Section 15: Commentary
SEISMIC ISOLATION

C15.1 SCOPE

Isolating structures from the damaging effects of earthquakes is not a new idea. The first patents for base isolation schemes were obtained in the 1870s, but until the past two decades, few structures were built using these ideas. Early concerns were focused on the displacements at the isolation interface. These have been largely overcome with the successful development of mechanical energy dissipators. When used in combination with a flexible device such as an elastomeric bearing, an energy dissipator can control the response of an isolated structure by limiting both the displacements and the forces. Interest in seismic isolation, as an effective means of protecting bridges from earthquakes, was revived in the 1970s. To date there are several hundred bridges in New Zealand, Japan, Italy, and the United States using seismic isolation principles and technology for their seismic design.

Seismically isolated buildings such as the University of Southern California Hospital in Los Angeles, and the West Japan Postal Savings Computer Center in Kobe, Japan, performed as expected in the 1994 Northridge and 1995 Kobe earthquakes. Records from these isolated structures show good correlation between the analytical prediction and the recorded performance.

The basic intent of seismic isolation is to increase the fundamental period of vibration such that the structure is subjected to lower earthquake forces. However, the reduction in force is accompanied by an increase in displacement demand that must be accommodated within the isolation system. Furthermore, flexible bridges can be lively under service loads.

The three basic elements in seismic isolation systems that have been used to date are

(a) a vertical-load carrying device that provides lateral flexibility so that the period of vibration of the total system is lengthened sufficiently to reduce the force response,

(b) a damper or energy dissipator so that the relative deflections across the flexible mounting can be limited to a practical design level, and

(c) a means of providing rigidity under low (service) load levels, such as wind and braking forces.

Flexibility – Elastomeric and sliding bearings are two ways of introducing flexibility into a structure. The typical force response with increasing period (flexibility) is shown schematically in the typical acceleration response curve in Figure C15.1-1. Reductions in base shear occur as the period of vibration of the structure is lengthened. The extent to which these forces are reduced primarily depends on the nature of the earthquake ground motion and the period of the fixed-base structure. However, as noted above, the additional flexibility needed to lengthen the period of the structure will give rise to relative displacements across the flexible mount. Figure C15.1-2 shows a typical displacement response curve from which displacements are seen to increase with increasing period (flexibility).

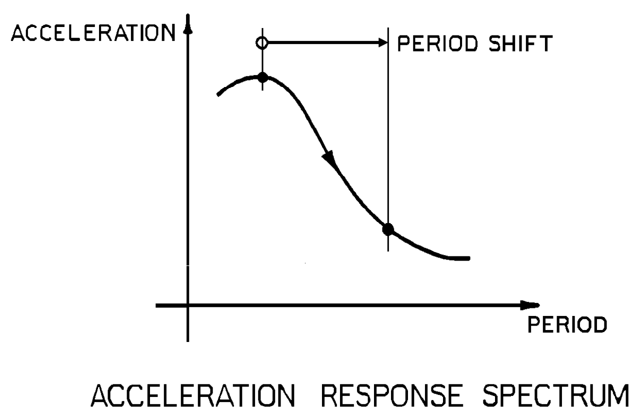


Figure C15.1-1 Typical Acceleration Response Curve

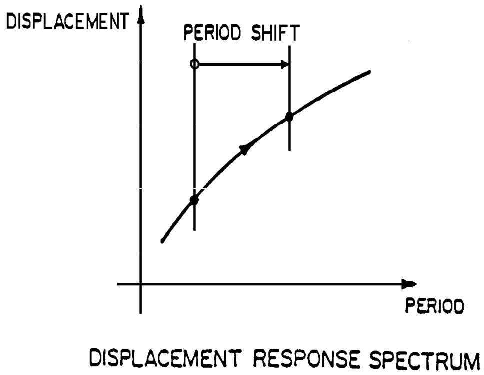


Figure C15.1-2 Typical Displacement Response Curve

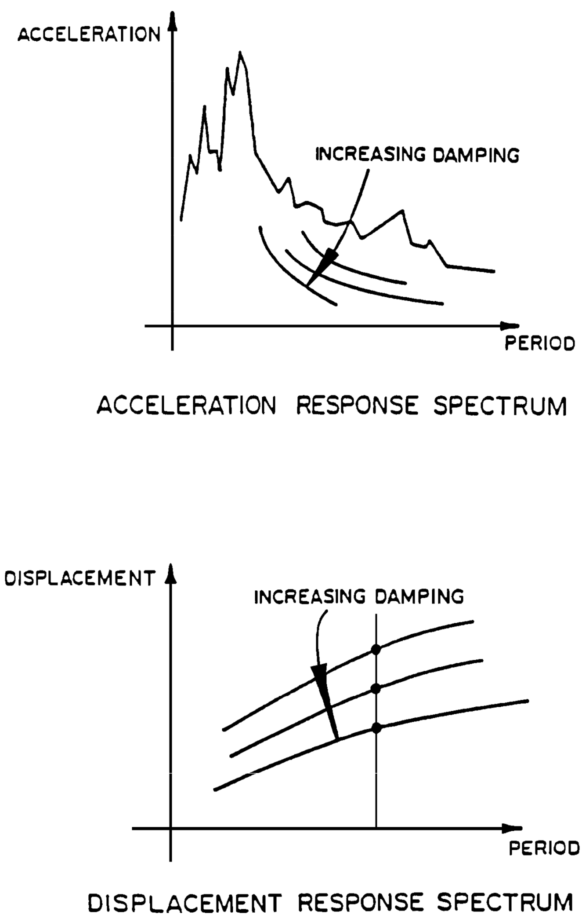
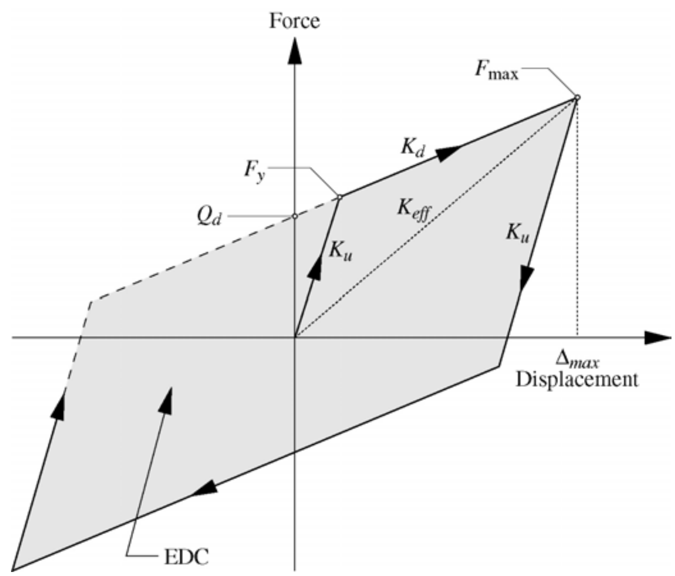


