

Chapter 6 Commentary

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENT DESIGN REQUIREMENTS

6.1 GENERAL

6.1.1 Scope. The general requirements establish minimum design levels for architectural, mechanical, electrical, and other nonstructural systems and components recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical, electrical, and other nonstructural components. Several classes of components are not subject to the *Provisions*:

1. All components in Seismic Design Category A are exempted because of the lower seismic input for these items.
2. All mechanical and electrical components in Seismic Design Categories B and C are exempted if they have an importance factor (I_p) equal to 1.00 because of the low acceleration and the classification that they do not contain hazardous substances and are not required to function to maintain life safety.
3. All components in all Seismic Design Categories, weighing less than 400 pounds (1780 N), and mounted 4 ft (1.22 m) or less above the floor are exempted if they have an importance factor (I_p) equal to 1.00, because they do not contain hazardous substances, are not required to function to maintain life safety, and are not considered to be mounted high enough to be a life-safety hazard if they fall.

Storage racks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.3.5.

Storage tanks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.4.7.

When performing seismic design of nonstructural components, be aware that there may be important non-seismic requirements outside the scope of the building code, that may be affected by seismic bracing. For example, thermal expansion is often a critical design consideration in pressure piping systems, and bracing must be arranged in a manner that accommodates thermal movements. The design for seismic loads should not compromise the functionality, durability, or safety of the overall system, and this may require substantial collaboration and cooperation between the various disciplines in the design team. In some cases, such as essential facilities or hazardous environments, it may be appropriate to consider performance levels higher than what is required by the building code (for example, operability of a piping system, rather than leak tightness).

For some components, such as exterior walls, the wind design forces may be higher than the seismic design forces. Even when this occurs, the seismic detailing requirements may still govern the overall structural design. Whenever this is a possibility, it should be investigated early in the structural design process.

6.2 GENERAL DESIGN REQUIREMENTS

6.2.2 Component importance factor. The component importance factor (I_p) represents the greater of the life-safety importance of the component and the hazard-exposure importance of the structure. This

factor indirectly accounts for the functionality of the component or structure by requiring design for a higher force level. Use of higher I_p requirements together with application of the requirements in Sec. 6.4.2 and 6.4.3 should provide better, more functional component. While this approach will provide a higher degree of confidence in the probable seismic performance of a component, it may not be sufficient for all components. For example, individual ceiling tiles may still fall from the ceiling grid. Seismic qualification approaches presently in use by the Department of Energy (DOE) and the Nuclear Regulatory Commission (NRC) should be considered by the registered design professional and/or the owner when the consequences of failure would be unacceptable.

Components that could fall from the structure are among the most hazardous building components in an earthquake. These components may not be integral with the structural system and may cantilever horizontally or vertically from their supports. Critical issues affecting these components include their weight, their attachment to the structure, their breakage characteristics (glass) and their location (over an entry or exit, public walkway, atrium, or lower adjacent structure). Examples of items that may pose a falling hazard include parapets, cornices, canopies, marquees, glass, and precast concrete cladding panels. In addition, mechanical and electrical components may pose a falling hazard (for example, a rooftop tank or cooling tower, which if separated from the structure would fall to the ground).

Special consideration should be given to components that could block means of egress or exitways if they were to fall during an earthquake. The term “means of egress” has been defined in the same way throughout the country, since egress requirements have been included in building codes because of fire hazard. The requirements for exitways include intervening aisles, doors, doorways, gates, corridors, exterior exit balconies, ramps, stairways, pressurized enclosures, horizontal exits, exit passageways, exit courts, and yards. Example items that should be included when considering egress include walls around stairs and corridors, and veneers, cornices, canopies, and other ornaments above building exits. In addition, heavy partition systems vulnerable to failure by collapse, ceilings, soffits, light fixtures, or other objects that could fall or obstruct a required exit door or component (rescue window or fire escape) could be considered major obstructions. Examples of components that do not pose a significant falling hazard include fabric awnings and canopies and architectural, mechanical, and electrical components which, if separated from the structure, will fall in areas that are not accessible (in an atrium or light well not accessible to the public, for instance).

In Sec. 1.2.1 the intent is that Group III structures shall, in so far as practical, be provided with the capacity to function after an earthquake. To facilitate this, all nonstructural components and equipment in structures in Seismic Use Group III, and in Seismic Design Category C or higher, should be designed with an I_p equal to 1.5. All components and equipment are included because damage to vulnerable unbraced systems or equipment may disrupt operations following an earthquake, even if they are not “life-safety” items. Nonessential items can be considered “black boxes.” There is no need for component analysis as discussed in Sec. 6.4.2 and 6.4.3, since operation of these secondary items is not critical to the post-earthquake operability of the structure. Instead, the design may focus on their supports and attachments.

6.2.3 Consequential damage. Although the components included in Tables 6.3-1 and 6.4-1 are listed separately, significant interrelationships exist among them and should not be overlooked. For example, exterior, nonstructural, spandrel walls may shatter and fall on the streets or walks below, seriously hampering accessibility and egress functions. Further, the rupture of one component could lead to the failure of another that is dependent on the first. Accordingly, the collapse of a single component ultimately may lead to the failure of an entire system. Widespread collapse of suspended ceilings and light fixtures in a building may render an important space or major exit stairway unusable.

Consideration also was given to the design requirements for these components to determine how well they are conceived for their intended functions. Potential beneficial and/or detrimental interactions with the structure were examined. The interrelationship between components and their attachments were surveyed. Attention was given to the performance relative to each other of architectural, mechanical,

and electrical components; building products and finish materials; and systems within and without the building structure. It should be noted that the modification of one component in Table 6.3-1 or 6.4-1 could affect another and, in some cases, such a modification could help reduce the risk associated with the interrelated unit. For example, landscaping barriers around the exterior of certain buildings could decrease the risk due to falling debris although this should not be interpreted to mean that all buildings must have such barriers.

The design of components that are in contact with or in close proximity to structural or other nonstructural components must be given special study to avoid damage or failure when seismic motion occurs. An example is where an important element, such as a motor generator unit for a hospital, is adjacent to a non-load-bearing partition. The failure of the partition might jeopardize the motor generator unit and, therefore, the wall should be designed for a performance level sufficient to ensure its stability.

Where nonstructural wall components may affect or stiffen the structural system because of their close proximity, care must be exercised in selecting the wall materials and in designing the intersection details to ensure the desired performance of each component.

6.2.4 Flexibility. In the design and evaluation of support structures and the attachment of architectural components, flexibility should be considered. Components that are subjected to seismic relative displacements (that is, components that are connected to both the floor and ceiling level above) should be designed with adequate flexibility to accommodate imposed displacements. This is covered in Sec. 6.2.7. In the design and evaluation of equipment support structures and attachments, flexibility will reduce the fundamental frequency of the supported equipment and increase the amplitude of its induced relative motion. This lowering of the fundamental frequency of the supported component often will bring it into the range of the fundamental frequency of the supporting building or into the high energy range of the input motion. In evaluating the flexibility/stiffness of the component attachment, the effects of flexibility in the load path of the components should be considered especially in the region near the anchor points.

6.2.5 Component force transfer. It is required that components be attached to the structure and that all the required attachments be fully detailed in the design documents, or be specified in accordance with approved standards. These details should take into account the force levels and anticipated deformations expected or designed into the structure. For the purposes of the load path check, it is essential that detailed information concerning the components, including size, weight, and location of component anchors, be communicated to the registered design professional responsible for the structure during the design process.

The calculation of forces as prescribed in Sec. 6.2.6 recognizes the unique dynamic and structural characteristics of the components as compared to structures. Components typically lack the desirable attributes of structures (such as ductility, toughness, and redundancy) that permit the use of greatly reduced lateral design forces. This is reflected in the lower values for R_p given in Tables 6.3-1 and 6.4-1, as compared to R values for structures. In addition, components may exhibit unique dynamic amplification characteristics, as reflected in the values for a_p in Tables 6.3-1 and 6.4-1. Thus, for the calculation of the component integrity and connection to the supporting structure, greater forces are used, as a percentage of component mass, than are typically calculated for the overall seismic-force-resisting system. It is the intent of this provision that component forces be accommodated in the design of the structure as required to prevent local overstress of the immediate vertical and lateral load-carrying systems. Inasmuch as the component masses are included, explicitly or otherwise, in the design of the seismic-force-resisting system, it is generally sufficient for verification of a complete load path to check only for local overstress conditions in the vicinity of the component in question. One approach to achieve this is to check the capacity of the first structural element in the load path (for example, the floor beam directly under a component) for combined dead, live, operating, and seismic loads, using the horizontal and vertical loads from Sec. 6.2.6 for the seismic demand. This procedure is repeated for

each structural element or connection in the load path until the load case including horizontal and vertical loads from Sec. 6.2.6 no longer governs the design of the element. This will occur when the component design loads generated by Sec. 6.2.6 become small relative to the dead and live load demands on the structural element. Where component forces have increased due to the nature of the anchorage system, these load increases, which take the form of reductions in R_p , or increases in F_p , need not be considered in the check of the load path.

An area of concern that is often overlooked is the reinforcement and positive connection of housekeeping slabs to the supporting structure. Lack of such reinforcement and connections has led to costly failures in past earthquakes. Therefore, the housekeeping slabs must be considered as part of the continuous load path be adequately reinforced, and be positively fastened to the supporting structure.

The exact size and location of loads might not be known until the component is ordered. Therefore, the designer should make conservative assumptions in the design of the supporting structural elements. The design of the supporting structural elements must be checked once the final magnitude and location of the design loads have been established.

If an architectural component were to fail during an earthquake, the mode of failure probably would be related to faulty design of the component, interrelationship with another component that fails, interaction with the structural framing, deficiencies in its type of mounting, or inadequacy of its attachments or anchorage. The last is perhaps the most critical when considering seismic safety.

Building components designed without any intended structural function—such as infill walls—may interact with the structural framing and be forced to act structurally as a result of excessive building deformation. The build up of stress at the connecting surfaces or joints may exceed the limits of the materials. Spatial tolerances between such components thus become a governing factor. These requirements therefore emphasize the ductility and strength of the attachments for exterior wall elements and the interrelationship of elements.

Traditionally, mechanical equipment that does not include rotating or reciprocating components (such as tanks and heat exchangers) is anchored directly to the building structure. Mechanical and electrical equipment containing rotating or reciprocating components often is isolated from the structure by vibration isolators (such as rubber-in-shear, springs, or air cushions). Heavy mechanical equipment (such as large boilers) often is not restrained at all, and electrical equipment other than generators, which are normally isolated to dampen vibrations, usually is rigidly anchored (for example, switchgear and motor control centers). The installation of unattached mechanical and electrical equipment should be virtually eliminated for buildings covered by the *Provisions*.

Friction produced solely by the effects of gravity cannot be counted on to resist seismic forces as equipment and fixtures often tend to “walk” due to rocking when subjected to earthquake motions. This often is accentuated by vertical ground motions. Because such frictional resistance cannot be relied upon, positive restraint must be provided for each component.

