must be large enough to sustain MCE displacements; which are largest at building corners, due to torsion. An isolator unit with a diameter of 35.4 in., has a corresponding bearing area of about A_b = 950 square in. The maximum long-term face pressure is about 1,100 psi (i.e., 1,053 kips/950 in.^{2}), which is less than the limit for most elastomeric bearing compounds. Average long-term face pressure is about 500 psi (i.e., 477 kips/950 in.^{2}) indicating the reasonably good distribution of loads among all isolator units.

The minimum effective stiffness is 6.9 kips/in. per isolator unit at the design displacement (i.e., about 16 in.). The height of the isolator unit is primarily a function of the height of the rubber, h_r, required to achieve this stiffness, given the bearing area, A_b, and the effective stiffness of the rubber compound. The EOC design accommodates rubber compounds with minimum effective shear modulus (at 150 percent shear strain) ranging from $G_{150\%}$ = 65 psi to 110 psi. Numerous elastomeric bearing manufacturers have rubber compounds with a shear modulus that falls within this range. Since rubber compounds (and in particular, high-damping rubber compounds) are nonlinear, the effective stiffness used for design must be associated with a shear strain that is close to the strain level for the design earthquake (e.g., 150 percent shear strain). For a minimum effective shear modulus of 65 psi, the total height of the rubber, h_r, would be:

$$h_r = \frac{G_{150\%}A_b}{k_{eff}} = \frac{65\text{ lb/in.}^2 \times 950\text{ in.}^2}{6{,}900\text{ lb/in.}} = 8.9\text{ in.} \cong 9\text{ in.}$$

The overall height of the isolator unit, H, including steel shim and flange plates, would be about 15 in. For compounds with an effective minimum shear modulus of 110 psi, the rubber height would be proportionally taller (about 15 in.), and the total height of the isolator unit would be about 24 in.

11.5.4.3.2 Typical Isolation System Detail

For the EOC design, the isolation system has a similar detail at each column, as shown in Figure 11.5-11. The column has an extra large base plate that bears directly on the top of the isolator unit. The column base plate is circular, with a diameter comparable to that of the top plate of the isolator unit. Heavy, first floor girders frame into, and are moment connected to the columns (moment connections are required at this floor only). The columns and base plates are strengthened by plates that run in both horizontal directions, from the bottom flange of the girder to the base. Girders are stiffened above the seat plates, and at temporary jacking locations. The top plate of the isolator unit is bolted to the column base plate, and the bottom plate of the isolator unit is bolted to the foundation.

The foundation connection accommodates the removal and replacement of isolator units, as required by *Provisions* Sec. 13.6.2.8 [13.2.5.8]. The bottom plate of the isolator unit bears on a steel plate, that has a shear lug at the center grouted to the reinforced concrete foundation. Anchor bolts pass through holes in this plate and connect to threaded couplers that are attached to deeply embedded rods.