Figure C15.1-3 Response Curves for Increasing Damping

Energy Dissipation – Relative displacements can be controlled if additional damping is introduced into the structure at the isolation level. This is shown schematically in figure C15.1-3.

Two effective means of providing damping are hysteretic energy dissipation and viscous energy dissipation. The term *viscous* refers to energy dissipation that is dependent on the magnitude of the velocity. The term *hysteretic* refers to the offset between the loading and unloading curves under cyclic loading. Figure C15.1-4 shows an idealized force-displacement hysteresis loop where the enclosed area is a measure of the energy dissipated during one cycle (EDC) of motion.



Q_d = Characteristic strength
F_y = Yield force
F_{max} = Maximum force
K_d = Post-elastic stiffness
K_u = Elastic (unloading) stiffness
K_{eff} = Effective stiffness
Δ_{max} = Maximum bearing displacement
EDC = Energy dissipated per cycle = Area of hysteresis loop (shaded)

Figure C15.1-4 Characteristics of Bilinear Isolation Bearings

Rigidity Under Low Lateral Loads – While lateral flexibility is very desirable for high seismic loads, it is clearly undesirable to have a bridge that will vibrate perceptibly under frequently occurring loads, such as wind or braking. External energy dissipators and modified elastomers may be used to provide rigidity at these service loads by virtue of their high initial elastic stiffness (K_u in Figure C15.1-4). As an alternative, friction in sliding isolation bearings may be used to provide the required rigidity.

Example – The principles for seismic isolation are illustrated by figure C15.1-5. The dashed line is the elastic ground response spectrum as specified in Article 3.4.1. The solid line represents the composite response spectrum for an isolated bridge. The period shift provided by the flexibility of the isolation system reduces the spectral acceleration from A_1 to A_2. The increased damping provided by the isolation system further reduces the spectral acceleration from A_2 to A_3. Note that spectral acceleration A_1 and A_3 are used to determine forces for the design of conventional and isolated bridges, respectively.

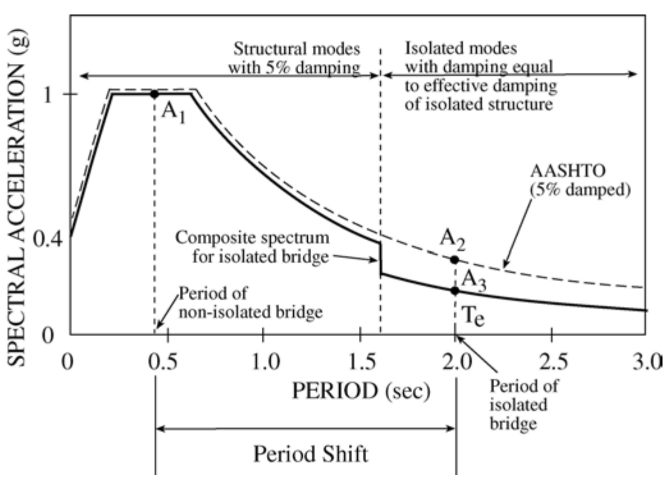


Figure C15.1-5 Response Spectrum for Isolated Bridge

C15.2 DEFINITIONS

ISOLATION SYSTEM

The isolation system does not include the substructure and deck.

OFFSET DISPLACEMENT

The offset displacement is used for prototype testing and designing the isolator units.

C15.3 NOTATION

A_r is defined as the overlap area between the top-bonded and bottom-bonded elastomer areas of a displaced bearing, as shown in figure C15.3-1.

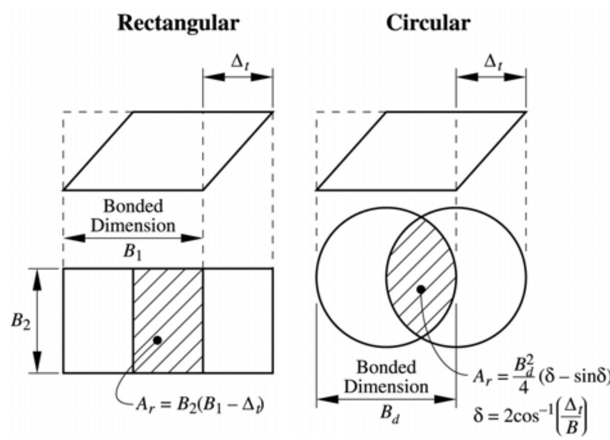


Figure C15.3-1 Definition at Overlap Area

$\bar{k}$ = Material constant related to hardness. (Refer to Roeder, Stanton, and Taylor 1987 for values.)