6.2.6 Seismic forces. The design seismic force is dependent upon the weight of the system or component, the component amplification factor, the component acceleration at point of attachment to the structure, the component importance factor, and the component response modification factor.

The seismic design force equations presented originated with a study and workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) with funding from the National Science Foundation (NSF) (Bachman et al., 1993). The participants examined recorded acceleration data in response to strong earthquake motions. The objective was to develop a “supportable” design force equation that considered actual earthquake data as well as component location in the structure, component anchorage ductility, component importance, component safety hazard upon separation from the structure, structural response, site conditions, and seismic zone. Additional studies have further revised the equation to its present form (Drake and Bachman, 1994 and 1995). In addition, the term C_a has been replaced by the quantity $0.4S_{DS}$ to conform to changes in Chapter 3. BSSC Technical

Subcommittee 8 believes that Eq. 6.2-1, 6.2-3, and 6.2-4 achieve the objectives without unduly burdening the practitioner with complicated formulations.

The component amplification factor (a_p) represents the dynamic amplification of the component relative to the fundamental period of the structure (T). It is recognized that at the time the components are designed or selected, the structural fundamental period is not always defined or readily available. It is also recognized that the component fundamental period (T_p) is usually only accurately obtained by expensive shake-table or pull-back tests. A listing is provided of a_p values based on the expectation that the component will usually behave in either a rigid or flexible manner. In general, if the fundamental period of the component is less than 0.06 sec, no dynamic amplification is expected. It is not the intention of the *Provisions* to preclude more accurate determination of the component amplification factor when reasonably accurate values of both the structural and component fundamental periods are available. Figure C6.2-1 is from the NCEER work and is an acceptable formulation for a_p as a function of T_p/T . Minor adjustments in the tabulated a_p values were made in the 1997 Edition to be consistent with the 1997 *Uniform Building Code*.

The component response modification factor (R_p) represents the energy absorption capability of the component's structure and attachments. Conceptually, the R_p value considers both the overstrength and deformability of the component's structure and attachments. In the absence of current research, it is believed these separate considerations can be adequately combined into a single factor. The engineering community is encouraged to address the issue and conduct research into the component response modification factor that will advance the state of the art. These values are judgmentally determined utilizing the collective wisdom and experience of the responsible committee. In general, the following benchmark values were used:

$R_p = 1.5$, low deformability element

$R_p = 2.5$, limited deformability element

$R_p = 3.5$, high deformability element

Minor adjustments in the tabulated R_p values were made in the 1997 Edition to correlate with F_p values determined in accordance with the 1997 *Uniform Building Code*. Researchers have proposed a procedure for validating values for R_p with respect to documented earthquake performance (Bachman and Drake, 1996).

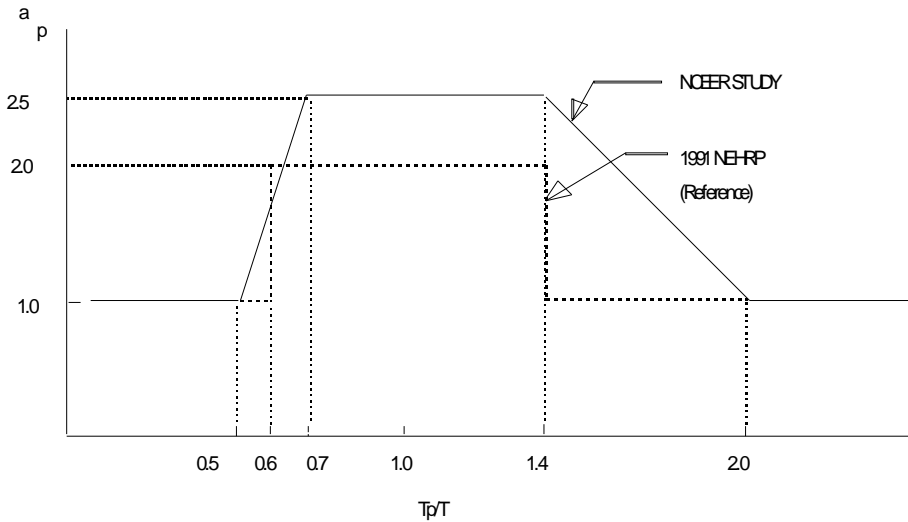


Figure C6.2-1 NCEER formulation for a_p as function of structural and component periods

Eq. 6.2-1 represents a trapezoidal distribution of floor accelerations within the structure, linearly varying from the acceleration at the ground ($0.4S_{DS}$) to the acceleration at the roof ($1.2S_{DS}$). The ground acceleration ($0.4S_{DS}$) is intended to be the same acceleration used as design input for the structure itself and includes site effects.

Examination of recorded in-structure acceleration data in response to large California earthquakes reveals that a reasonable maximum value for the roof acceleration is four times the input ground acceleration to the structure. Earlier work (Drake and Bachman, 1996, 1995 and 1996) indicated that the maximum amplification factor of four seems suitable (Figure C6.2-2). However, a close examination of recently recorded strong motion data at sites with peak ground accelerations in excess of $0.1g$ indicates that an amplification factor of three is more appropriate (Figure C6.2-3). In the lower portions of the structure (the lowest 20 percent of the structure), both the amplification factors of three and four do not bound the mean plus one standard deviation accelerations. However, the minimum design force in Eq. 6.2-4 provides a lower bound in this region.

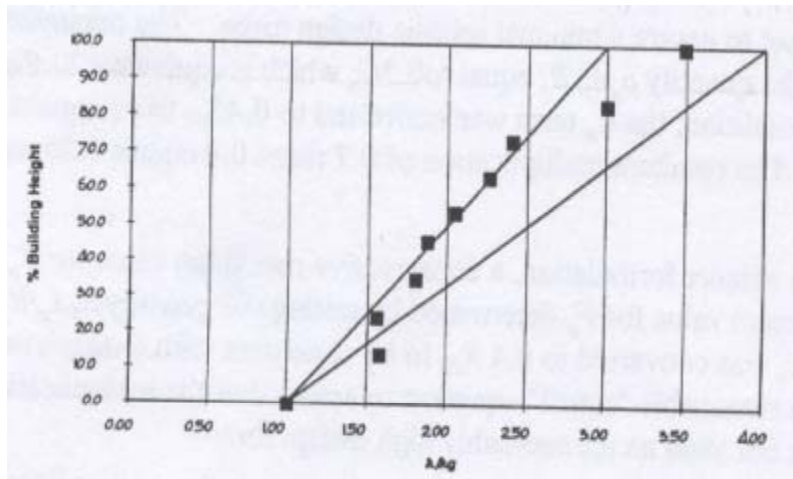


Figure C6.2-2 Revised NEHRP equation vs (Mean + 1σ) acceleration records -all sites

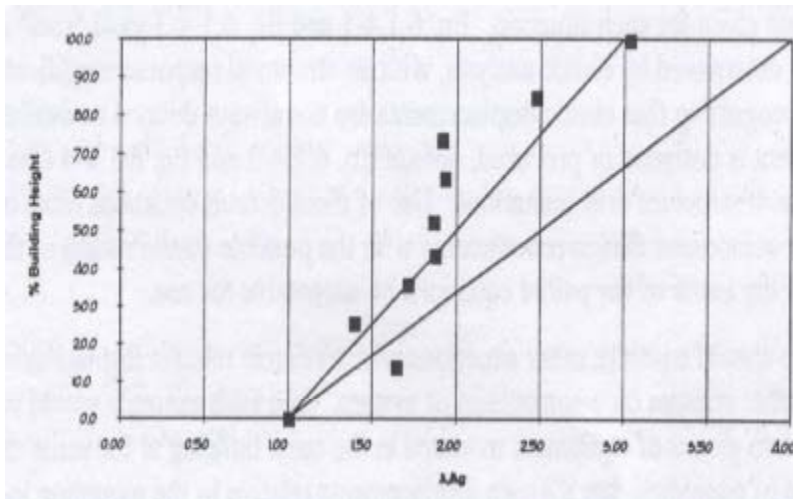


Figure C6.2-3 Revised NEHRP equation vs (Mean + 1σ) acceleration records - sites with $A_g \geq 0.1g$

At periods greater than T_s (where $T_s = S_{D1}/S_{DS}$), the acceleration response of structures ground reduces because the design ground motion acceleration response spectra beyond T_s starts to reduce by the ratio of T_s/T (where T is the fundamental period of the primary structure). Since this reduction in the design forces for the primary structure is accounted for in design force equations, it is justifiable to make a similar type of reduction in the design forces of non-structural components. However, in observing the actual in-structure response spectra of acceleration recordings measured at the roof levels of buildings with a range of fundamental periods, this reduction in response typically begins at periods about 25 percent greater than T_s . Therefore, the transition period, T_{flx} at the top of the primary structure at which forces begin reducing by a ratio of T_p/T has been lengthened by 25 percent to account for this observation. At the ground level ($z = 0$) the adjustment is 0 percent since the effect of the structure response has no influence on the non-structural component response. A linear interpolation is used between the top and bottom of the structure.

A lower limit for F_p is set to assure a minimal seismic design force. The minimum value for F_p was determined by setting the quantity $a_p A_p / R_p$ equal $0.3S_{DS}$ for consistency with current practice.

To meet the need for a simpler formulation, a conservative maximum value for F_p also was set. Eq. 6.2-3 is the maximum value for F_p determined by setting the quantity $a_p A_p / R_p$ equal to 4.0. Eq. 6.2-3 also serves as a reasonable “cutoff” equation to assure that the multiplication of the individual factors does not yield an unreasonably high design force.

To clarify the application of vertical seismic design forces in combination with horizontal design forces and service loads, a cross-reference was provided to Sec. 4.2.2. The value for F_p calculated in accordance with Chapter 6 should be substituted for the value of Q_E in Sec. 4.2.2.

For elements with points of attachment at varying heights, it is recommended that F_p be determined individually at each height (including minima) and the values averaged.

Alternatively for each point of attachment a force F_p shall be determined based on Eq. 6.2-1. Minima and maxima in Sec. 6.2.6 must be utilized in determining each F_p . The weight W_p used in determining each F_p should be based on the tributary weight of the component associated with the point of attachment. For designing the component, the attachment force F_p should be distributed relative to the component’s mass distribution over the area used to establish the tributary weight (in the instance of tilt-up walls, a uniform horizontal load would be applied half-way up the wall equal to F_p min.). With the exception of out-of-plane wall anchorage to flexible diaphragms, which is covered by Eq. 4.6-1, each anchorage force should be based on simple statistics determined using all the distributed loads applied to the complete component. Cantilever parapets that are part of a continuous element should be separately checked for parapet forces.

The seismic force on any component must be applied at the center of gravity of the component and must be assumed to act in any horizontal direction. Vertical forces on nonstructural components are specified in Sec. 6.2.6.