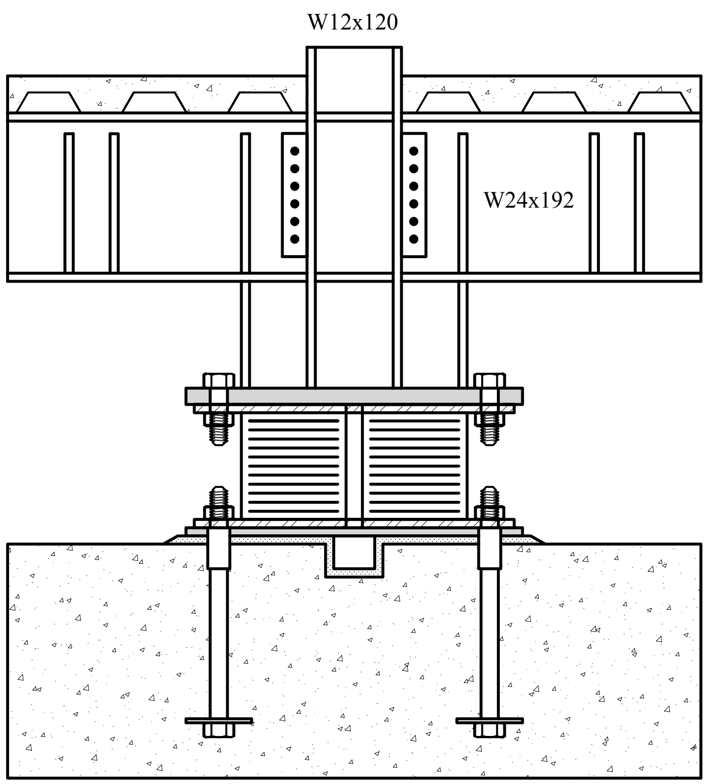


Figure 11.5-11 Typical detail of the isolation system at columns (for clarity, some elements not shown).

With the exception of the portion of the column above the first floor slab, each element shown in Figure 11.5-11 is an integral part of the isolation system (or foundation), and is designed for the gravity and unreduced design earthquake loads. In particular, the first floor girder, the connection of the girder to the column, and the connection of the column to the base plate, are designed for gravity loads and forces caused by horizontal shear and P-delta effects due to the unreduced design earthquake load (as shown earlier in Figure 11.4-1).

11.5.4.3.3 Design of Connections of Isolator Units

Connection of the top plate of the isolator unit, to the column base plate; and the connection of the bottom plate to the foundation, are designed for load combinations that include maximum downward forces (1.4D + 0.5L + Q_{DE}), and minimum downward (uplift) forces (0.7D - Q_{DE}). The reactions to bolts at the top and bottom plates of isolator units (ignoring shear friction and shear capacity of the lug at the base) are approximately equal, and include shear, axial load, and moment, in the most critical direction of response for individual bolts. Moments include the effects of P-delta and horizontal shear across the isolator unit, as described by the equations shown in Figure 11.4-1:

$$M = \frac{P\Delta}{2} + \frac{VH}{2}$$

In this case, $H_1 = H_2 = H/2$, where H is the height of the isolator unit (assumed to be 24 in., the maximum permissible height of isolator units). For maximum downward acting loads, maximum tension on any one of N (12) uniformly spaced bolts located in a circular pattern of diameter, D_b, must be equal to 43 in. and is estimated as follows:

$$F = \left(\frac{M}{S} - \frac{P}{A_b}\right)\frac{A_b}{N} = \left(\frac{4M}{A_b \cdot D_b} - \frac{P}{A_b}\right)\frac{A_b}{N} = \frac{1}{N}\left(\frac{4M}{D_b} - P\right) = \frac{1}{N}\left(\frac{2(P\Delta + VH)}{D_b} - P\right)$$

$$= \frac{1}{12}\left(\frac{2(1{,}519\text{ kips})(19.7\text{ in.}) + 175\text{ kips}(24\text{ in.})}{43\text{ in.}} - 1{,}519\text{ kips}\right) = \frac{1{,}587 - 1{,}519}{12} \cong 6\text{ kips}$$

In this calculation, the vertical load, P = 1,519 kips, is the maximum force occurring at Column B2, the deflection is based on D_{TD} = 19.7 in., and the shear force is calculated as V = 8.9 kips/in. × 19.7 in. The calculation indicates only a modest amount of tension force in anchor bolts, due to the maximum downward loads on the isolator units. However, the underlying assumption of the plane section's remaining plane is not conservative if top and bottom flange plates yield during a large lateral displacement of the isolator units. Rather than fabricate bearings with overly thick flange plates, manufacturers usually recommend anchor bolts that can carry substantial amounts of shear and tension, which can arise due to local bending of flanges.