LL_s, the seismic live load, shall be determined by the engineer as a percentage of the total live load considered applicable for the design.

C15.4 ANALYSIS PROCEDURES

The basic premise for the analysis (consistent with those for buildings and hospitals) is twofold. First, the energy dissipation of the isolation system can be expressed in terms of equivalent viscous damping; and second, the stiffness of the isolation system can be expressed as an effective linear stiffness. These two basic assumptions permit both the single and multimodal methods of analysis to be used for seismic isolation design.

The force deflection characteristics of a bilinear isolation system (Figure C15.1-4) have two important variables, some of which are influenced by environmental and temperature effects. The key variables are K_d, the stiffness of the second slope of the bilinear curve, and Q_d, the characteristic strength. The area of the hysteresis loop, EDC, and hence the damping coefficient, are affected primarily by Q_d. The effective stiffness K_{eff} is influenced by Q_d and K_d.

The two important design variables of an isolation system are K_{eff} and B, the damping coefficient, since they affect the period (Equation 15.4.1-4), the displacement (Equation 15.4.1-3), and the base shear forces (Equation 15.4.1-2). Since K_{eff} and B, the damping coefficient, are affected differently by K_d and Q_d, the impact variations in K_d and Q_d have on the key design variables needs to be assessed (Figure C15.4-1). Article 15.5 provides a method to determine λ_{min} and λ_{max} values for both K_d and Q_d.

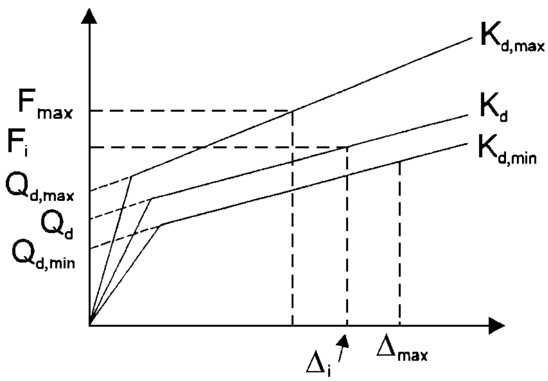


Figure C15.4-1 Impact Variations on Key Design Variables

The design forces on the columns and abutments generally will be at their maximum value when both K_d and Q_d are their maximum values. Therefore, an analysis is required using $Q_{d,max}$ and $K_{d,max}$ to determine the maximum forces that will occur on the substructures. The design displacements will be at their maximum value when both Q_d and K_d are at their minimum values. Therefore, an analysis is required using $Q_{d,min}$ and $K_{d,min}$ to determine the maximum displacements that will occur across the isolator units.

Using the design properties of the isolator units, Q_d and K_d (Figures C15.1-4 and C15.4-1), the design forces F_i and displacements Δ_i are first calculated with Equations 15.4.1-1, 15.4.1-2a, and 15.4.1-3. The design properties K_d and Q_d are then multiplied by $\lambda_{max,Kd}$, $\lambda_{max,Qd}$, $\lambda_{min,Kd}$, and $\lambda_{min,}Q_d$ as prescribed in Article 15.5.1.2 to obtain upper- and lower-bound values of K_d and Q_d. The analyses are then repeated using the upper-bound values, $K_{d,max}$ and $Q_{d,max}$ to determine F_{max}, and the lower-bound values $K_{d,min}$ and $Q_{d,min}$ to determine Δ_{max}. These upper- and lower-bound values account for all anticipated variations in the design properties of the isolation system resulting from temperature, aging, scragging, velocity, wear or travel, and contamination. The exception is that only one analysis is required using the design properties, provided that the maximum and minimum values of the forces and displacements are within ± 15 percent of the design values.

The λ_{max} and λ_{min} factors for each of the six variables are to be determined by the system characterization tests prescribed in Article 15.10.1, or the default values given in appendix 15A.

The prototype tests of Article 15.10.2 are required to validate the design properties of the isolation system. Prototype tests do not include any of the variables from the characterization tests that affect the design properties of the isolation system, because they are incorporated in the design process through the use of system property modification factors.

In order to provide guidance on some of the available systems, potential variations in the key parameters are as follows:

- Lead-Rubber Isolator Unit – The value of Qd is influenced primarily by the lead core. In cold temperatures, natural rubber will cause the most significant increase in Qd. The value of Kd depends on the properties of the rubber. Rubber properties are affected by aging, frequency of testing, strain, and temperature.

- High-Damping Rubber Isolator Unit – The value of Qd is a function of the additives to the rubber. The value of Kd is also a function of the additives to the rubber. High-damping rubber properties are affected by aging, frequency of testing, strain, temperature, and scragging.

- Friction Pendulum System® – The value of Qd is a function primarily of the dynamic coefficient of friction and axial load. The value of Kd is a function of the curvature of the sliding surface. The dynamic coefficient of friction is affected by aging, temperature, velocity of testing, contamination, and length of travel or wear.

- Eradiquake® – The value of Q_d is a function of the dynamic coefficient of the disc bearing and the preload friction force, when it is used. The value of K_d is a function of whatever springs are incorporated in the device. The dynamic coefficient of friction is affected by aging, temperature, velocity of testing, contamination, and length of travel or wear. The variations in spring properties depend on the materials used.

- Viscous Damping Devices – These can be used in conjunction with either elastomeric bearings or sliders. The value of Q_d is a function of both the viscous damper and the bearing element. The value of K_d is primarily a function of the bearing element.