6.2.7 Seismic relative displacements. The seismic relative displacement equations were developed as part of the NCEER/NSF study and workshop described above. It was recognized that displacement equations were needed for use in the design of cladding, stairwells, windows, piping systems, sprinkler components, and other components that are connected to the structure(s) at multiple levels or points of connection.

Two equations are given for each situation. Eq. 6.2-5 and Eq. 6.2-7 produce “real” structural displacements as determined by elastic analysis, with no structural response modification factor (R) included. Recognizing that elastic displacements are not always defined or available at the time the component is designed or procured, default Eq. 6.2-6 and Eq. 6.2-8, which allow the use of structure drift limitations, also are provided. Use of these default equations must balance the need for a timely component design/procurement with the possible conservatism of their use. It is the intention that the lesser of the paired equations be acceptable for use.

The designer also should consider other situations where seismic relative displacements could impose unacceptable stresses on a component or system. One such example would be a component connecting two pieces of equipment mounted in the same building at the same elevation, where each piece of equipment has its own displacements relative to the mounting location. In this case, the designer must accommodate the total of the separate seismic displacements relative to the equipment mounting location. The height over which D_p , the displacement demand, must be accommodated is often less than the story height h_{sx} , and should be carefully considered. For example, a glazing system sandwiched between two rigid precast concrete spandrel panels may need to accommodate the entire displacement demand in less than 1/3 of the story height. Similar demands can occur when pipes, ducts and conduit that are connected to the top of a tall component are braced to the floor or roof above.

For some items, such as ductile piping, relative seismic displacements between support points generally are of more significance than forces. Piping made of ductile materials such as steel or copper can accommodate relative displacements by local yielding but with strain accumulations well below failure levels. However, components made of less ductile materials can only accommodate relative

displacement effects by use of flexible connections, avoiding local yielding. Further, it is the intent of the *Provisions* to consider the effects of seismic support relative displacements and displacements caused by seismic forces on mechanical and electrical component assemblies such as piping systems, cable and conduit systems, and other linear systems, and the equipment to which they attach. Impact of components should also be avoided although ductile materials have been shown to be capable of accommodating fairly significant impact loads. With protective coverings, ductile mechanical and electrical components and many more fragile components can be expected to survive all but the most severe impact loads.

6.2.8 Component anchorage. Depending on the specifics of the design condition, ductile design of anchors in concrete or masonry may be intended to satisfy one or all of the following objectives: (1) to ensure adequate load redistribution between anchors in a group, (2) to allow for anchor overload without precipitous failure, and/or (3) to dissipate seismic energy. Unless specific attention is paid to the conditions necessary to ensure the desired hysteretic response (adequate gauge length, anchor spacing, edge distance, steel properties, etc.), it is not recommended that anchors be relied upon for energy dissipation. Inasmuch as the anchor provides the transfer of load from a relatively deformable material (such as steel) to a low deformability material (such as concrete or masonry), achieving deformable, energy-absorbing behavior in the anchor itself is often difficult. On the other hand, the concept of providing a fuse, or deformable link, in the load path to the anchor is encouraged. This approach allows the designer to provide the necessary level of ductility and overstrength in the connection while at the same time protecting the anchor from overload and eliminates the need to balance steel strength and deformability in the anchor with variable edge distances and anchor spacings.

Previous restrictions on the anchor l/d ratio as a means of defining ductile vs. non-ductile anchors have been deleted from the *Provisions* in recognition of the difficulty in defining the conditions necessary for real ductile behavior. For example, a single anchor with the necessary embedment to force ductile failure of the anchor bolt in tension may still experience concrete fracture (a non-ductile failure mode) if the edge distance is small, if the anchor is placed in a group of tension-loaded anchors with reduced spacing, or if the anchor is loaded in shear instead of tension. In fact, many if not most anchor applications, such as building cladding attachments and large equipment anchorages, are subject primarily to shear loading. In these cases, even if the anchor steel is ductile, shear failure of the bolt may be non-ductile, particularly if the deformation of the anchor is constrained by rigid elements on either side of the joint. It is therefore left to the designer to establish the necessary criteria for ductile anchor failure.

Post-installed expansion and undercut anchors may now be qualified as suitable for seismic applications using the testing procedures outlined in ACI 355.2-01, *Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete (355.2-01)* and *Commentary (355.2R-01)*. The design of qualified anchors in concrete is addressed in Sec. 9.6 of the *Provisions*. No such standard exists as yet for chemical anchors, and caution should be exercised in their use in earthquake environments, particularly with respect to the effects of earthquake-induced cracking of the concrete or masonry on anchor capacity. The capacity of anchors in masonry is rarely governed by steel capacity, and as such masonry anchors should in general be considered to be non-ductile. For this reason, the design of anchors in masonry should be carried out with an R_p of 1.5.

For purposes of the *Provisions*, a chemical anchor is a post-installed anchor rod, usually steel, which is inserted into a drilled hole in concrete or masonry together with a polymer or cementitious grout and which derives its tension capacity primarily from bond. On the other hand, reference to adhesives is intended to include steel plates and other structural elements adhered to the surface of another structural component with adhesive. An example of this type of application is the attachment of computer access floors base plates to a floor slab with epoxy. This type of connection is typically non-ductile.

Allowable loads for anchors should not be increased for earthquake loading. Possible reductions in allowable loads for particular anchor types to account for loss of stiffness and strength should be determined by means of appropriate dynamic testing.

Anchors that are used to support towers, masts, and equipment often are provided with double nuts to allow for leveling during installation. Where baseplate grout is provided at such double-nutted anchors, it should not be relied upon to carry loads since it can shrink and crack or is often omitted altogether. In this case, the anchors are loaded in tension, compression, shear, and flexure and should be designed accordingly. Prying forces on anchors, which result from a lack of rotational stiffness in the connected part, can be critical for anchor design and must be considered explicitly.

For anchorages that are not provided with a mechanism to transfer compression loads, the design for overturning must reflect the actual stiffness of the baseplate, equipment, housing, etc., in determining the location of the compression centroid and the distribution of uplift loads to the anchors.

Possible reductions in allowable loads for particular anchor types to account for loss of stiffness and strength should be determined through appropriate dynamic testing.

While the requirements do not prohibit the use of single anchor connections, it is considered good practice to use at least two anchors in any load-carrying connection whose failure might lead to collapse, partial collapse, or disruption of a critical inertial load path.

Tests have shown that there are consistent shear ductility variations between bolts anchored to drilled or punched plates with nuts and connections using welded, shear studs. Recommendations for design are not presently available but this issue should be considered in critical connections subject to dynamic or seismic loading.

It is important to relate the anchorage demands defined by Chapter 6 with the material capacities defined in the other chapters (e.g., Chapters 9 and 11).

6.2.8.5 Power-actuated fasteners. Generally, power-actuated fasteners in concrete tend to exhibit variations in load capacity that are somewhat larger than post-installed drilled anchors. Therefore, the suitability of power-actuated fasteners should be demonstrated by a simulated seismic test program prior to their use. When properly installed in steel, such fasteners are reliable, showing high capacities with very low variability.

6.2.9 Construction documents. The committee believes that each quality assurance activity specified in Chapter 2 should have a clearly defined basis. As a result, construction documents are required for all components for which Chapter 2 requires special inspection or testing.

The committee believes that, in order to provide a reasonable level of assurance that the construction and installation of components is consistent with the basis of the supporting seismic design, appropriate construction documents are needed. Of particular concern are systems involving multiple trades and suppliers. In these cases, it is important that a registered design professional prepare construction documents for use by the various trades and suppliers in the course of construction.

6.3 ARCHITECTURAL COMPONENTS

The requirements of Sec. 6.3 are intended to reduce the threat of life safety hazards posed by components and elements from the standpoint of stability and integrity. There are several circumstances where such components may pose a threat.

1. Where loss of integrity and/or connection failure under seismic motion poses a direct hazard in that the components may fall on building occupants.
2. Where loss of integrity and/or connection failure may result in a hazard for people outside of a building because components such as exterior cladding and glazing may fall on them.
3. Where failure or upset of interior components may impede access to a required exit.

The requirements are intended to apply to all of the circumstances listed above. Although the safety hazard posed by exterior cladding is obvious, judgment may be needed in assessing the extent to which the requirements should be applied to other hazards.

Property loss through damage to architectural components is not specifically addressed in the *Provisions*. Function and operation of a building also may be affected by damage to architectural components if it is necessary to cease operations while repairs are undertaken. In general, requirements to improve life-safety also will reduce property loss and loss of building function.

In general, functional loss is more likely to be affected by loss of mechanical or electrical components. Architectural damage, unless very severe, usually can be accommodated on a temporary basis. Very severe architectural damage results from excessive structural response that often also results in significant structural damage and building evacuation.

6.3.1 Forces and displacements. Components that could be damaged by or could damage other components and are fastened at multiple locations to a structure should be designed to accommodate seismic relative displacements. Such components include glazing, partitions, stairs, and veneers.

Certain types of veneer elements, such as aluminum or vinyl siding and trim, possess high deformability. These systems are generally light and can undergo large deformations without separating from the structure. However, care must be taken when designing these elements to ensure that the low deformability components that may be part of the curtain wall system, such as glazing panels, have been detailed to accommodate the expected deformations without failure.

Specific requirements for cladding are provided. Glazing, both exterior and interior, and partitions must be capable of accommodating story drift without causing a life-safety hazard. Design judgment must be used with respect to the assessment of life-safety hazard and the likelihood of life-threatening damage. Special detailing to accommodate drift for typical replaceable gypsum board or demountable partitions is not likely to be cost-effective, and damage to these components poses a low hazard to life safety. Nonstructural fire-resistant enclosures and fire-rated partitions may require some special detailing to ensure that they retain their integrity. Special detailing should provide isolation from the adjacent or enclosing structure for deformation equivalent to the calculated drift (relative displacement). In-plane differential movement between structure and wall is permitted. Provision also must be made for out-of-plane restraint. These requirements are particularly important in relation to the larger drifts experienced in steel or concrete moment frame structures. The problem is less likely to be encountered in stiff structures, such as those with shear walls.

Differential vertical movement between horizontal cantilevers in adjacent stories (such as cantilevered floor slabs) has occurred in past earthquakes. The possibility of such effects should be considered in the design of exterior walls.

6.3.2 Exterior nonstructural wall elements and connections. The *Provisions* requires that nonbearing wall panels that are attached to or enclose the structure be designed to resist the (inertial) forces and to accommodate movements of the structure resulting from lateral forces or temperature change. The force requirements often overshadow the importance of allowing thermal movement and may therefore require special detailing in order to prevent moisture penetration and allow thermal movements.

Connections should be designed so as to prevent the loss of load-carrying capacity in the event of significant yielding. Between points of connection, panels should be separated from the structure sufficiently to avoid contact due to seismic action.

The *Provisions* requires allowance for story drift. This required allowance can amount to 2 in. (50 mm) or more from one floor to the next and may present a greater challenge to the designer than requirements for the forces. In practice, separations between adjacent panels, intended to limit contact and resulting panel mis-alignment and/or damage under all but extreme building response, are limited to about 3/4 in.

