Chapter 6 Commentary

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS DESIGN REQUIREMENTS

6.1 GENERAL: The general requirements establish minimum design levels for architectural, mechanical, electrical, and other nonstructural systems and *components* (hereinafter referred to as "*components*") recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical, electrical, and other nonstructural *components*. Several exemptions are made 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 are 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 fell.

The seismic force on any *component* shall be applied at the center of gravity of the *component* and shall be assumed to act in any horizontal direction. Vertical forces on architectural *components* are specified in Sec. 6.1.3. Vertical forces on mechanical and electrical *components* are specified in Sec. 6.3.2.

In the design and evaluation of support structures and the attachment of the architectural *component*, flexibility should be considered. *Components* that are subjected to seismic relative displacements (i.e., *components* that are connected to both the floor and ceiling level above) should be designed with adequate flexibility to accommodate imposed displacements. 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 load path in the *components* should be considered especially in the region near the anchor points.

Although the *components* included in Tables 6.2.2 and 6.3.2 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 6.2.2 or 6.3.2 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 nonload-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*.

6.1.2 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.

The calculation of forces as prescribed in Sec. 6.1.3 recognizes the unique dynamic and structural characteristics of the *components* as compared to *structures*. *Components* typically lack attributes of *structures*, i.e., ductility, toughness, and redundancy, which factor in to the calculation of reduced lateral design forces. This is reflected in the lower values for R_p given in Tables 6.2.2 and 6.3.2, 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 6.2.2 and 6.3.2. 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 lateral load resisting system. It is the intent of this provision that *component* forces be accommodated in the *structure* design 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 lateral load resisting system, it is generally sufficient for verification of a complete load path to only check for local overstress conditions in the vicinity of the *component* in question. 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 increase of 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, and be positively fastened to the supporting *structure*.

For the purposes of the load path check, it is essential that detailed information on 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. Note, until the *component* is ordered, the exact size and location of loads will generally not be known. Therefore, the designer should make conservative assumptions in the design of the supporting structural elements. The design of the 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* (e.g., tanks, 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 (rubber-in-shear, springs, air cushions). Heavy mechanical equipment (e.g., 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 (e.g., switchgear, 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 the vertical ground motions. Because frictional resistance cannot be relied upon, positive restraint must be provided for each *component*.

6.1.3 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 with changes in Chapter 4. BSSC Technical Subcommittee 8 believes that Eq. 6.1.3-1 through 6.1.3-3 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 C 6.1.3-1 is from the NCEER work and is an acceptable formulation for a_p as a function of T_p/T. Minor adjustments from the 1994 *Provisions* have been made in the tabulated a_p values 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

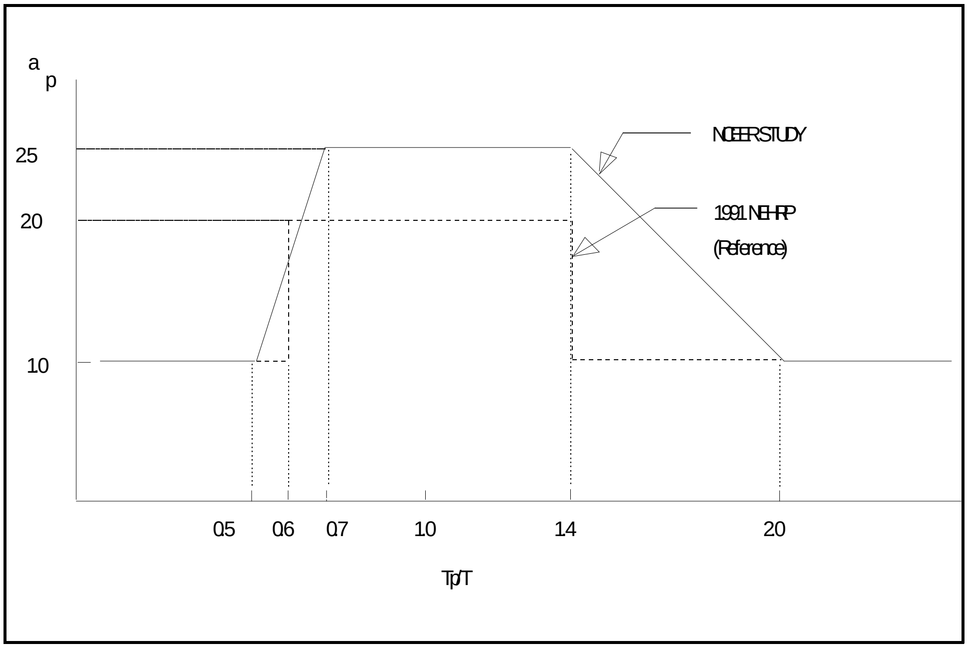


FIGURE C6.1.3-1 NCEER formulation for a_p as function of structural and component periods.