C15.4.1 Capacity Spectrum Method

The capacity spectrum method of Article 4.4 and Article 5.4.1 is based on the same principles used in the original derivation of the simplified seismic isolation design approach. The only difference is the sequence in which it is applied. For non-isolated bridges, it is recommended that a designer sum the strength of the columns to obtain C_c and then determine if the displacement capacity of the columns is adequate using Equation 5.4.1-1. If not, the columns must be strengthened. In an isolation design the bridge achieves its single degree of freedom response characteristics by virtue of using flexible isolation bearings rather than having columns of very similar stiffness characteristics. The design procedure uses the stiffness characteristics of the isolation bearings sized to resist the non-seismic loads to determine the design displacement (Equation 15.4.1-3). The lateral force that the substructure must resist is then calculated using Equation 15.4.1-2 where K_{eff} is the sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure; and C_d is the lateral force demand coefficient. The derivation of the isolation design equations follows.

For the design of conventional bridges, the form of the elastic seismic demand coefficient in the longer period segment of the spectra is

$$C_d = \frac{F_v S_1}{T}$$

For seismic isolation design, the elastic seismic demand coefficient is directly related to the elastic ground-response spectra and damping of the isolation system.

$$C_d = \frac{F_v S_1}{T_{eff} B}$$

where B is the damping coefficient given in Table 15.4.1-1. Note that for 5 percent damping, $B = 1.0$. The quantity C_d is a dimensionless design coefficient, which when multiplied by g produces the spectral acceleration. This spectral acceleration (S_A) is related to the spectral displacement (S_D) by the relationship

$$S_A = \omega^2 S_D$$

where ω is the circular natural frequency and is given by $2\pi/T_{eff}$. Therefore, since $S_A = C_S \cdot g$

$$S_A = \frac{F_v S_1}{T_{eff} B} g$$

and

$$S_D = \frac{1}{\omega^2} \frac{F_v S_1}{T_{eff} B} g$$

$$= \frac{T_{eff}^2 F_v S_1}{(2\pi)^2 T_{eff} B}(9.81) \frac{\text{m}}{\text{sec}^2}; \frac{T_{eff}^2 F_v S_1}{(2\ )^2 T_{eff} B}(386.4) \frac{\text{inches}}{\text{sec}^2}$$

$$= \frac{0.249 F_v S_1 T_{eff}}{B} \text{m}; \frac{9.79 F_v S_1 T_{eff}}{B} \text{inches}$$

Denoting S_D as Δ (Article 15.4), which is the deck displacement relative to the ground, the above is approximated by

$$\Delta = \frac{0.25 F_v S_1 T_{eff}}{B} \text{m}; \frac{10 F_v S_1 T_{eff}}{B} \text{inches}$$

An alternate form for C_d is possible. The quantity C_d is defined by the relationship

$$F = C_d W$$

where F is the earthquake design force and W is the weight of the structure. Therefore,

$$C_d = \frac{F}{W} = \frac{K_{eff} \times \Delta}{W}$$

where K_{eff} is defined below. The equivalence of this form to the previous form is evident by observing that $K_{eff} = \omega^2 W/g$, from which

$$C_d = \frac{\omega^2 W}{g} \times \frac{\Delta}{W} = \frac{(2\pi)^2}{T_{eff}^2} \times \frac{1}{9.81} \times \frac{0.25 F_v S_1 T_{eff}}{B} = \frac{F_v S_1}{B T_{eff}};$$

$$C_d = \frac{\omega^2 W}{g} \times \frac{\Delta}{W} = \frac{(2\pi)^2}{T_{eff}^2} \times \frac{1}{386.4} \times \frac{9.79 F_v S_1 T_{eff}}{B}$$

In calculating the effective stiffness, the configuration, flexibility, and individual stiffnesses of the isolator units (k_{iso}) and substructure (k_{sub}) shall be taken into account:

$$K_{eff} = \sum_j \left(\frac{k_{sub} k_{iso}}{k_{sub} + k_{iso}} \right) = \sum_j K_{eff,j}$$

where the sum Σ extends over all substructures.

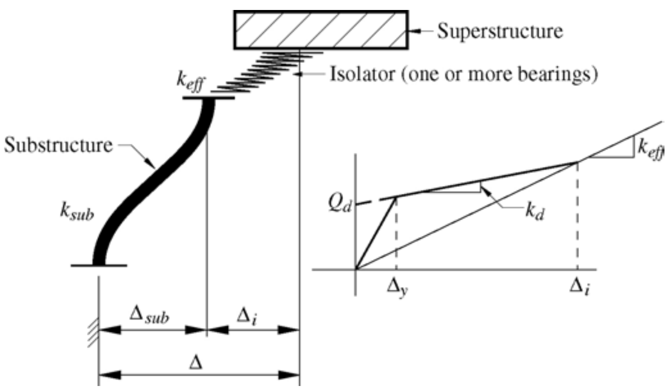


Figure C15.4.1-1 (figure shows only one isolator and one substructure)

The corresponding equivalent viscous damping may be calculated as follows:

$$\beta = \frac{\text{Energy Dissipated}}{2\pi K_{eff} \Delta^2} = \frac{\text{Total Dissipated Energy}}{2\pi \sum_j \left(K_{eff,j} \Delta^2 \right)}$$

$$\beta = \frac{2Q_d(\Delta_i - \Delta_y)}{\pi(\Delta_i + \Delta_{sub})^2 K_{eff}} = \frac{2\sum_j \left[Q_d (\Delta_i - \Delta_y) \right]}{\pi \sum_j \left[K_{eff,j} (\Delta_i + \Delta_{sub})^2 \right]}$$

Hysteretic Energy Dissipated at Isolator = $4Q_d(\Delta_i - \Delta_y)$

Note: These equations exclude contribution to damping from the substructure.

If damping is truly linear viscous, then damping coefficient in Table 15.4.1-1 may be extended to 50 percent (B =2).