(19 mm) for practical joint detailing with acceptable appearance. The *Provisions* calls for a minimum separation of 1/2 in. (13 mm). The design should be consistent with the manufacturing and construction tolerances of the materials used to achieve this dimension.

If wind loads govern, connectors and panels should allow for not less than two times the story drift caused by wind loads determined using a return period appropriate to the site location.

The *Provisions* requirements are in anticipation of frame yielding to absorb energy. Appropriate isolation can be achieved by means of slots, but the use of long rods that flex is preferable because this approach does not depend on precise installation to achieve the desired action. The rods must be designed to carry tension and compression in addition to induced flexural stresses. Care must be used in allowing inelastic bending in the rods. Threaded rods pushed into the strain-hardening region of the stress-strain curve are subject to brittle low-cycle fatigue failures. Floor-to-floor wall panels are usually rigidly attached to and move with the floor structure nearest the panel bottom. In this condition, isolation connections are used at the upper attachments so that panels translate with the load supporting structure below and are not subjected to large in-plane forces due to movement of the building. Panels also can be supported at the top with isolation connections at the bottom.

When determining the length of slot or displacement demand for the connection, the cumulative effect of tolerances in the supporting frame and cladding panel must be considered.

To minimize the effects of thermal movements and shrinkage on architectural cladding panels, the connection system is generally detailed to be statically determinate. As a result, cladding panel support systems often lack redundancy and failure of a single connection can have catastrophic consequences. In recognition of this, the *Provisions* require that fasteners be designed for approximately 4 times the required panel force and that the connecting member be ductile. This is intended to ensure that the energy absorption takes place in the connecting member and not at the connection itself and that the more brittle fasteners remain essentially elastic under seismic loading. The factor of 4 has been incorporated into the a_p and R_p factors in consideration of installation and material variability and the consequences of a brittle connection failure in a statically determinate system.

6.3.3 Out-of-plane bending. Most walls are subject to out-of-plane forces when a building is shaken by an earthquake. These forces and the bending they induce must be considered in the design of wall panels, nonstructural walls, and partitions. This is particularly important for systems composed of brittle materials or materials with low flexural strength. The conventional limits based upon deflections as a proportion of the span may be used with the applied force as derived in Sec. 6.2.6.

Judgment must be used in assessing the deflection capability of the component. The intent is that a heavy material (such as concrete block) or an applied finish (such as brittle heavy stone or tile) should not fail in a hazardous manner as a result of out-of-plane forces. Deflection in itself is not a hazard. A steel-stud partition might undergo considerable deflection without creating a hazard; but if the same partition supports a marble facing, a hazard might exist and special detailing may be necessary.

6.3.4 Suspended ceilings. Suspended ceiling systems usually are fabricated using a wide range of building materials with individual components having different material characteristics. Some systems are homogeneous whereas others incorporate suspension systems with acoustic tile or lay-in panels. Seismic performance during recent, large earthquakes in California has raised two concerns:

1. Support of the individual panels at walls and expansion joints, and
2. Interaction with fire sprinkler systems.

In an attempt to address these concerns, alternate methods were developed in a cooperative effort by representatives of the ceiling and fire sprinkler industries and registered design professionals. It is hoped that future research and investigation will result in further improvements in the *Provisions*.

Consideration must be given to the placement of seismic bracing, the relation of light fixtures and other loads placed into the ceiling diaphragm, and the independent bracing of partitions in order to effectively maintain the performance characteristics of the ceiling system. The ceiling system may require bracing and allowance for the interaction of components.

Dynamic testing of suspended ceiling systems constructed according to the requirements of current industry seismic standards (*UBC Standard 25-2*) performed by ANCO Engineers, Inc. (1983) has demonstrated that the splayed wires, even with the vertical compression struts, may not adequately limit lateral motion of the ceiling system due to the flexibility introduced by the straightening of the wire end loops. In addition, splay wires usually are installed slack to prevent unleveling of the ceiling grid and to avoid above-ceiling utilities. Not infrequently, bracing wires are omitted because of obstructions. Testing also has shown that system performance without splayed wires or struts was good if sufficient clearance is provided at penetrations and closure angles are wide enough.

The lateral seismic restraint for a non-rigidly braced suspended ceiling is primarily provided by the ceiling coming into contact with the perimeter wall. The wall provides a large contact surface to restrain the ceiling. The key to good seismic performance is that the width of the closure angle around the perimeter is adequate to accommodate ceiling motion and that penetrations, such as columns and piping, have adequate clearance to avoid concentrating restraining loads on the ceiling system. The behavior of an unbraced ceiling system is similar to that of a pendulum; therefore, the lateral displacement is approximately proportional to the level of velocity-controlled ground motion and the square root of the suspension length. Therefore, a new section has been added that permits exemption from force calculations if certain displacement criteria are met. The default displacement limit has been determined based on anticipated damping and energy absorption of the suspended ceiling system assuming minimal significant impact with the perimeter wall.

6.3.5 Access floors. Performance of computer access floors during past earthquakes and during cyclic load tests indicate that typical raised access floor systems may behave in a brittle manner and may exhibit little reserve capacity beyond initial yielding or failure of critical connections. Recent testing indicates that individual panels may “pop out” of the supporting grid during seismic motions. Consideration should be given to mechanically fastening the individual panels to the supporting pedestals or stringers in egress pathways.

For systems with floor stringers, it is acceptable practice to calculate the seismic force, F_p , for the entire access floor system within a partitioned space and then distribute the total force to the individual braces or pedestals. Stringerless systems need to be evaluated very carefully to ensure a viable seismic load path.

Overtuning effects for the design of individual pedestals is a concern. Each pedestal usually is specified to carry an ultimate design vertical load greatly in excess of the W_p used in determining the seismic force F_p . It is non-conservative to use the design vertical load simultaneously with the design seismic force when considering anchor bolts, pedestal bending, and pedestal welds to base plates. The maximum concurrent vertical load when considering overturning effects is therefore limited to the value of W_p used in determining F_p . “Slip on” heads are not mechanically fastened to the pedestal shaft and provide doubtful capacity to transfer overturning moments from the floor panels or stringers to the pedestal.

To preclude brittle failure, each element in the seismic load path must demonstrate the capacity for elastic or inelastic energy absorption. Buckling failure modes also must be prevented. Lesser seismic force requirements are deemed appropriate for access floors designed to preclude brittle and buckling failure modes.

6.3.6 Partitions. Partitions are sometimes designed to run only from floor to a suspended ceiling which provides doubtful lateral support. Partitions subject to these requirements must have independent

lateral support bracing from the top of the partition to the building structure or to a substructure attached to the building structure.

6.3.7 Glass in glazed curtain walls, glazed storefronts, and glazed partitions. Glass performance in earthquakes can fall into one of four categories:

1. The glass remains unbroken in its frame or anchorage.
2. The glass cracks but remains in its frame or anchorage while continuing to provide a weather barrier, and to be otherwise serviceable.
3. The glass shatters but remains in its frame or anchorage in a precarious condition, likely to fall out at any time.
4. The glass falls out of its frame or anchorage, either in fragments, shards, or whole panels.

Categories 1. and 2. provide both life safety and immediate occupancy levels of performance. In the case of category 2., even though the glass is cracked, it continues to provide a weather enclosure and barrier, and its replacement can be planned over a period of time. (Such glass replacement need not be performed in the immediate aftermath of the earthquake.) Categories 3. and 4. cannot provide for immediate occupancy, and their provision of a life safety level of performance depends on the post-breakage characteristics of the glass and the height from which it can fall. Tempered glass shatters into multiple, pebble-size fragments that fall from the frame or anchorage in clusters. These broken glass clusters are relatively harmless to humans when they fall from limited heights, but when they fall from greater heights they could be harmful.

The requirement that $\Delta_{fallout}$ not be less than $1.25ID_p$ is derived from *Earthquake Safety Design of Windows*, published in November 1982 by the Sheet Glass Association of Japan. Eq. 6.3-1 is based on a similar equation in Bouwkamp and Meehan (1960) that permits calculation of the story drift required to cause glass-to-frame contact in a given rectangular window frame. Both calculations are based on the principle that a rectangular window frame (specifically, one that is anchored mechanically to adjacent stories of the primary structural system of the building) becomes a parallelogram as a result of story drift, and that glass-to-frame contact occurs when the length of the shorter diagonal of the parallelogram is equal to the diagonal of the glass panel itself.

The 1.25 factor in the requirements described above reflect uncertainties associated with calculated inelastic seismic displacements in building structures. Wright (1989) stated that “post-elastic deformations, calculated using the structural analysis process, may well underestimate the actual building deformation by up to 30 percent. It would therefore be reasonable to require the curtain wall glazing system to withstand 1.25 times the computed maximum interstory displacement to verify adequate performance.” Therefore, Wright’s comments form the basis for employing the 1.25 factor in these requirements.

Introduction. Seismic design requirements for glass in building codes have traditionally been non-existent or have been limited to the general statement that “drift be accommodated.” No distinction has been made regarding the seismic performance of different types of glass, different frames, and different glazing systems. Yet, significant differences exist in the performance of various glass types subjected to simulated earthquake conditions. Controlled laboratory studies were conducted to investigate the cracking resistance and fallout resistance of different types of glass installed in the same storefront and mid-rise wall systems. Effects of glass surface prestress, lamination, wall system type, and dry versus structural silicone glazing were considered. Laboratory results revealed that distinct magnitudes of story drift cause glass cracking and glass fallout in each glass type tested. Notable differences in seismic resistance exist between glass types commonly used in contemporary building design.

Test rig and experimental plan. In-plane dynamic racking tests were performed using the rig shown in Figure C6.3-1. Rectangular steel tubes at the top and bottom of the facility are supported on roller assemblies, which permit only horizontal motion of the tubes. The bottom steel tube is driven by a

computer-controlled hydraulic ram, while the top tube is attached to the bottom tube by means of a fulcrum and pivot arm assembly. This mechanism causes the upper steel tube to displace the same amount as the lower steel tube, but in the opposite direction, which doubles the amount of story drift that can be imposed on a test specimen from ± 76 mm (± 3 in.) to ± 152 mm (± 6 in.). The test facility accommodated up to three glass test panels, each 1.5 m (5 ft) wide by 1.8 m (6 ft) high. A more detailed description of the dynamic racking test rig is included in Behr and Belarbi (1996).