Tension forces in anchor bolts can occur due to local uplift of isolator units. The ETABS model did not calculate uplift forces, since it was assumed that the isolator units would yield in tension. An estimate of the uplift load may be based on a maximum tension yield stress of about 150 psi, which produces a tension force, F_t, of no more than 150 kips for isolator units with a bearing area of 950 in.^{2} Applying uplift and shear loads to the isolator unit, the maximum tension force in each bolt is estimated at:

$$F_t = \left(\frac{M}{S} + \frac{P}{A_b}\right)\frac{A_b}{N} = \frac{1}{N}\left(\frac{2(P\Delta + VH)}{D_b} + P\right)$$

$$= \frac{1}{12}\left(\frac{2(150\text{ kips}(19.7\text{ in.}) + 175\text{ kips}(24\text{ in.})}{43\text{ in.}} + 150\text{ kips}\right) = \frac{333 + 150}{12} \cong 40\text{ kips}$$

The maximum shear load per bolt is:

$$F_v = \frac{V}{N} = \frac{175}{12} \cong 15 \text{ kips}$$

Twelve 1¼-in. diameter anchor bolts of A325 material are used for these connections. Alternatively, eight 1½-in. diameter bolts are permitted for isolator units manufactured with eight, rather than twelve, bolt holes. Bolts are tightened as required for a slip-critical connection (to avoid slip in the unlikely event of uplift).

Design of stiffeners and other components of the isolation system detail are not included in this example, but would follow conventional procedures for design loads. Design of the isolator unit including top and bottom flange plates is the responsibility of the manufacturer. *Guide* Sec. 11.5.6 includes example performance requirements of specifications that govern isolator design (and testing) by the manufacturer.

11.5.5 Design Verification Using Nonlinear Time History Analysis

The design is verified (and in some cases isolation system design properties are improved) using time history analysis of models that explicitly incorporate isolation system nonlinearity, including lateral force-deflection properties and uplift of isolator units, which are subject to net tension loads. Using three sets of horizontal earthquake components the EOC is analyzed separately for design earthquake (DE) and MCE ground shaking,. All analyses are repeated for two models of the EOC, one with upper-bound (UB) stiffness properties and the other with lower-bound (LB) stiffness properties of isolator units.

11.5.5.1 Ground Motion

Each pair of horizontal earthquake components is applied to the model in three different orientations relative to the principal axes of the building. Time histories are first applied to produce the maximum response along the X axis of the EOC model. The analyses are then repeated with the time histories, they are rotated 90° to produce the maximum response along the Y axis of the EOC model. Additionally, the time histories are rotated 45° to produce the maximum response along an X-Y line (to check if this orientation would produce greater response in certain elements than the two principal axis orientations). A total of 18 analyses (2 models × 3 component orientations × 3 sets of earthquake components) are performed separately for both DE and the MCE loads. Results are based on the maximum response of the parameter of interest calculated by each DE or MCE analysis, respectively. Parameters of interest include:

Design earthquake

1. Peak isolation system displacement
2. Maximum story shear forces (envelope over building height)

Maximum considered earthquake

1. Peak isolation system displacement
2. Maximum downward load on any isolator unit ($1.5D + 1.0L + Q_{\text{MCE}}$)
3. Maximum uplift displacement of any isolator unit ($0.8L - Q_{\text{MCE}}$).