If damping exceeds 30 percent, and a B of 1.7 is used, then a time-history analysis is not required.

Equations 15.4.1-1 and 15.4.1-2 are strictly applicable to hysteretic systems, that is, systems without added damping of truly viscous nature such as viscous dampers.

For systems with added viscous damping, as in the case of elastomeric or sliding systems with viscous dampers, Equations 15.4.1-3a and 15.4.1-3b are valid, provided that the damping coefficient B is based on the energy dissipated by all elements of the isolation system, including the viscous dampers. Equivalent damping shall be determined by Equation 15.10.3-2. The seismic force shall be determined in three distinct stages as follows:

1. At the stage of maximum bearing displacement. The seismic force shall be determined by Equation 15.4.1-1. Note that at this stage, the viscous damping forces are zero.

2. At the stage of maximum velocity and zero bearing displacement. The seismic force shall be determined as the combination of characteristic strength of the isolation bearings and the peak viscous damper force. The latter shall be determined at a velocity equal to $2\pi d_d/T_{eff}$, where d_d is the peak damper displacement. (Note that displacement d_d is related to bearing displacement Δ_i).

3. At the stage of maximum total inertia force (that is, superstructure acceleration). The seismic force shall be determined by

$$F = (f_1 + 2\,\beta_d\, f_2)\, C_d\, W$$

where C_d is determined by Equation 15.4.1-2; K_{eff} is determined from the contribution of all elements of the isolation system other than viscous dampers; β_d is the portion of the effective damping ratio of the isolated bridge contributed by the viscous dampers and

$$f_1 = \cos\,[\,\tan^{-1}\,(2\beta_d)]$$

$$f_2 = \sin\,[\,\tan^{-1}\,(2\beta_d)]$$

The modified equation provides an estimate of the maximum total inertia force on the bridge superstructure. The distribution of this force to elements of the substructure shall be based on bearing displacements equal to $f_1\Delta_i$, and substructure displacements equal to $f_1\Delta_{sub}$, and damper velocities equal to $f_2(2\pi d_d/T_{eff})$ where d_d is the peak damper displacement.

C15.4.2 Uniform Load Method

The uniform load method of analysis given in Article 5.4.2.2 is appropriate for seismic isolation design.

C15.4.3 Multimode Spectral Method

The guidelines given in Article 5.4.2.3 are appropriate for the response spectrum analysis of an isolated structure with the following modifications:

(a) The isolation bearings are modeled by use of their effective stiffness properties determined at the design displacement Δ_i (Figure C15.1-4).

(b) The ground response spectrum is modified to incorporate the effective damping of the isolated structure (Figure C15.1-5).

The response spectrum required for the analysis needs to be modified to incorporate the higher damping value of the isolation system. This modified portion of the response spectrum should only be used for the isolated modes of the bridge and will then have the form shown in figure C15.1-5.

The effective damping of the structure system shall be used in the multimode spectral analysis method. Structure system damping shall include all structural elements and be obtained by rational method as discussed in C15.4.1.

C15.4.4 Time-History Method

When a time-history analysis is required, the ground-motion time histories may be frequency scaled so they closely match the appropriate ground-response spectra for the site.

A two-dimensional nonlinear analysis may be used on normal structures without skews or curves.

C15.5 DESIGN PROPERTIES OF THE ISOLATION SYSTEM

C15.5.1 Nominal Design Properties

For an explanation of the system property modification factors concept, see Constantinou et al. (1999).

C15.5.2.1 Minimum and Maximum System Property Modification Factors

All λ_{min} values are unity at this time. The Task Group that developed these provisions determined that available test data for λ_{min} values would produce forces and displacements that are within 15 percent of the design values. If the engineer believes a particular system may produce displacements outside of the ±15-percent range, then a λ_{min} analysis should be performed.

C15.5.2.2 System Property Adjustment Factors

It is the opinion of the Task Group that developed these provisions that only operational bridges need to consider all maximum λ factors at the same time. The reduction factor for other bridges is based on engineering judgment.

Example:

$\lambda_{max,c}$ = 1.2 without adjustment factor

$\lambda_{max,c} = 1 + (1.2 - 1)\,0.67 = 1.13$ for adjustment factor of 0.67

C15.6 CLEARANCES

Adequate clearance shall be provided for the displacements resulting from the seismic isolation analysis in either of the two orthogonal directions. As a design alternate in the longitudinal direction, a knock-off abutment detail (Figure C3.3.5) may be provided for the seismic displacements between the abutment and deck slab. Adequate clearance for the seismic displacement must be provided between the girders and the abutment. In addition, the design rotation capacity of the bearing shall exceed the maximum seismic rotation.

The purpose of the minimum clearance default value is to guard against analysis procedures that produce excessively low clearances.

Displacements in the isolators resulting from longitudinal forces, wind loads, centrifugal forces, and thermal effects will be a function of the force-deflection characteristics of the isolators. Adequate clearance at all expansion joints must be provided for these movements.

C15.7 DESIGN FORCES FOR SDAP A1 AND A2

This section permits utilization of the real elastic force reduction provided by seismic isolation. It should be noted, however, that F_vS_1 has a maximum value of 0.25 for SDAP A bridges and is specified to have a minimum value of 0.25 if seismic isolation is used.

C15.9 OTHER REQUIREMENTS

C15.9.1 Non-Seismic Lateral Forces

Since an element of flexibility is an essential part of an isolation system, it is also important that the isolation system provide sufficient rigidity to resist frequently occurring wind and other service loads. The displacements resulting from non-seismic loads need to be checked.

C15.9.1.2 Cold Weather Requirements

Low temperatures increase the coefficient of friction on sliding systems and the shear modulus and characteristic strength of elastomeric systems. These changes increase the effective stiffness of the isolation system.