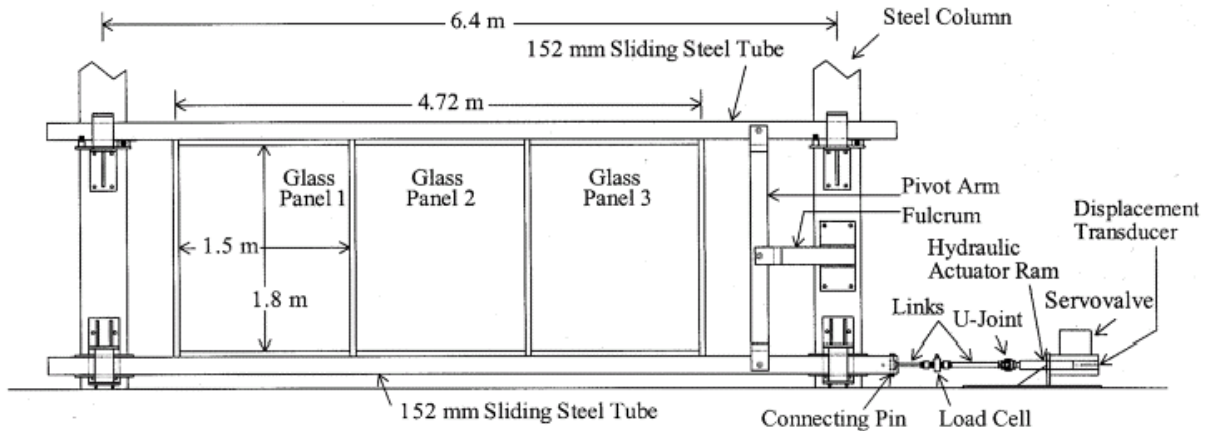


Figure C6.3-1 Dynamic racking test rig.

Several types of glass, shown in Table C6.3-1, were tested under simulated seismic conditions in the storefront and mid-rise dynamic racking tests. These glass types, along with the wall systems employed in the tests, were selected after polling industry practitioners and wall system designers for their opinions regarding common glass types and common wall system types employed in contemporary storefront and mid-rise wall constructions.

Storefront wall system tests. Tests were conducted on various glass types that were dry-glazed within a wall system, as commonly used in storefront applications. Loading histories for the storefront wall system tests were based on dynamic analyses performed on a “typical” storefront building that was not designed specifically for seismic resistance (Pantelides et al., 1996). Two types of tests were conducted on the storefront wall systems: (1) serviceability tests, wherein the drift loading history of the glass simulated the response of a storefront building structure to a “maximum probable” earthquake event; and (2) ultimate tests, wherein drift amplitudes were twice those of the serviceability tests, which was a simplified means of approximating the loading history of a “maximum credible” earthquake event. As indicated in Table C6.3-1, five glass types were tested, all dry-glazed in a storefront wall system. Three glass panels were mounted side by side in the test facility, after which horizontal (in-plane) racking motions were applied.

Table C6.3-1 Glass Types Included in Storefront and Mid-Rise Dynamic Racking Tests

Glass Type	Storefront Tests	Mid-Rise Tests
6 mm (1/4 in.) annealed monolithic	✓	✓
6 mm (1/4 in.) heat-strengthened monolithic		✓
6 mm (1/4 in.) fully tempered monolithic	✓	✓

6 mm (1/4 in.) annealed monolithic with 0.1 mm pet film (film not anchored to wall system frame)		✓
6 mm (1/4 in.) annealed laminated	✓	✓
6 mm (1/4 in.) heat-strengthened laminated		✓
6 mm (1/4 in.) heat-strengthened monolithic spandrel		✓
25 mm (1 in.) annealed insulating glass units	✓	✓
25 mm (1 in.) heat-strengthened insulating glass units		✓

The serviceability test lasted approximately 55 seconds and incorporated drift amplitudes ranging from ± 6 to ± 44 mm (± 0.25 to ± 1.75 in.). The drift pattern in the ultimate test was formed by doubling each drift amplitude in the serviceability test. Both tests were performed at a nominal frequency of 0.8 Hz.

Experimental results indicated that for all glass types tested, serviceability limit states associated with glass edge damage and gasket seal degradation in the storefront wall system were exceeded during the moderate earthquake simulation (that is, the serviceability test). Ultimate limit states associated with major cracking and glass fallout were reached for the most common storefront glass type, 6 mm (1/4 in.) annealed monolithic glass, during the severe earthquake simulation (that is, the ultimate test). This observation is consistent with a reconnaissance report of damage resulting from the Northridge Earthquake (EERI, 1994). More information regarding the storefront wall system tests is included in Behr, Belarbi, and Brown (1995). In addition to the serviceability and ultimate tests, increasing-amplitude “crescendo tests,” similar to those described below for the mid-rise tests, were performed at a frequency of 0.8 Hz on selected storefront glass types. Results of these crescendo tests are reported in Behr, Belarbi, and Brown (1995) and are included in some of the comparisons made below.

Mid-rise curtain wall system tests. Another series of tests focused on the behavior of glass panels in a popular curtain wall system for mid-rise buildings. All mid-rise glass types in Table C6.3-1 were tested with a dry-glazed wall system that uses polymeric (rubber) gaskets wedged between the glass edges and the curtain wall frame to secure each glass panel perimeter. In addition, three glass types were tested with a bead of structural silicone sealant on the vertical glass edges and dry glazing gaskets on the horizontal edges (that is, a “two-side structural silicone glazing system”). Six specimens of each glass type were tested.

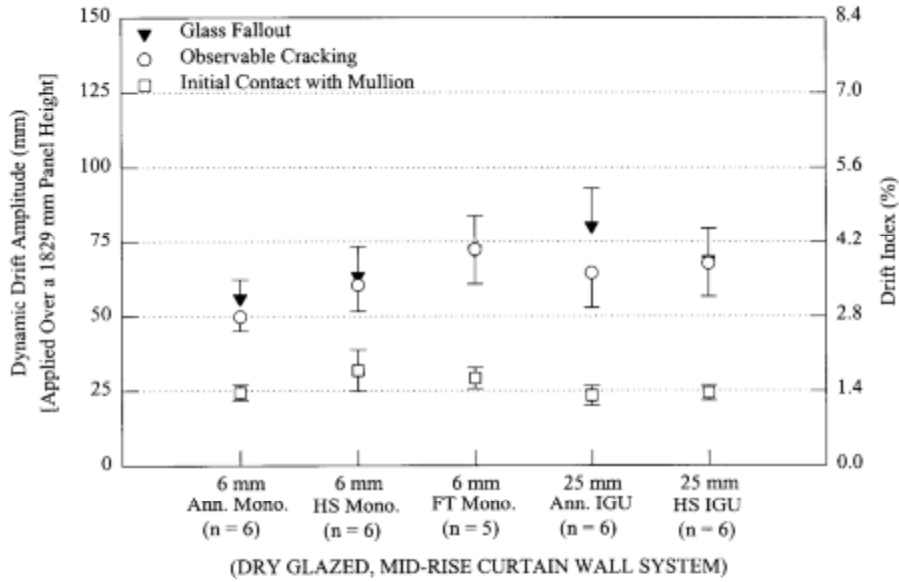
Crescendo tests were performed on all mid-rise test specimens. As described by Behr and Belarbi (1996), the crescendo test consisted of a series of alternating “ramp-up” and “constant amplitude” intervals, each containing four sinusoidal-shaped drift cycles. Each drift amplitude “step” (that is, the increase in amplitude between adjacent constant amplitude intervals, which was achieved by completing the four cycles in the intermediary ramp-up interval) was ± 6 mm (± 0.25 in.). The entire crescendo test sequence lasted approximately 230 seconds. Crescendo tests on mid-rise glass specimens were conducted at 1.0 Hz for dynamic racking amplitudes from 0 to 114 mm (0 to 4.5 in.), 0.8 Hz for amplitudes from 114 to 140 mm (4.5 to 5.5 in.), and 0.5 Hz for amplitudes from 140 to 152 mm (5.5 to 6 in.). These frequency reductions at higher racking amplitudes were necessary to avoid exceeding the capacity of the hydraulic actuator ram in the dynamic racking test rig.

The drift magnitude at which glass cracking was first observed was called the “serviceability drift limit,” which corresponds to the drift magnitude at which glass damage would necessitate glass replacement. The drift magnitude at which glass fallout occurred was called the “ultimate drift limit,” which corresponds to the drift magnitude at which glass damage would become a life safety hazard. This ultimate drift limit for architectural glass is related to “ $A_{fallout}$ ” in Sec. 6.3.7 of the *Provisions*, noting that horizontal racking displacements (drifts) in the crescendo tests were typically applied to test specimens having panel heights of only 1.8 m (6 ft).

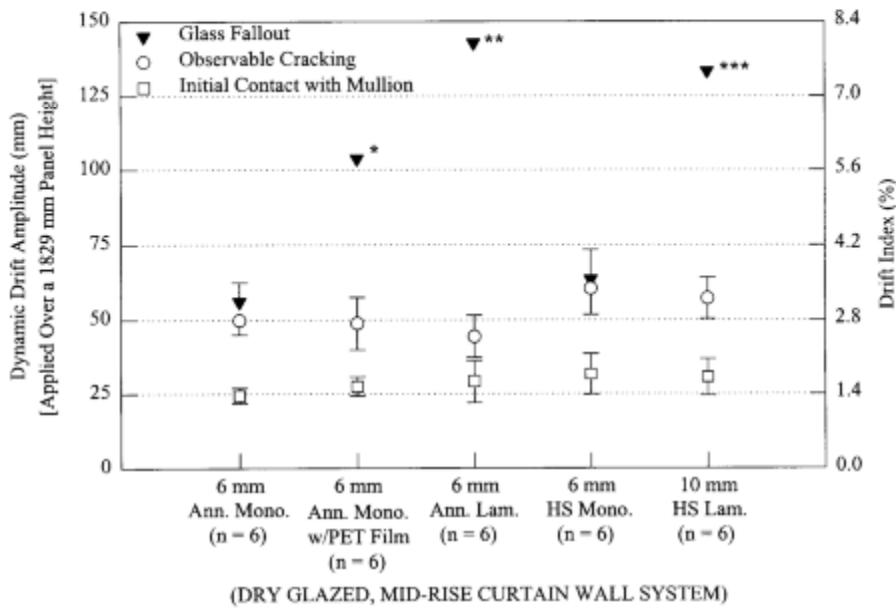
In addition to recording the serviceability drift limit and ultimate drift limit for each glass test specimen, the drift magnitude causing first contact between the glass panel and the aluminum frame was also recorded. To establish when this contact occurred, thin copper wires were attached to each corner of the glass panel and were connected to an electronics box. If the copper wire came into contact with the aluminum frame, an indicator light on an electronics box was actuated. Measured drifts causing glass-to-aluminum contact correlated well with those predicted by Eq. 6.3-1.