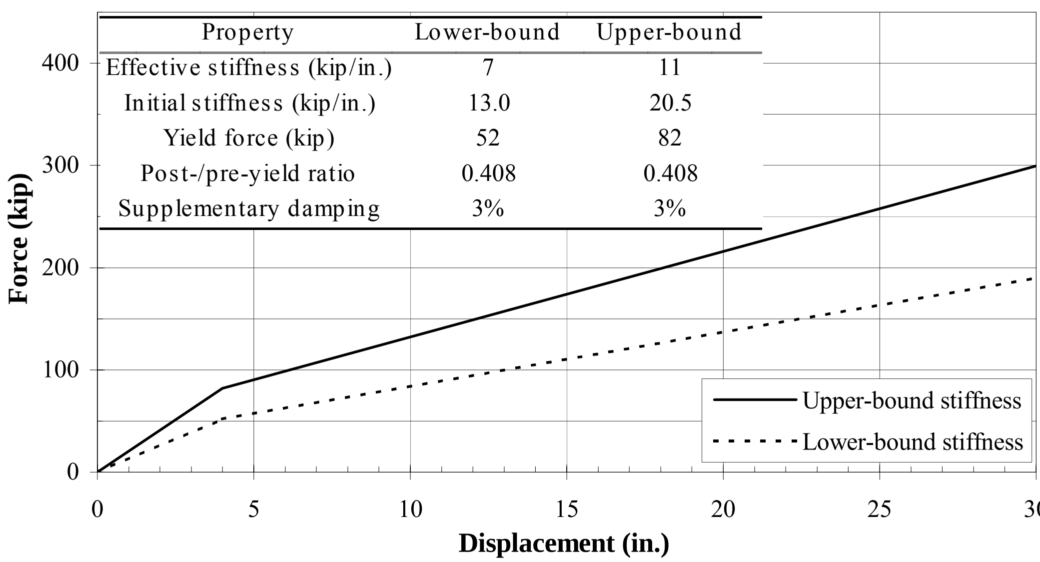
11.5.5.2 Bilinear Stiffness Modeling of Isolator Units

Nonlinear force-deflection properties of the isolator units are modeled using a bilinear curve; whose hysteretic behavior is a parallelogram, which is supplemented by a small amount of viscous damping. A bilinear curve is commonly used to model the nonlinear properties of elastomeric bearings; although other approaches are sometimes used, including a trilinear curve that captures stiffening effects of some rubber compounds at very high strains.

Using engineering judgement, the initial stiffness, the yield force, the ratio of post-yield to pre-yield stiffness for the bilinear curves, and the amount of supplementary viscous damping, β_v, are selected. The effective stiffness and effective damping values are used for preliminary design. The lower-bound bilinear stiffness curve is based on k_{Dmin} = 6.9 kips/in. (rounded to 7 kips/in.). The upper-bound bilinear stiffness curve is based on an effective stiffness of k_{Dmax} = 8.9 kips/in. times 1.2 (which the result is rounded to 11 kips/in.). The 1.2 factor is taken into account for the effects of aging over the design life of the isolators. Both upper-bound and lower-bound stiffness curves are based on effective damping (combined hysteretic and supplementary viscous) of $\beta_D \cong$ 15 percent. Figure 11.5-12 illustrates upper-bound and lower-bound bilinear stiffness curves, and summarizes the values of the parameters that define these curves.

For isolators with known properties, parameters defining bilinear stiffness may be based on test data provided by the manufacturer. For example, the bilinear properties shown in Figure 11.5-12, are compared with effective stiffness and effective damping test data that are representative of a HK090H6 high-damping rubber bearing manufactured by Bridgestone Engineered Products Company, Inc. Bridgestone is one of several manufacturers of high-damping rubber bearings and provides catalog data on design properties of standard isolators. The HK090H6 bearing has a rubber height of about 10 in., a diameter of 0.9 m, without the cover, and a rubber compound with a relatively low shear modulus (less than 100 psi at high strains).

Plots of the effective stiffness and damping of the HK090H6 bearing, are shown in Figure 11.5-13 (solid symbols), and are compared with plots of the calculated effective stiffness and damping (using the equations from Figure 11.4-2 and the bilinear stiffness curves shown in Figure 11.5-12). The effective stiffness curve of the HK090H6 bearing, is based on data from the third cycle of testing; and therefore, best represents the minimum effective (or lower-bound) stiffness. The plots indicate the common trend in effective stiffness of high-damping elastomeric bearings– significant softening up to 100 percent shear strain, fairly stable stiffness from 100 percent to 300 percent shear strain, and significant stiffening beyond 300 percent shear strain. The trend in effective damping is a steady decrease in damping with amplitude beyond about 150 percent shear strain.