The test temperatures used to determine low-temperature performance in Article 15.10.1 represent 75 percent of the difference between the base temperature and the extreme temperature in Table 14.7.5.2-2.

C15.9.2 Lateral Restoring Force

The basic premise of these seismic isolation design provisions is that the energy dissipation of the system can be expressed in terms of equivalent viscous damping and the stiffness by an effective linear stiffness. The requirement of this section provides the basis for which this criteria is met.

The purpose for the lateral restoring force requirement is to prevent permanent cumulative displacements and to accommodate isolator installation imperfections, such as out of level.

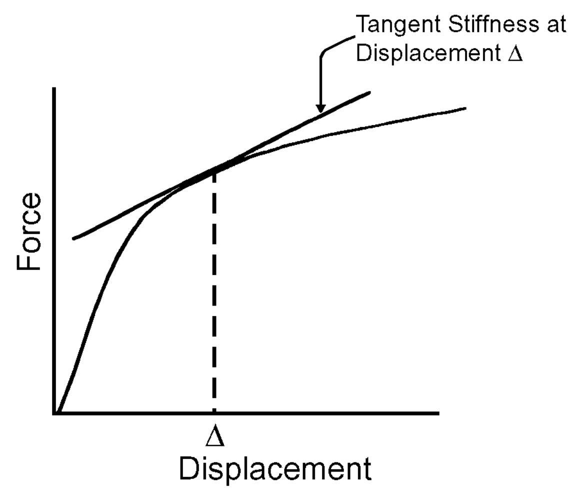


Figure C15.9.2-1 Tangent Stiffness of Isolation System

The lateral restoring force requirements are applicable to systems with restoring force that is dependent on displacement, that is, spring-like restoring force. However, it is possible to provide constant restoring force that is independent of displacement. There are two known means for providing constant restoring force: (a) using compressible fluid springs with preload and (b) using sliding bearings with a conical surface. Figure C15.9.2-2 illustrates a typical force-displacement relation of these devices.

The requirement for lateral restoring force in these cases is that the combined constant lateral restoring force of the isolation system is at least equal to 1.05 times the combined characteristic strength of the isolation system under service conditions. For example, when constant restoring force devices are combined with frictional elements (e.g., sliding bearings), the restoring force must be at least equal to 1.05 times the static friction force. This requirement ensures that the restoring force is sufficiently large to overcome the characteristic strength and, thus, provide re-centering capability.

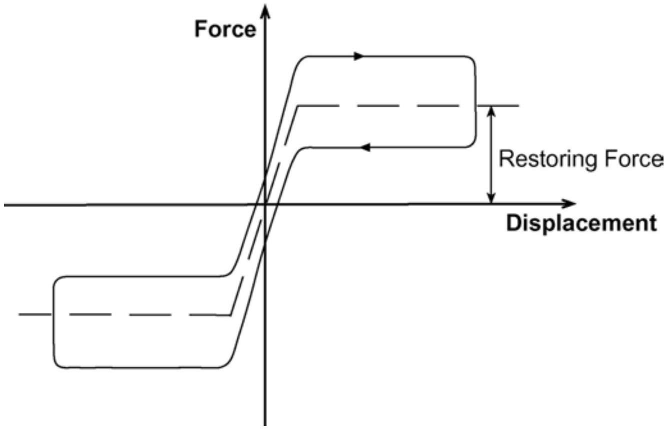


Figure C15.9.2-2 Force-Displacement Relation of Systems with Constant Restoring Force

C15.9.3 Vertical Load Stability

This section provides minimum requirements for the design of the isolation system. The detailed design requirements of the system will be dependent on the type of system. The 1.2 factor accounts for vertical acceleration effects and uncertainty in the dead load.

C15.9.4 Rotational Capacity

Larger construction rotations may be allowed, provided that they do not damage the isolator unit.

C15.10 REQUIRED TESTS OF ISOLATION SYSTEMS

The code requirements are predicated on the fact that the isolation system design is based on tested properties of isolator units. This section provides a comprehensive set of prototype tests to confirm the adequacy of the isolator properties used in the design. Systems that have been previously tested with this specific set of tests on similar type and size of isolator units do not need to have these tests repeated. Design properties must therefore be based on manufacturers' preapproved or certified test data. Extrapolation of design properties from tests of similar type and size of isolator units is permissible.

Isolator units used for the system characterization tests (except shaking table), prototype tests, and quality control tests shall have been manufactured by the same manufacturer with the same materials.

C15.10.1 System Characterization Tests

These tests are usually not project specific. They are conducted to establish the fundamental properties of individual isolator units as well as the behavior of an isolation system. They are normally conducted when a new isolation system or isolator unit is being developed or a substantially different version of an existing isolation system or isolator unit is being evaluated.

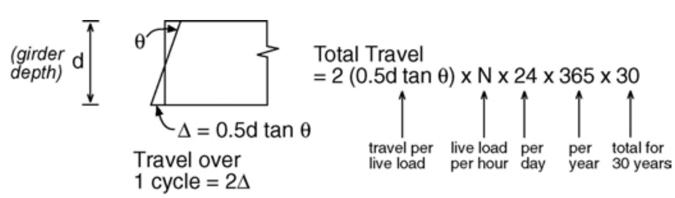
Several guidelines for these tests have been developed. The NIST Guidelines are currently being developed into the ASCE Standard for Testing Seismic Isolation Systems, Units, and Components. This new standard currently exists in draft form. Testing guidelines have also been developed and used for the HITEC evaluation of seismic isolation and energy dissipation devices.