Glass failure patterns from crescendo tests. Glass failure patterns were recorded during each storefront test and mid-rise test. Annealed monolithic glass tended to fracture into sizable shards, which then fell from the curtain wall frame. Heat-strengthened monolithic glass generally broke into smaller shards than annealed monolithic glass, with the average shard size being inversely proportional to the magnitude of surface compressive prestress in the glass. Fully tempered monolithic glass shattered into much smaller, cube-shaped fragments. Annealed monolithic glass with unanchored 0.1 mm (4 mil) PET film also fractured into large shards, much like un-filmed annealed monolithic glass, but the shards adhered to the film. However, when the weight of the glass shards became excessive, the entire shard/film conglomeration sometimes fell from the glazing pocket as a unit. Thus, unanchored 0.1 mm PET film was not observed to be totally effective in terms of preventing glass fallout under simulated seismic loadings, which agrees with field observations made in the aftermath of the 1994 Northridge Earthquake (Gates and McGavin, 1998). Annealed and heat-strengthened laminated glass units experienced fracture on each glass ply separately, which permitted these laminated glass units to retain sufficient rigidity to remain in the glazing pocket after one (or even both), glass plies had fractured due to glass-to-aluminum contacts. Annealed and heat-strengthened laminated glass units exhibited the highest resistance to glass fallout during the dynamic racking tests.

Quantitative drift limit data from crescendo tests. Serviceability and ultimate drift limit data obtained during the crescendo tests are presented in four panels in Figure C6.3-2. Figure C6.3-2a shows the effects of glass surface prestress (that is, annealed, heat-strengthened and fully tempered glass) on seismic drift limits; Figure C6.3-2b shows the effects of lamination (that is, monolithic glass, monolithic glass with unanchored 0.1 mm PET film, and laminated glass); Figure C6.3-2c shows the effects of wall system type (that is, lighter, more flexible, storefront wall system versus the same glass types tested in a heavier, stiffer, mid-rise wall system); and Figure C6.3-2d shows the effects of structural silicone glazing (that is, dry glazing versus two-side structural silicone glazing). Each symbol plotted in Figure C6.3-2 is the mean value for specimens of a given glass type, along with \pm one standard deviation error bars. In those cases where error bars for a particular glass type overlap, only one side of the error bar is plotted. In cases where the glass panel did not experience fallout by the end of the crescendo test, a conservative ultimate drift limit magnitude of 152 mm (6 in.) (the racking limit of the test facility) is assigned for plotting purposes in Figure C6.3-2. (This ultimate drift limit, shown with a “▼” symbol in Figure C6.3-2, is related to the term “ $A_{fallout}$ ” in Sec. 6.3.7 of the *Provisions*.) No error bars are plotted for these “pseudo data points,” since the drift magnitude at which the glass panel would actually have experienced fallout could not be observed; certainly, the actual ultimate drift limits for these specimens are greater than ± 152 mm (± 6 in.).



(a) Effects of Glass Surface Prestress



*1 of 6 specimens did not fall out. **5 of 6 specimens did not fall out. ***2 of 6 specimens did not fall out.

(b) Effects of Lamination

Figure C6.3-2 Seismic drift limits from crescendo tests on architectural glass

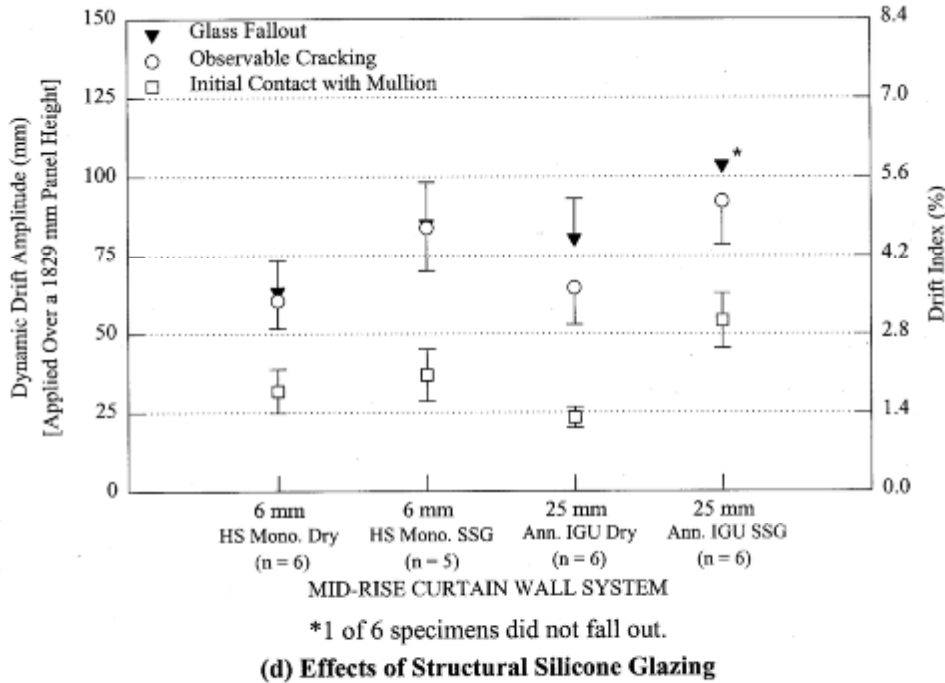
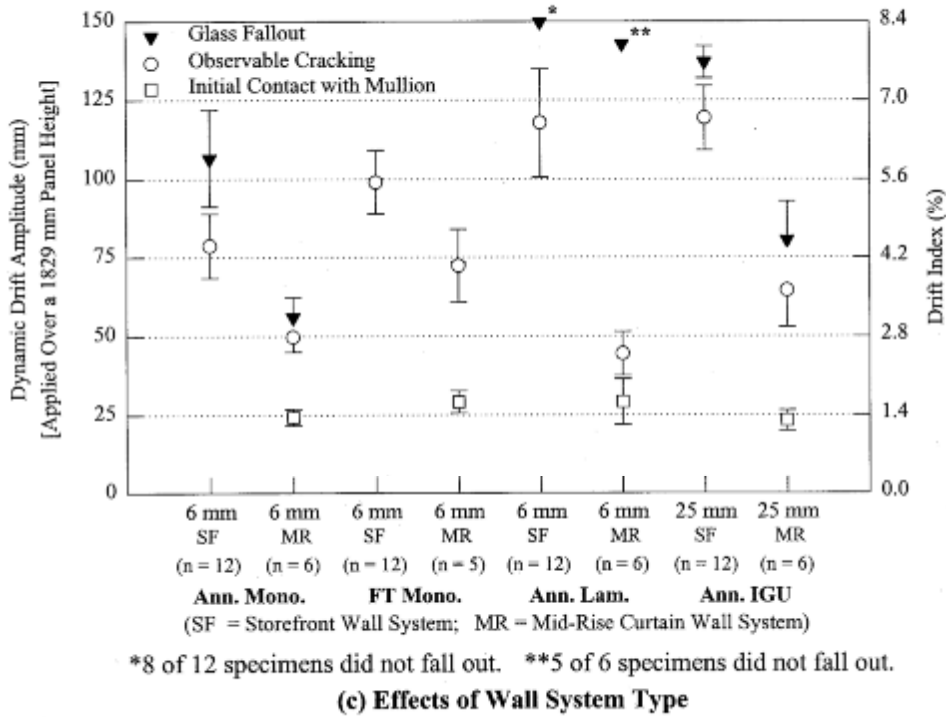


Figure C6.3-2 (continued) Seismic drift limits from crescendo tests on architectural glass

The ± 152 mm (± 6 in.) racking limit of the test rig, when applied over the 1829 mm (72 in.) height of glazing panel specimens represents a severe story drift index of more than 8 percent. This 8 percent

drift index exceeds, by a significant margin, provisions in Sec. 4.5.1 (Table 4.5-1) that set allowable drift limits between 0.7 percent and 2.5 percent, depending on structure type and Seismic Use Group. Thus, the drift limits, Δ_a , in Table 4.5-1 are considerably lower than the racking limits of the laboratory facility used for the crescendo tests. In building design, however, values of $\Delta_{fallout}$ would need to be significantly higher than the story drifts exhibited by the primary building structure in order to provide an acceptable safety margin against glass fallout.

Summary observations from Figure C6.3-2. Effects of glass surface prestress - Figure C6.3-2a illustrates the effects of glass surface prestress on observed seismic drift limits. To eliminate all variables except for glass surface prestress, data from only the mid-rise curtain wall tests are plotted. Slight increases in cracking and fallout drift limits can be seen for 6 mm (0.25 in.) monolithic glass as the level of glass surface prestress increases from annealed to heat-strengthened to fully tempered glass. However, effects of glass surface prestress on observed seismic drift limits were statistically significant only when comparing 6 mm fully tempered monolithic glass to 6 mm annealed monolithic glass. All six of the 6 mm fully tempered monolithic glass specimens shattered when initial cracking occurred, causing the entire glass panels to fall out. Similar behavior was observed in four of the six 6 mm heat-strengthened monolithic glass specimens. No appreciable differences in seismic drift limits existed between annealed and heat-strengthened 25 mm (1 in.) insulating glass units.

Effects of lamination -- Figure C6.3-2b shows the effects of lamination configuration on seismic drift limits. Lamination had no appreciable effect on the drift magnitudes associated with first observable glass cracking. In a dry-glazed system, the base glass type (and not the lamination configuration) appeared to control the drift magnitude associated with glass cracking. However, lamination configuration had a pronounced effect on glass fallout resistance (that is, $\Delta_{fallout}$). Specifically, monolithic glass types were more prone to glass fallout than were either annealed monolithic glass with unanchored 0.1 mm PET film or annealed laminated glass. All six annealed monolithic glass panels experienced glass fallout during the tests; five of six annealed monolithic glass specimens with unanchored 0.1 mm PET film experienced fallout; only one of six annealed laminated glass panels experienced fallout.

Laboratory tests also showed that heat-strengthened laminated glass had higher fallout resistance than did heat-strengthened monolithic glass. Heat-strengthened monolithic glass panels fell out at significantly lower drift magnitudes than did heat-strengthened laminated glass units. Heat-strengthened laminated glass units tended to fall out in one large piece, instead of exhibiting the smaller shard fallout behavior of heat-strengthened monolithic glass.

Effects of wall system type -- Figure C6.3-2c illustrates the effects of wall system type on observed seismic drift limits. For all four glass types tested in both the storefront and mid-rise wall systems, the lighter, more flexible storefront frames allowed larger drift magnitudes before glass cracking or glass fallout than did the heavier, stiffer mid-rise curtain wall frames. This observation held true for all glass types tested in both wall system types.

Effects of structural silicone glazing -- As shown in Figure C6.3-2d, use of a two-side structural silicone glazing system increased the dynamic drift magnitudes associated with first observable glass cracking in both heat-strengthened monolithic glass and annealed insulating glass units. During the crescendo tests, glass panels were observed to “walk” horizontally across the frame after the beads of structural silicone sealant had sheared. Because the mid-rise curtain wall crescendo tests were performed on single glass panels, the glass specimen was unobstructed as it walked horizontally across the frame. In a multi-panel curtain wall assembly on an actual building, adjacent glass panels could collide, which could induce glass cracking at lower drift magnitudes than those observed in the single-panel tests performed in this study. It is also clear from Figure C6.3-2d that glass specimens with two-side structural silicone glazing exhibited higher resistance to glass fallout than did comparable dry-glazed glass specimens.