Property	Lower-bound	Upper-bound
Effective stiffness (kip/in.)	7	11
Initial stiffness (kip/in.)	13.0	20.5
Yield force (kip)	52	82
Post-/pre-yield ratio	0.408	0.408
Supplementary damping	3%	3%

Figure 11.5-12 Stiffness and damping properties of EOC isolator units (1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN).

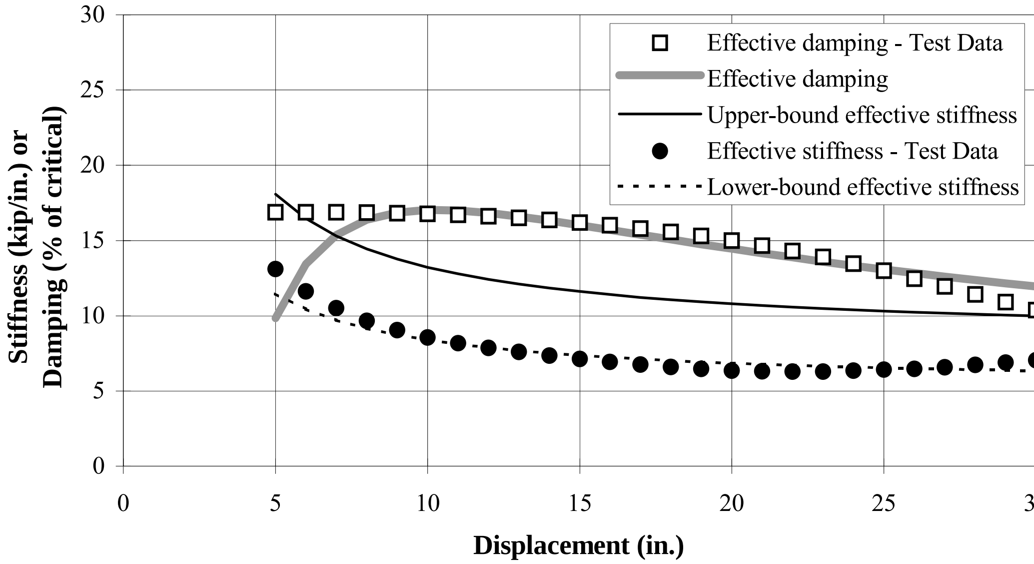


Figure 11.5-13 Comparison of modeled isolator properties to test data (1.0 in. = 25.4 mm, 1.0 kip/in. = 0.175 kN/mm).

The effective stiffness and damping of the bilinear stiffness curves capture the trends of the HK090H6 bearing reasonably well, as shown in Figure 11.5-13. Lower-bound effective stiffness fits the HK090H6 data well for response amplitudes of interest (i.e., the displacements in the range of about 15 in. through 25 in.). Likewise, the effective damping of the bilinear stiffness curves, is conservatively less than the effective damping based on test data for the same displacement range. These comparisons confirm that the bilinear stiffness properties of isolator units used for nonlinear analysis, are valid characterizations of the force-deflection properties of the Bridgestone HK090H6 bearing, also presumably, the bearings of other manufacturers that have comparable values of effective stiffness and damping.

11.5.5.3 Summary of Results for Time History Analyses

Tables 11.5-10 and 11.5-11, compare the results of the design earthquake and maximum considered earthquake time history analyses, respectively, with the corresponding values calculated using the ELF procedure. The peak response values noted for the time history analyses are, the maxima of X-axis and Y-axis directions of response. Because torsional effects were neglected in the time history analyses for this example, the displacement at the corner of the EOC was assumed to be 1.1 times the center displacement; this assumption may not be conservative. The reported shears occur below the indicated level.