Minor adjustments from the 1994 *Provisions* have been made in the tabulated R_p values 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).

Eq. 6.1.3-1 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 will include 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 C6.1.3-2). 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 C6.1.3-3). 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. 6.1.3-3 provides a lower bound in this region.

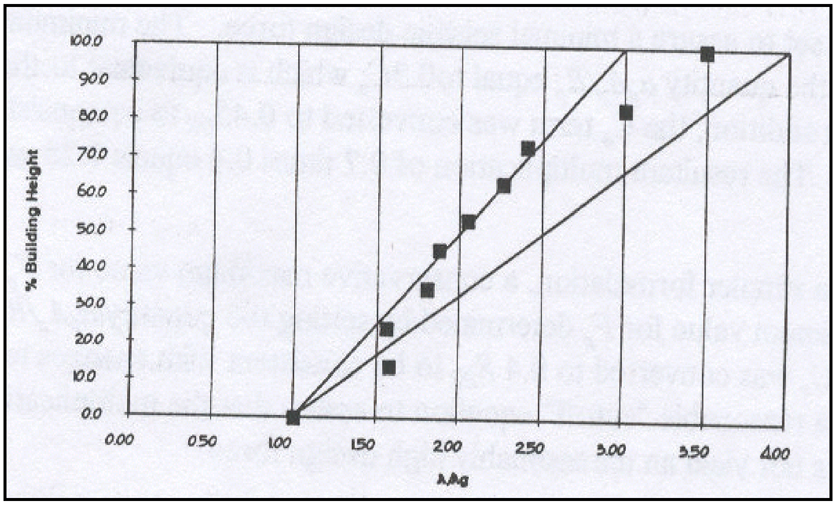


FIGURE C6.1.3-1 Revised NEHRP equation vs (Mean + 1σ) acceleration records -all sites.

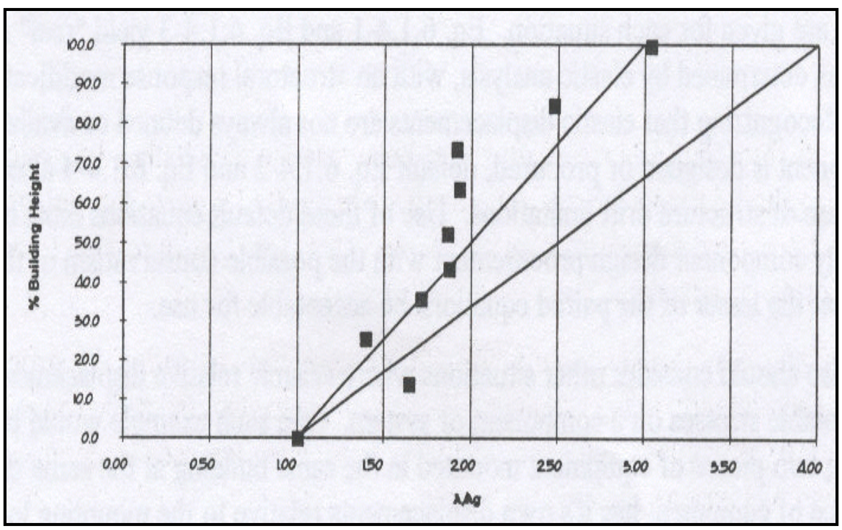


FIGURE C6.1.3-3 Revised NEHRP equation vs (Mean +1σ) acceleration records - sites with A_g ≥0.1g.

Examination of the same data indicates that the in-structure accelerations do not decrease with larger building periods as might be expected from reviewing typical response spectra. One reason for invalidating the traditional response spectra shape might be that structures with longer fundamental periods may have designs governed by drift requirements. These structures would be stiffer with more elastic capacity and also may have lower damping at higher acceleration responses. Also, site soil amplifications are greater at longer periods than at shorter periods. As a result of these studies, the structural period effect introduced into the 1994 *Provisions* for *components* has been removed from the 1997 *Provisions*.