C15.10.1.1 Low-Temperature Test

The test temperatures represent 75 percent of the difference between the base temperature and the extreme temperature in Table 14.7.5.2-2. Prior to testing, the core temperature of the isolator unit shall reach the specified temperature.

C15.10.1.2 Wear and Fatigue Tests

The movement that is expected from live load rotations is dependent on structure type, span length and configuration, girder depth, and average daily traffic. The total movement resulting from live load rotations can be calculated as follows:



Test 1, Thermal – This test verifies the lateral force exerted by the isolation system at maximum thermal displacement.

Test 2, Wind and Braking – This test verifies the resistance of the isolation system under service load conditions.

Test 3, Seismic – This test verifies the dynamic response of the isolation system for various displacements.

The sequence of fully reversed cycles is important to developing hysteresis loops at varying displacements. By starting with a multiple of 1.0 times the total design displacement, the performance of the unscragged and scragged bearing may be directly compared.

Test 4, Seismic – This verifies the survivability of the isolator after a major earthquake. The test is started from a displaced position to reflect the uncertainty of the starting position when an earthquake occurs. The seismic displacements shall be superimposed on the offset load displacement so that the peak displacements will be asymmetric.

Test 5, Wind and Braking – This test verifies service load performance after a seismic event.

Test 6, Seismic Performance Verification – The seismic performance verification test verifies the performance of the bearing after the sequence of tests has been completed.

Test 7, Stability Verification – Stability is demonstrated if the isolator shows a positive incremental force carrying capacity satisfying the requirements of Article 15-4.

An isolation system needs a positive incremental force-carrying capability to satisfy the requirements of Article 15.9.2. The purpose of this requirement is to ensure that the hysteretic elements of the system are stable. A viscous damper will have a negative incremental force-carrying capacity toward the point of maximum displacement. Since this is acceptable performance, it needs to be deleted from the other components prior to their stability evaluation.

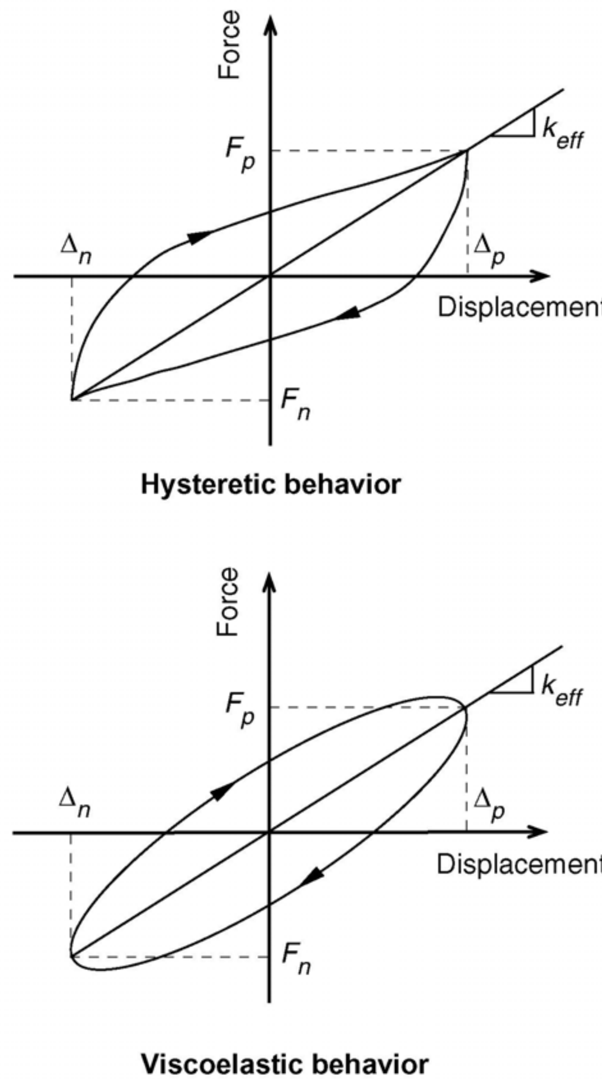


Figure C15.10.3-1 Definition of Effective Stiffness

C15.10.3.1 System Adequacy

For Test 4, If the change in effective stiffness is greater than 20 percent, the minimum effective stiffness value should be used to calculate the system displacements, and the maximum effective stiffness values should be used to calculate the structure and isolation system forces.

A decrease in stiffness during cyclic testing may occur in some systems and is considered acceptable if the degradation is recoverable within a time frame acceptable to the engineer. That is, the bearing will return to its original stiffness after a waiting period.

For Test 4, A decrease in EDC during cyclic testing may occur in some systems and is considered acceptable if the degradation is recoverable within a time frame acceptable to the engineer.

At the conclusion of testing, the test specimens shall be externally inspected or, if applicable, disassembled and inspected for the following faults, which shall be cause for rejection:

(1) Lack of rubber-to-steel bond.
(2) Laminate placement fault.
(3) Surface cracks on rubber that are wider or deeper than 2/3 of the rubber cover thickness.
(4) Material peeling.
(5) Lack of polytetrafluorethyene(PTFE)-to-metal bond.
(6) Scoring of stainless steel plate.
(7) Permanent deformation.
(8) Leakage.

C15.11 ELASTOMERIC BEARINGS

Elastomeric bearings used for seismic isolation will be subjected to earthquake-induced displacements (Δ_i) and must therefore be designed to safely carry the vertical loads at these displacements. Since earthquakes are infrequently occurring events, the factors of safety required under these circumstances will be different from those required for more frequently occurring loads.