Table 11.5-10 Design Earthquake Response Parameters - Results of ELF Procedure and Nonlinear Time History Analysis

Response parameter	ELF procedure	Time history analysis		
		Peak response	Model	Record
Isolation system displacement (in.)				
Displacement at center of EOC, D_D	16.3 in.	14.3 in.	LB stiffness	Sylmar
Displacement at corner of EOC, D_{TD}	19.7 in.	15.7 in.*		
Superstructure forces – story shear (kips)				
Penthouse roof	740 kips	400 kips	UB stiffness	Newhall
Roof (penthouse)	2,780 kips	1,815 kips		
Third floor	4,160 kips	3,023 kips		
Second floor	4,940 kips	4,180 kips		
First floor	5,100 kips	5,438 kips		

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.
* Displacement includes an arbitrary 10 percent increase for possible torsional response.

Table 11.5-11 Maximum Considered Earthquake Response Parameters – Results of ELF Procedure and Nonlinear Time History Analysis

Response parameter	ELF procedure	Time history analysis		
		Peak response	Model	Record
Displacement at center of EOC, D_M	24.5 in.	26.4 in.	LB stiffness	El Centro No. 6
Displacement at corner of EOC, D_{TM}	29.7 in.	29.0 in.[1]		

1.0 in. = 25.4 mm.
* Displacement includes an arbitrary 10 percent increase for possible torsional response.

The results of the design earthquake time history analyses, verify that the results of the ELF procedure are generally conservative. A design displacement of 16 in., and total design displacement of 20 in., are conservative bounds on calculated displacements, even if significant torsion should occur. The story shears calculated using the ELF procedure, are larger than those from the dynamic analyses at all superstructure elevations. The higher the elevation, the larger the margin between the story shears, which is an indication that the inverted triangular pattern (the results from *Provisions* Eq. 13.3.5 [13.3-9]), produces conservative results. At the second story (below the third floor), the critical level for the brace design the story shear from the dynamic analyses (3,023 kips) is only about three-quarters of the story shear from the ELF procedure (4,160 kips). The results of the MCE time history analyses show that the ELF procedure can underestimate the maximum displacement. Accordingly, a maximum displacement of 27 in., and total maximum displacement of 30 in., are used for the design specifications.

Tables 11.5-12 and 11.5-13, respectively, report the maximum downward forces on isolator units and the maximum uplift displacements determined from the maximum considered earthquake time history analyses.

These tables report two values for each isolator location representing both the X-axis and Y-axis orientations of the strongest direction of shaking of the time history record.

Table 11.5-12 Maximum Downward Force (kips) on Isolator Units ($1.5D + 1.0L + Q_{MCE}$)*

Column line	Upper-bound stiffness model – Newhall record – X-axis/Y-axis orientations			
	1	2	3	4
A	417/411	930/1,076	651/683	889/1,027
B	1,048/881	1,612/**1,766**	1,696/1,588	1,308/1,429
C	656/628	1,145/1,127	1,301/1,300	1,319/1,318

1.0 kip = 4.45 kN.
* Forces on Column Lines 5, 6 and 7 (not shown) are enveloped by those on Column Lines 3, 2, and 1, respectively; forces on Column Lines D and E (not shown) are enveloped by those on Column Lines B and A, respectively.

Table 11.5-13 Maximum Uplift Displacement (in.) Of Isolator Units ($0.8D$ - Q_{MCE})[1]

Column line	Upper-bound stiffness model – Newhall record – X-axis/Y-axis orientations			
	1	2	3	4
A	No uplift	0.00/**0.39**	No uplift	0.00/0.17
B	0.26/0.00	0.19/0.12	No uplift	No uplift
C	No uplift	No uplift	No uplift	No uplift

1.0 in. = 25.4 mm.
* Displacement on Column Lines 5, 6 and 7 (not shown) enveloped by those on Column Lines 3, 2, and 1, respectively; displacement on Column Lines D and E (not shown) are enveloped by those on Column Lines B and A, respectively.