A lower limit for F_p is set to assure a minimal seismic design force. The minimum value for F_p determined by setting the quantity a_pA_p/R_p equal to$0.7C_a$ which is equivalent to the minimum used in current practice. In addition, the C_a term was converted to $0.4S_{DS}$ to be consistent with changes to Chapter 1. The resultant multiplication of 0.7 times 0.4 equals 0.28 was rounded to 0.3 for simplicity.

To meet the need for a simpler formulation, a conservative maximum value for F_p also was set. Eq. 6.1.3-2is the maximum value for F_p determined by setting the quantity a_pA_p/R_p equal to 4.0. In addition, the term C_a was converted to 0.4 S_{DS} to be consistent with changes to Chapter 4. Eq. 6.1.3-2also 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. 2.2.6. The value for F_p calculated in accordance with Chapter 6 should be substituted for the value of Q_E in Sec. 2.2.6.

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

Alternatively for each point of attachment a force F_p shall be determined based on Eq. 6.1.3-1. Minimums and maximums of Eq. 6.1.3 shall 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 *components* mass distribution over the area used to establish the tributary weight (e.g. for 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. 5.2.6.3.3, each anchorage force should be based on simple statics 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.

6.1.4 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 to support 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. 6.1.4-1 and Eq. 6.1.4-3 yield "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. 6.1.4-2 and Eq. 6.1.4-4 also are provided that allow the use of structure drift limitations. 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.

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 or avoiding local yielding. It is further the intent of the *Provisions* to consider the effects of seismic support relative displacements and displacements caused by seismic force on mechanical and electrical *component* assemblies such as piping systems, cable and conduit systems, and other linear systems, most typically, 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.

6.1.5 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. 6.3.13 and 6.3.14 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 unacceptable consequences of failure are anticipated.

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, will fall to the ground.

Special consideration should be given *components* that could block means of egress or exitways apply to items that, if they fall during an earthquake, could block the means of egress for the occupants of the structure. The term "means of egress" has been defined 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 passage ways, exit courts, and yards. Example items that should be included when considering egress include walls around stairs, corridors, 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 the *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).

Sec. 1.3.1 requires 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. 6.3.13 and 6.3.14, since operation of these secondary items is not critical to the post-earthquake operability of the structure.

Until recently, storage racks were primarily installed in low-occupancy ware houses. With the recent proliferation of warehouse-type retail stores, it has been judged necessary to address the relatively greater seismic risk that storage racks may pose to the general public, compared to more conventional retail environments. Under normal operating conditions, retail stores have a far higher occupancy load than an ordinary warehouse of a reasonable size. Failure of a storage rack system in the retail environment is much more likely to cause personal injury than a similar failure in a storage warehouse. Therefore, to provide an appropriate level of additional safety in areas open to the public, Sec 6.1.5 now requires that storage racks in occupancies open to the general public should be designed with an I_p value equal to 1.50. Storage rack contents, while beyond the scope of the *Provisions* pose a potentially serious threat to life should they fall from the shelves in an earthquake. Restraints should be provided to prevent the contents of rack shelving open to the general public from falling in strong ground shaking.

6.1.6 Component Anchorage: In general, it is not recommended that anchors be relied upon for energy dissipation. Inasmuch as the anchor represents the transfer of load from a relatively deformable material (e.g., steel) to a low deformability material (e.g., concrete, masonry), the boundary conditions for ensuring deformable, energy-absorbing behavior in the anchor itself are at best difficult to achieve. 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 for balancing of steel strength and deformability in the anchor with variable edge distances and anchor spacings. The restriction on R_p values for shallow anchors is because of the concern for low deformation failure modes in the *component* anchorage. Anchorages that can be reasonable expected to fail in a low deformation manner should be designed using $R_p = 1.5$. Shallow anchors are defined as those anchors that have an embedment length diameter ratio of less than 8.

For purposes of the *Provisions*, chemical anchors are intended to include post installed metal fasteners which are inserted into holes in concrete or masonry and held in place by epoxy, resins or other chemicals. Adhesive anchorages are intended to include plates, angles, or other structural elements adhered to surfaces such as computer access floor base plates.