Conclusion. Dynamic racking tests showed that distinct and repeatable dynamic drift magnitudes were associated with glass cracking and glass fallout in various types of glass tested in storefront and mid-rise

wall systems. Seismic resistance varied widely between glass types commonly employed in contemporary building design. Annealed and heat-strengthened laminated glass types exhibited higher resistance to glass fallout than did monolithic glass types. Annealed monolithic glass with unanchored 0.1 mm PET film exhibited total fallout of the glass shard/adhesive film conglomeration in five out of six of the crescendo tests performed.

Glass panels glazed within stiffer aluminum frames were less tolerant of glass-to-aluminum collisions and were associated with glass fallout events at lower drift magnitudes than were the same glass types tested in a more flexible aluminum frame. Glazing details were also found to have significant effects on the seismic performance of architectural glass. Specifically, architectural glass within a wall system using a structural silicone glaze on two sides exhibited higher seismic resistance than did identical glass specimens dry-glazed on all four sides within a comparable wall framing system.

6.4 MECHANICAL AND ELECTRICAL COMPONENTS

The primary focus of these requirements is on the design of attachments and equipment supports for mechanical and electrical components.

The requirements are intended to reduce the hazard to life posed by the loss of component structural stability or integrity. The requirements should increase the reliability of component operation but do not directly address the assurance of functionality. For critical components where operability is vital, the requirements of Sec. 2.4.5 provide methods for seismically qualifying the component.

The design of mechanical and electrical components must consider two levels of earthquake safety. For the first safety level, failure of the mechanical or electrical component itself poses no significant hazard. In this case, the only hazard posed by the component is if the support and the means by which the component and its supports are attached to the building or the ground fails and the component could slide, topple, fall, or otherwise move in a manner that creates a hazard for persons nearby. In the first category, the intent of these requirements is only to design the support and the means by which the component is attached to the structure, defined in the Definitions as “supports” and “attachments.” For the second safety level, failure of the mechanical or electrical equipment itself poses a significant hazard. This could be a case either of failure of a containment having hazardous contents or contents required after the earthquake or of functional failure of a component required to remain operable after an earthquake. In this second category, the intent of these requirements is to provide guidance for the design of the component as well as the means by which the component is supported and attached to the structure. The requirements should increase the survivability of this second category of component but the assurance of functionality may require additional considerations.

Examples of this second category include fire protection piping or an uninterruptible power supply in a hospital. Another example involves the rupture of a vessel or piping that contains sufficient quantities of highly toxic or explosive substances such that a release would be hazardous to the safety of building occupants or the general public. In assessing whether failure of the mechanical or electrical equipment itself poses a hazard, certain judgments may be necessary. For example, small, flat-bottom tanks themselves may not need to be designed for earthquake loads, but the hazard of a large fluid spill associated with seismic failure of large, flat-bottom tanks suggests that the design of many, if not most, of such tanks should consider earthquake loads. Distinguishing between large and small, in this case, may require an assessment of potential damage caused by a spill of the fluid contents as outlined in Sec. 6.2.3 and Chapter 1 of ASCE-7

It is intended that the requirements provide guidance for the design of components for both conditions in the second category. This is primarily accomplished by increasing the design forces with an importance factor, I_p . However, this directly affects only structural integrity and stability. Function and operability of mechanical and electrical components may be affected only indirectly by increasing design forces. For complex components, testing or experience may be the only reasonable way to improve the assurance of function and operability. On the basis of past earthquake experience, it may

be reasonable to conclude that if structural integrity and stability are maintained, function and operability after an earthquake will be provided for many types of equipment components. On the other hand, mechanical joints in containment components (tanks, vessels, piping, etc.) may not remain leaktight in an earthquake even if leaktightness is re-established after the earthquake. Judgment may suggest a more conservative design related in some manner to the perceived hazard than would otherwise be provided by these requirements.

It is not intended that all equipment or parts of equipment be designed for seismic forces. Determination of whether these requirements need to be applied to the design of a specific piece of equipment or a part of that equipment will sometimes be a difficult task. When I_p is specified to be 1.0, damage to, or even failure of, a piece or part of a component is not a concern of these requirements so long as a hazard to life does not exist. Therefore, the restraint or containment of a falling, breaking, or toppling component (or its parts) by the use of bumpers, braces, guys, wedges, shims, tethers, or gapped restraints often may be an acceptable approach to satisfying these requirements even though the component itself may suffer damage. Judgment will be required if the intent of these requirements is to be fulfilled. The following example may be helpful: Since the threat to life is a key consideration, it should be clear that a nonessential air handler package unit that is less than 4 ft (1.2 m) tall bolted to a mechanical room floor is not a threat to life as long as it is prevented from significant motions by having adequate anchorage. Therefore, earthquake design of the air handler itself need not be performed. However, most engineers would agree that a 10-ft (3.0 m) tall tank on 6-ft (1.8 m) angles used as legs mounted on the roof near a building exit does pose a hazard. It is the intent of these requirements that the tank legs, the connections between the roof and the legs, the connections between the legs and the tank, and possibly even the tank itself be designed to resist earthquake forces. Alternatively, restraint of the tank by guys or bracing could be acceptable. Certain suspended components are exempt from lateral bracing requirements, provided they meet prescriptive force and interaction requirements.

It is not the intent of the *Provisions* to require the seismic design of shafts, buckets, cranks, pistons, plungers, impellers, rotors, stators, bearings, switches, gears, nonpressure retaining casings and castings, or similar items. Where the potential for a hazard to life exists, it is expected that design efforts will focus on equipment supports including base plates, anchorages, support lugs, legs, feet, saddles, skirts, hangers, braces, or ties.

Many mechanical and electrical components consist of complex assemblies of mechanical and/or electrical parts that typically are manufactured in an industrial process that produces similar or identical items. Such equipment may include manufacturer's catalog items and often are designed by empirical (trial-and-error) means for functional and transportation loadings. A characteristic of such equipment is that it may be inherently rugged. Rugged, as used herein, refers to an amplexness of construction that provides such equipment with the ability to survive strong motions without significant loss of function. By examining such equipment, an experienced design professional usually should be able to confirm such ruggedness. The results of an assessment of equipment ruggedness will be used in determining an appropriate method and extent of seismic design or qualification efforts.

It also is recognized that a number of professional and industrial organizations have developed nationally recognized codes and standards for the design and construction of specific mechanical and electrical components. In addition to providing design guidance for normal and upset operating conditions and various environmental conditions, some have developed earthquake design guidance in the context of the overall mechanical or electrical design. It is the intent of these requirements that such codes and standards having earthquake design guidance be used; normally the developers of such codes and standards are more familiar with the expected failure modes of the components for which they have developed design and construction rules. In particular, such codes and standards may be based on considerations that are not immediately obvious to the structural design professional. For example, in the design of industrial piping, seismic loads are not typically additive to thermal expansion loads. Given the potential for misunderstanding and mis-application of codes and standards specific to the design of mechanical and electrical systems, it is recommended that a registered design professional

familiar with these *Provisions*, as well as the those of the referenced code or standard used to evaluate the capacity of the mechanical or electrical components, should be involved in the review and acceptance of the seismic design process for such components. In addition, even if such codes and standards do not have earthquake design guidance, it is generally regarded that construction of mechanical and electrical equipment to nationally recognized codes and standards such as those approved by the American National Standards Institute provide adequate strength (with a safety margin often greater than that provided by structural codes) to accommodate all normal and upset operating loads. In this case, it could also be assumed that the component (especially if constructed of ductile materials) will not break up or break away from its supports in such a way as to pose a life-safety hazard. Earthquake damage surveys confirm this.

Specific guidance for selected components or conditions is provided in Sec. 6.4.5 through 6.4.9.

Testing is a well established alternative method of seismic qualification for small to medium size equipment. Several national standards have testing requirements adaptable for seismic qualification. AC 156, *Acceptance Criteria for Seismic Qualification Testing of Nonstructural Components*, is an acceptable shake table testing protocol, which meets the force requirements of the *Provisions* as well as ASCE 7-02.

6.4.1 Component period. Determination of the fundamental period of an item of mechanical or electrical equipment using analytical or in-situ testing methods can become very involved and can produce nonconservative results (that is, underestimated fundamental periods) if not properly performed.

When using analytical methods, it is absolutely essential to define in detail the flexibility of the elements of the equipment base, load path, and attachment to determine K_p . This base flexibility typically dominates equipment component flexibility and thus fundamental period.

When using test methods, it is necessary to ensure that the dominant mode of vibration of concern for seismic evaluation is excited and captured by the testing. This dominant mode of vibration typically cannot be discovered in equipment in-situ tests that measure only ambient vibrations. In order to excite the mode of vibration with the highest fundamental period by in-situ tests, relatively significant input levels of motion are required (that is, the flexibility of the base and attachment needs to be exercised). A procedure such as the resonant frequency search in AC 156 may be used to identify the dominant modes of vibration of the component.

Many types of mechanical components have fundamental periods below 0.06 sec and may be considered to be rigid. Examples include horizontal pumps, engine generators, motor generators, air compressors, and motor driven centrifugal blowers. Other types of mechanical equipment also are very stiff but may have fundamental periods up to approximately 0.125 sec. Examples of these mechanical equipment items include vertical immersion and deep well pumps, belt driven and vane axial fans, heaters, air handlers, chillers, boilers, heat exchangers, filters, and evaporators. These fundamental period estimates do not apply when the equipment is on vibration-isolator supports.

Electrical equipment cabinets can have fundamental periods of approximately 0.06 to 0.3 sec depending upon weight, stiffness of the enclosure assembly, flexibility of the enclosure base, and load path through to the attachment points. Tall, narrow motor control centers and switchboards lie in the upper end of this period range. Low- and medium-voltage switchgear, transformers, battery chargers, inverters, instrumentation cabinets, and instrumentation racks usually have fundamental periods ranging from 0.1 to 0.2 sec. Braced battery racks, stiffened vertical control panels, benchboards, electrical cabinets with top bracing, and wall-mounted panelboards have fundamental periods ranging from 0.06 to 0.1 sec.

6.4.2 and 6.4.3 Mechanical and electrical components. Past earthquakes have demonstrated that most mechanical and electrical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the operational and transportation loads for which the equipment is designed are typically larger than those due to earthquakes. For this reason, the

requirements focus primarily on equipment anchorage and attachments. However, it was felt that mechanical components required to maintain containment of flammable or hazardous materials should themselves be designed for seismic forces.

In addition, the reliability of equipment operability after an earthquake can be increased if the following items are also considered in design:

1. Internal assemblies, subassemblies, and electrical contacts are attached sufficiently to prevent their being subjected to differential movement or impact with other internal assemblies or the equipment enclosure.
2. Operators, motors, generators, and other such components that are functionally attached to mechanical equipment by means of an operating shaft or mechanism are structurally connected or commonly supported with sufficient rigidity such that binding of the operating shaft will be avoided.
3. Any ceramic or other nonductile components in the seismic load path are specifically evaluated.
4. Adjacent electrical cabinets are bolted together and cabinet lineups are prevented from banging into adjacent structural members.