Table 11.5-12 indicates a maximum downward force of 1,766 kips (at column B2), which is somewhat less than the value predicted using the ELF procedure (2,006 kips in Table 11.5-7). Table 11.5-13 includes the results from the controlling analysis, and indicates a maximum uplift displacement of 0.39 in. (at column A2), which is significantly less than the value predicted using the ELF procedure (0.94 in. as noted in Table 11.5-8). Based on these results, the design specification (*Guide* Sec. 11.5.2.10) uses values of 2,000 kips maximum downward force and ½ in. of maximum uplift displacement.

11.5.6 Design and Testing Criteria for Isolator Units

Detailed design of the isolator units for the EOC facility, is the responsibility of the bearing manufacturer subject to the design and testing (performance)of the criteria included in the construction documents (drawings and/or specifications). Performance criteria typically includes a basic description, and requirements for isolator unit sizes, the design life and durability, the environmental loads and fire-resistance criteria, Quality Assurance and Quality Control (including QC testing of production units), the design forces and displacements, and prototype testing requirements. This section summarizes key data and performance criteria for the EOC, including the criteria for prototype testing of isolator units, as required by *Provisions* Sec. 13.9.2 [13.6.1].

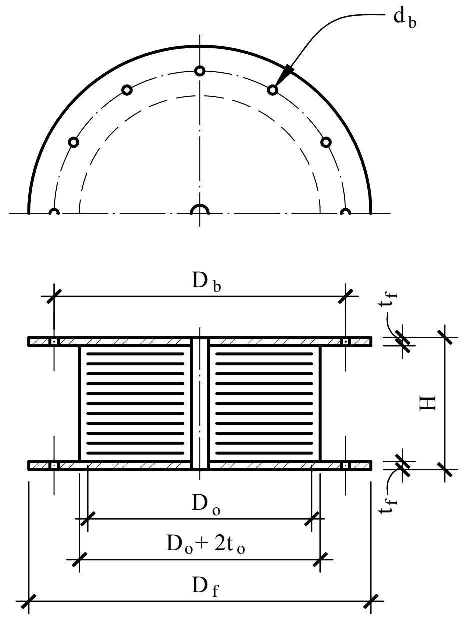


Figure 11.5-14 Isolator dimensions.

11.5.6.1 Dimensions and Configuration

Steel shim plate diameter, $D_o \geq 35.4$ in.

Rubber cover thickness, $t_o \geq 0.5$ in.

Rubber layer thickness, $t_r \leq 0.375$ in.

Gross rubber diameter, $D_o + 2t_o \leq 38$ in.

Bolt pattern diameter, $D_b = 43$ in.

Bolt hole diameter, $d_b = 1.5$ in.

Steel flange plate diameter, $D_f = 48$ in.

Steel flange plate thickness, $t_f \geq 1.5$ in.

Total height, $H \leq 24$ in.

11.5.6.2 Prototype Stiffness and Damping Criteria

Minimum effective stiffness (third cycle of test, typical vertical load) k_{Dmin}, $k_{Mmin} \geq 7.0$ kips/in.

Maximum effective stiffness (first cycle test, typical vertical load) k_{Dmax}, $k_{Mmax} \leq 9.0$ kips/in.

The k_{Dmin}, k_{Dmax}, k_{Mmin}, and $k_{Mmax,}$ are properties of the isolation system as a whole (calculated from the properties of individual isolator units using Provisions Eq. 13.9.5.1-1 through 13.9.5.1-4 [13.6-3 through 13.6-6]). Individual isolator units may have stiffness properties that fall outside the limits (by, perhaps, 10 percent), provided the average stiffness of all isolator units complies with the limits.

Effective damping	
(minimum of three cycles of test, typical vertical load)	$\beta_D \geq$ 15 percent
(minimum of three cycles of test, typical vertical load)	$\beta_M \geq$ 12 percent
Vertical Stiffness	
(average of three cycles of test, typical vertical load ± 50 percent)	$k_v \geq$ 5,000 kips/in.