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 through 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, 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 as such. 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 or single anchor connections, it is considered necessary to use at least two anchors in any load-carrying device whose failure might lead to collapse.

Tests have shown that there are consistent shear ductility variations between bolts anchored to drilled or punched plates with nuts and connections using welded, headed studs. Recommendations for design are not presently available but 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.

6.1.6.5: Generally, powder driven fasteners in concrete tend to exhibit variations in load capacity that are somewhat larger than post-drilled anchors and do not provide the same levels of reliability even though some installation methods allow for the same reliability as post-drilled expansion anchors. As such, their qualification under a simulated seismic test program should be demonstrated prior to use. Such fasteners, when properly installed in steel, are reliable, showing high capacities with very low variability.

6.1.7 Construction Documents: It is deemed important by the committee that there be a clearly defined basis for each quality assurance activity specified in Chapter 3. As result *construction documents* are required for all *components* requiring special inspection or testing in Chapter 3.

It is also deemed important by the committee that there be some reasonable level of assurance that the construction and installation of *components* be consistent with the basis of the supporting seismic design. 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 the use by the multiple trades and suppliers to follow in the course of construction.

6.2 ARCHITECTURAL COMPONENT DESIGN:

6.2.1 General: The primary focus of the *Provisions* is on the design of attachments, connections, and supports for architectural *components*.

"Attachments" are means by which *components* are secured or restrained to the seismic force resisting system of the structure. Such attachments and restraints may include anchor bolting, welded connections, and fasteners.

"Architectural *component* supports" are those members or assemblies of members, including braces, frames, struts and attachments, that transmit all loads and forces between the *component* and the building structure. Architectural *component* supports also transmit lateral forces and/or provide structural stability for the *component* to which they connect.

The requirements 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 in which *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.

6.2.2 Architectural Component Forces and Displacements: *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 glazing, partitions, stairs, and veneer.

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.

6.2.3 Architectural Component Deformation: 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* has a low life-safety hazard. 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 shear wall structures.

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

6.2.4 Exterior Nonstructural Wall Elements and Connections: The *Provisions* requires that nonbearing wall panels that are attached to or enclose the structure shall be designed to resist the (inertial) forces and shall 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 such that, if they were to yield, they would do so in a high deformation manner without loss of load-carrying capacity. Between points of connection, panels should be separated from the building structure to avoid contact under seismic action.

The *Provisions* document requires allowance for story drift. This required allowance can be 2 in. (51 mm) or more from one floor to the next and may present a greater challenge to the registered design professional than requirements for the forces. In practice, separations between panels are usually limited to about 3/4 in. (19 mm), with the intent of limiting contact, and hence panel alignment disruption and/or damage under all but extreme building response, and providing for practical joint detailing with acceptable appearance. The *Provisions* calls for a minimum separation of ½ in. (13 mm). The design should respect 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. The isolation can be achieved by using slots, but the use of long rods that flex is preferable because this approach is not dependent on installation precision to achieve the desired action. The rods must be designed to carry tension and compression in addition to induced flexural stresses. For floor-to-floor wall panels, the panel usually is rigidly fixed to and moves with the floor structure nearest the panel bottom. In this condition, the upper attachments become isolation connections to prevent building movement forces from being transmitted to the panels. and thus the panel translates with the load supporting structure. The panel also can be supported at the top with the isolation connection 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.

The *Provisions* requires 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.

To minimize the effects of thermal movements and shrinkage on architectural cladding panels, the connection system generally is statically determinant. As a result, cladding panel support systems often lack redundancy and failure of a single connection can have catastrophic consequences.

6.2.5 Out-of-Plane Bending: Most walls are subject to out-of-plane forces when a building is subjected to 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 and/or low flexural strength materials. The conventional limits based upon deflections as a proportion of the span may be used with the applied force as derived in Sec. 6.2.2.

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 suffer 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.

6.2.6 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 California earthquakes has raised two concerns:

a. The support of the individual panels at walls and expansion joints, and

b. The interaction with fire sprinkler systems.

The alternate methods provided have been developed in a cooperative effort by registered design professionals, the ceiling industry, and the fire sprinkler industry in an attempt to address these concerns. It is hoped that further research and investigation will result in further improvements in future editions of the *Provisions*.