Since the primary design parameter for earthquake loading is the displacement (Δ_i) of the bearing, the design procedures must be capable of incorporating this displacement in a logical, consistent manner. The requirements of Article 14.7.5.3 limit vertical loads by use of a limiting compressive stress, and therefore do not have a mechanism for including the simultaneous effects of seismic displacements. The shear displacement is also limited to half of the elastomer thickness. The British specifications BE 1/76 and BS 5400 recognize that shear strains are induced in reinforced bearings by compression, rotation, and shear deformations. In BE 1/76, the sum of these shear strains is limited to a proportion of the elongation-at-break of the rubber. The proportion (1/2 or 1/3 for service load combinations and 3/4 for seismic load combinations) is a function of the loading type. In BS 5400 and the 1995 draft Eurocode EN 1337, the limit is a constant 5.0.

Since the approach used in BE 1/76 and BS 5400 incorporates shear deformation as part of the design criteria, it can be readily modified for seismic isolation bearings. The design requirements given are based on the appropriate modifications to BE 1/76 and BS 5400.

In the extensive testing conducted for NCHRP Report No. 298 (Roeder, Stanton, and Taylor 1987), no correlation was found between the elongation-at- break and the ability of the elastomers to resist shearing strain without debonding from the steel reinforcement. Furthermore, the French code UIC772R and the BS 5400 also imply no dependence on ε_u, but rather use a single limit of 5.0 for the sum of the strains, regardless of the elastomer type.

C15.11.2 Shear Strain Components for Isolation Design

The allowable vertical load on an elastomeric bearing is not specified explicitly. The limits on vertical load are governed indirectly by limitations on the equivalent shear strain in the rubber due to different load combinations and to stability requirements.

The effects of creep of the elastomer shall be added to the instantaneous compressive deflection, when considering long-term deflections. They are not to be included in the calculation of Article 15.11.3. Long-term deflections shall be computed from information relevant to the elastomer compound used, if it is available. If not, the values given in Article 14.7.5.3.3.

For incompressible isotropic material $E = 3G$, however, this is not true for rubber. For rubber, $E = (3.8 \text{ to } 4.4)G$ depending on its hardness, which indicates anisotropy in rubber. Accordingly, Equation 15.11.2-1 is based on Equation 8 of the 1991 AASHTO Guide Specifications with E replaced by $4G$. It should be noted that the quantity $4G\ (1 + 2\ \bar{k}S^2)$ is the compression modulus of the bearing, as calculated on the assumption of incompressible rubber. For bearings with large shape factors, the assumption of incompressible rubber leads to significant overestimation of the compression modulus and, thus, underestimation of the shear strain due to compression. Equation 15.11.2-2 is introduced to account for the effects of rubber compressibility. It is based on the empirical relation that the compression modulus is given by $[1/(8G\ kS^2) + 1/K]^{-1}$.

The shear modulus (G) shall be determined from the secant modulus between 25- and 75-percent shear strain in accordance with ASTM D 4014, published by the American Society of Testing and Materials.

The design rotation is the maximum rotation of the top surface of the bearing relative to the bottom surface. Any negative rotation due to camber will counteract the DL and LL rotation and should be included in the calculation

C15.11.3 Load Combinations

Tests for NCHRP at the University of Washington, Seattle, have shown that static rotation is significantly less damaging than dynamic rotation.

C15.13.1 General

The sliding bearing is typically made from two dissimilar materials that slide against each other. Low friction is achieved when a softer material, usually PTFE and herein called the bearing liner, slides against a hard, smooth surface that is usually stainless steel and is herein called the mating surface. Lubrication may be used.

The restoring force may be provided either by gravity acting through a curved sliding surface or by a separate device such as a spring.

C15.13.2 Materials

Certain combinations of materials have been found to promote severe corrosion and are strongly discouraged (British Standards Institution 1979; 1983). Examples are

- structural steel and brass,
- structural steel and bronze,
- structural steel and copper,
- structural steel and aluminum, and
- chromium on structural steel (chrome plating of steel).

Chrome is porous, so structural steel is exposed to oxygen.

Other combinations of materials known to promote additional but not severe corrosion are

- stainless steel and brass,
- stainless steel and bronze, and
- stainless steel and copper.

C15.13.2.3 Mating Surface

Higher grades of stainless steel such as type 316, conforming to ASTM A 240, should be considered for applications in severe corrosive environments.

Measurements of surface roughness need to be reported together with information on profilometer stylus tip radius, traversing length and instrument cutoff length. It is recommended that the stylus tip radius not be more than 200 micro inches (5 micro meters) and the cutoff length be 0.03 inches (0.8 mm).

Table 15.13.4.1-1 Allowable Average Contact Stresses for PTFE

The rotation-induced edge stresses must be calculated by a rational method that accounts for the rotational stiffness and rotational demand of the bearing.

Table 15.13.4.2.1-1 Service Coefficients of Friction

Service coefficients of friction for various types of PTFE were determined at a test speed of 2.5 inches/min (63.5 mm/min) on a mirror finish (no. 8) stainless steel mating surface with scaled samples (Stanton, Roeder, and Campbell 1993).

C15.13.4.2.2 Seismic Coefficient of Friction

Typically the maximum seismic coefficient of friction for PTFE based material is reached at a testing velocity of 2 to 8 inches/sec (50 to 200 mm/sec).

C15.15.1 Scope

This chapter is intended to cover new isolation systems that are not addressed in the preceding chapters.

C15.15.2 System Characterization Tests

The purpose of these tests is to demonstrate that the principles on which the system is intended to function are realized in practice. The number and details of the test must be approved by the engineer.

C15.15.4 Fabrication, Installation, Inspection, and Maintenance Requirements

The maintenance requirements must be known at the time of submission of the design procedure in order that the engineer may assess their impact on the reliability and life-cycle costs of the system.

C15.15.5 Prototype Tests

The purpose of the prototype testing is to verify that the as-built bearing system satisfies the design requirements for the particular size and configuration used in the job in question.