11.5.6.3 Design Forces (Vertical Loads)

Maximum long-term load (individual isolator)	$1.2D + 1.6L$ = 1,200 kips
Typical load - cyclic load tests (average of all isolators)	$1.0D + 0.5L$ = 500 kips
Upper-bound load - cyclic load tests (all isolators)	$1.2D + 0.5L + \lvert E \rvert$ = 750 kips
Lower-bound load - cyclic load tests (all isolators)	$0.8D - \lvert E \rvert$ = 250 kips
Maximum short-term load (individual isolator)	$1.2D + 1.0L + \lvert E \rvert$ = 2,000 kips
Minimum short-term load (individual isolator)	$0.8D - \lvert E \rvert$ = tension force due to ½ in. of uplift

11.5.6.4 Design Displacements

Design earthquake displacement	D_D = 16 in.
Total design earthquake displacement	D_{TD} = 20 in.
Maximum considered earthquake displacement	D_M = 27 in.
Total maximum considered earthquake displacement	D_{TM} = 30 in.

11.5.6.5 Prototype Testing Criteria

Table 11.5-14 summarizes the prototype test criteria found in *Provisions* Sec. 13.9.2 [13.6.1] as well as the corresponding loads on isolator units of the EOC.

Table 11.5-14 Prototype Test Requirements

	Provisions criteria		EOC criteria	
No. of cycles	Vertical load	Lateral load	Vertical load	Lateral load
Vertical stiffness test				
3 cycles	Typical ± 50%	None	500 ± 250 kips	None
Cyclic load tests to check wind effects (*Provisions* Sec. 13.9.2.3 [13.6.1.2])				
20 cycles	Typical	Design force	500 kips	± 20 kips
Cyclic load tests to establish effective stiffness and damping (*Provisions* Sec. 13.9.2.3 [13.6.1.2])				
3 cycles	Typical	$0.25D_D$	500 kips	± 4 in.
3 cycles	Upper-bound*	$0.25D_D$	750 kips	± 4 in.
3 cycles	Lower-bound*	$0.25D_D$	250 kips	± 4 in.
3 cycles	Typical	$0.5D_D$	500 kips	± 8 in.
3 cycles	Upper-bound*	$0.5D_D$	750 kips	± 8 in.
3 cycles	Lower-bound*	$0.5D_D$	250 kips	± 8 in.
3 cycles	Typical	$1.0D_D$	500 kips	± 16 in.
3 cycles	Upper-bound*	$1.0D_D$	750 kips	± 16 in.
3 cycles	Lower-bound*	$1.0D_D$	250 kips	± 16 in.
3 cycles	Typical	$1.0D_M$	500 kips	± 27 in.
3 cycles	Upper-bound*	$1.0D_M$	750 kips	± 27 in.
3 cycles	Lower-bound*	$1.0D_M$	250 kips	± 27 in.
3 cycles	Typical	$1.0D_{TM}$	500 kips	± 30 in.
Cyclic load tests to check durability (*Provisions* Sec. 13.9.2.3 [13.6.1.2])				
$30S_{D1}/S_{DS}B_D(= 20)$**	Typical load	$1.0D_{TD}$	500 kips	± 20 in.
Static load test of isolator stability (*Provisions* Sec. 13.9.2.6 [13.6.1.5])				
N/A	Maximum	$1.0D_{TM}$	2,000 kips	30 in.
N/A	Minimum	$1.0D_{TM}$	½-in. of uplift	30 in.

1.0 in. = 25.4 mm, 1.0 kip = 4.45 kN.

* Tests with upper-bound and lower-bound vertical loads are required by *Provisions* Sec. 13.9.2.3 [13.6.1.2] for isolator units that are vertical-load-carrying elements.

** The *Provisions* contains a typographical error where presenting this expression. The errata to the *Provisions* correct the error.