Components that could be damaged or could damage other components and are fastened to multiple locations of a structure should be designed to accommodate seismic relative displacements. Examples of components that should be designed to accommodate seismic relative displacements include bus ducts, cable trays, conduit, elevator guide rails, and piping systems.

6.4.4 Supports and attachments. For some items such as piping, relative seismic displacements between support points generally are of more significance than inertial forces. Components made of ductile materials such as steel or copper can accommodate relative displacement effects by inelastically conforming to the support conditions. However, components made of less ductile materials can only accommodate relative displacement effects if appropriate flexibility or flexible connections are provided.

Of most concern are distribution systems that are a significant life-safety hazard and are routed between two separate building structures. Ductile components with bends and elbows at the building separation point or components that will be subject to bending stresses rather than direct tensile loads due to differential support motion are less prone to damage and are less likely to fracture and fall provided the supports can accommodate the imposed loads.

It is the intent of these requirements to ensure that all mechanical and electrical component supports be designed to accommodate the force and displacement effects prescribed. Component supports are differentiated here from component attachments to emphasize that the supports themselves, the structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers, even if fabricated with and/or by the mechanical or electrical component manufacturer, should be designed for seismic forces. This is regardless of whether the mechanical or electrical component itself is designed for seismic loads. The intention is to prevent a component from sliding, falling, toppling, or otherwise moving such that the component would imperil life.

6.4.5 Utility and service lines. For essential facilities, auxiliary on-site mechanical and electrical utility sources are recommended. It is recommended that an appropriate clause be included if existing codes for the jurisdiction do not presently provide for it.

Sec. 6.4.5 requires that adequate flexibility be provided for utilities at the interface of adjacent and independent structures to accommodate anticipated differential displacement. It affects architectural and mechanical/electrical fittings only where water and energy lines pass through the interface. The displacements considered must include the C_d factor of Sec. 4.3.1 and should be in accordance with *Provisions* Sec. 6.2.7.

Consideration may be necessary for nonessential piping that carries quantities of materials that could damage essential utilities in the event of pipe rupture.

Following a review of information from the Northridge and Loma Prieta earthquakes and discussions with gas company personnel, automatic earthquake shutoff of gas lines at structure entry points is no longer required. The primary justification for this is the consensus opinion that shutoff devices tend to cause more problems than they solve. Commercially available shutoff devices are often susceptible to inadvertent shutoff caused by passing vehicles and other non-seismic vibrations. This leads to disruption of service and often requires that local gas companies reset such devices and relight any pilot lights. In an earthquake, the majority of shutoff devices which actuate will be attached to undamaged gas lines. This results in a huge relight effort for the local utility at a time when resources are typically at a premium. If the earthquake occurs during the winter, a greater life hazard may exist from a lack of gas supply than from potential gas leaks. In the future, as shutoff devices improve and gas-fired appliances which use pilots are phased out, it may be justified to require shutoff devices.

This is not meant to discourage individuals and companies from installing shutoff devices. In particular, individuals and companies who are capable of relighting gas-fired equipment should seriously consider installation of these devices. In addition, gas valves should be closed whenever leaks are detected.

6.4.6 HVAC ductwork. Experience in past earthquakes has shown that, in general, HVAC duct systems are rugged and perform well in strong shaking motions. Bracing in accordance with the Sheet Metal and Air Conditioning Contractors National Association SMACNA 80, SMACNA 95, and SMACNA 98 has been shown to be effective in limiting damage to duct systems under earthquake loads. Typical failures have affected system function only and major damage or collapse has been uncommon. Therefore, industry standard practices should prove adequate for most installations. Expected earthquake damage should be limited to opening of the duct joints and tears in the ducts. Connection details that are prone to brittle failures, especially hanger rods subject to large amplitude cycles of bending stress, should be avoided.

Some ductwork systems carry hazardous materials or must remain operational during and after an earthquake. Such ductwork system would be assigned a value of I_p greater than 1.0. A detailed engineering analysis for these systems should be performed.

All equipment attached to the ducts and weighing more than 75 lb (334 N), such as fans, humidifiers, and heat exchangers, should be braced independently of the duct. Unbraced in-line equipment can damage the duct by swinging and impacting it during an earthquake. Items attached to the duct (such as dampers, louvers, and air diffusers) should be positively supported by mechanical fasteners (not friction-type connections) to prevent their falling during an earthquake.

Where it is desirable to limit the deflection of duct systems under seismic load, bracing in accordance with the SMACNA references listed in Sec. 6.1.1 may be used.

6.4.7 Piping systems. Experience in past earthquakes has shown that, in general, piping systems are rugged and perform well in strong shaking motions. Numerous standards and guidelines have been developed covering a wide variety of piping systems and materials. Construction in accordance with current requirements of the referenced national standards have been shown to be effective in limiting damage to and avoiding loss of fluid containment in piping systems under earthquake conditions. It is therefore the intention of the *Provisions* that nationally recognized standards be used to design piping systems provided that the force and displacement demands are equal to or exceed those outlined in Sec. 6.2.6 and 6.2.7 and provision is made to mitigate seismic interaction issues not normally addressed in the national standards.

The following industry standards, while not adopted by ANSI, are in common use and may be appropriate reference documents for use in the seismic design of piping systems: SMACNA *Guidelines for the Seismic Restraint of Mechanical Systems* and ASHRAE CH 50-95 *Seismic Restraint Design Piping*.

6.4.8 Boilers and pressure vessels. Experience in past earthquakes has shown that, in general, boilers and pressure vessels are rugged and perform well in strong shaking motions. Construction in accordance with current requirements of the *ASME Boiler and Pressure Vessel Code* (ASME BPV) has been shown to be effective in limiting damage to and avoiding loss of fluid containment in boilers and pressure vessels under earthquake conditions. It is therefore the intention of the *Provisions* that nationally recognized codes be used to design boilers and pressure vessels provided that the seismic force and displacement demands are equal to or exceed those outlined in Sec. 6.2.6 and 6.2.7. Until such nationally recognized codes incorporate force and displacement requirements comparable to the requirements of Sec. 6.2.6 and 6.2.7, it is nonetheless the intention to use the design acceptance criteria and construction practices of those codes.

6.4.9 Elevators. The *ASME Safety Code for Elevators and Escalators* (ASME A17.1) has adopted many requirements to improve the seismic response of elevators; however, they do not apply to some regions covered by this chapter. These changes are to extend force requirements for elevators to be consistent with the *Provisions*.

6.4.9.2 Elevator machinery and controller supports and attachments. ASME A17.1 has no seismic requirements for supports and attachments for some structures and zones where the *Provisions* are applicable. Criteria are provided to extend force requirements for elevators to be consistent with the intent and scope of the *Provisions*.

6.4.9.3 Seismic switches. The purpose of seismic switches as used here is different from that of ASME A17.1, which has incorporated several requirements to improve the seismic response of elevators (such as rope snag point guard, rope retainer guards, and guide rail brackets) and which does not apply to some buildings and zones covered by the *Provisions*. Building motions that are expected in these uncovered seismic zones are sufficiently large to impair the operation of elevators. The seismic switch is positioned high in the structure where structural response will be the most severe. The seismic switch trigger level is set to shut down the elevator when structural motions are expected to impair elevator operations.

Elevators in which the seismic switch and counterweight derail device have triggered should not be put back into service without a complete inspection. However, in the case where the loss of use of the elevator creates a life-safety hazard, an attempt to put the elevator back into service may be attempted. Operating the elevator prior to inspection may cause severe damage to the elevator or its components.

The building owner should have detailed written procedures in place defining for the elevator operator/maintenance personnel which elevators in the facility are necessary from a post-earthquake life safety perspective. It is highly recommended that these procedures be in-place, with appropriate personnel training, prior to an event strong enough to trip the seismic switch.

Once the elevator seismic switch is reset, it will respond to any call at any floor. It is important that the detailed procedure include the posting of “out-of-service for testing” signs at each door at each floor, prior to resetting the switch. Once the testing is completed and the elevator operator/maintenance personnel are satisfied that the elevator is safe to operate, the signs can be removed.

6.4.9.4 Retainer plates. The use of retainer plates is a very low cost provision to improve the seismic response of elevators.

RELATED CONCERNS

Maintenance. Mechanical and electrical devices installed to satisfy the requirements of the *Provisions* (for example, resilient mounting components or certain protecting devices) require maintenance to ensure their reliability and to provide protection in case of a seismic event for which they are designed. Specifically, rubber-in-shear mounts or spring mounts (if exposed to weathering) may deteriorate with time and, thus, periodic testing is required to ensure that their damping action will be available during an earthquake. Pneumatic mounting devices and electric switchgear must be maintained free of dirt and corrosion. How a regulatory agency could administer such periodic inspections has not been determined, so requirements to cover these situations have not been included.

Tenant improvements. It is intended that the requirements in Chapter 6 also apply to newly constructed tenant improvements that are listed in Tables 6.3-1 and 6.4-1 and that are installed at any time during the life of the structure.

Minimum standards. Criteria represented in the *Provisions* represent minimum standards. They are designed to minimize hazard for occupants and to improve the likelihood of functioning of facilities required by the community to deal with the consequences of a disaster. They are not designed to protect the owner's investment, and the designer of the facility should review with the owner the possibility of exceeding these minimum standards so as to limit his economic risk.

The risk is particularly acute in the case of sealed, air-conditioned structures where downtime after a disaster can be materially affected by the availability of parts and labor. The parts availability may be significantly worse than normal because of a sudden increase in demand. Skilled labor also may be in short demand since available labor forces may be diverted to high priority structures requiring repairs.

Architect-Engineer design integration. The subject of architect-engineer design integration is raised here because it is believed that all members of the profession should clearly understand that Chapter 6 is a compromise based on concerns for enforcement and the need to develop a simple, straightforward approach. It is imperative that, from the outset, architectural input concerning definition of occupancy classification and the required level of seismic resistance be properly considered in the structural engineer's approach to seismic safety if the design profession as a whole is to make any meaningful impact on the public awareness of this matter. It is hoped that, as the design profession gains more knowledge and sophistication in the use of seismic design, a more comprehensive approach to earthquake design requirements will be developed.

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End note: The American Architectural Manufacturers Association (AAMA) has issued AAMA 501.4-2000: "Recommended Static Test Method for Evaluating Curtain Wall and Storefront Systems Subjected to Seismic and Wind Induced Interstory Drifts." In contrast with the dynamic displacements employed in the crescendo tests described in this section, static displacements are employed in AAMA's recommended test method. Correlations between the results of the static and dynamic test methods have not yet been established with regard to the seismic performance of architectural glazing systems.

Note that this section addresses glass in frames subject to interstory drift. Glass frames not subject to interstory drift, such as freestanding guards and screens, are not covered by this section.