Consideration shall be given to the placement of seismic bracing and 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 wire even with the vertical compression strut 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 adequate width of closure angles and penetration clearance was provided.

The lateral seismic restraint for a non-rigidly braced suspended ceiling is primarily provided by the ceiling coming in 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.

6.2.7 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 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.

It is acceptable practice for systems with floor stringers 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.

Overturning 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 plate. The maximum concurrent vertical load when considering overturning effects is therefore limited to the 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 behavior, 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.

6.2.8 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.

6.2.9 Steel Storage Racks: Storage racks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.3.3.

6.2.10 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions: Glass performance in earthquakes can fall into one of four categories:

a. The glass remains unbroken in its frame or anchorage.

b. The glass cracks but remains in its frame or anchorage while continuing to provide a weather barrier, and be otherwise serviceable.

c. The glass shatters but remains in its frame or anchorage in a precarious condition, liable to fall out at any time.

d. The glass falls out of its frame or anchorage, either in fragments, shards, or whole panels.

Categories a. and b. provide both life safety and immediate occupancy levels of performance. In the case of category b., 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 c. and d. 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.

6.2.10.1 General: Eq. 6.2.10.1-2 is derived from *Earthquake Safety Design of Windows*, published in November 1982 by the Sheet Glass Association of Japan. Eq. 6.2.10.1-2 is derived from a similar equation in Bouwkamp and Meehan (1960) that permits calculation of the interstory drift required to cause glass-to-frame contact in a given rectangular window frame. Both equations 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 interstory 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 Eqs. 6.2.10.1-1 and 6.2.10.1-2 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 Eqs. 6.2.10.1-1 and 6.2.10.1-2.

6.2.10.2 Seismic Drift Limits for Glass Components

Introduction

Seismic design requirements for glass in building codes have traditionally been non-existent or 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 interstory 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 Facility and Experimental Plan

In-plane dynamic racking tests were performed using the facility shown in Figure C6.2.10.2-1. 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 interstory 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 x 1.8 m (6 ft) high. A more detailed description of the dynamic racking test facility is included in Behr and Belarbi (1996).

Several types of glass, shown in Table C6.2.10.2-1, 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 dry-glazed within a wall system 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 C6.2.10.2-1, 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 C6.2.10.2-1. - 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		✓

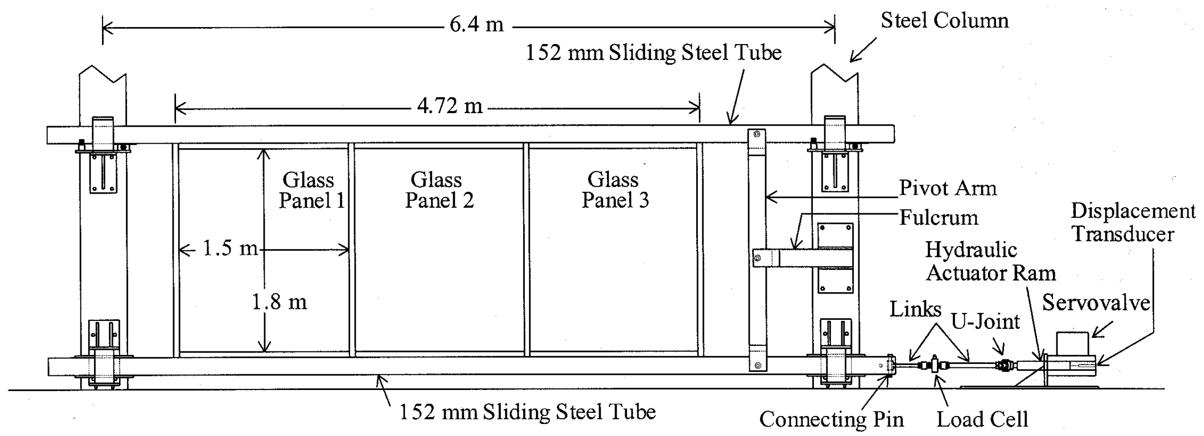


Figure C6.2.10.2 - 1 - Dynamic Racking Test Facility (1 m = 3.28 ft)

FIGURE C6.2.10.2-1 Dynamic racking test Facility.

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 (i.e., 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 (i.e., 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 C6.2.10.2-1 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 (i.e., 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" (i.e., 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 facility.

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 "$\Delta_{fallout}$" in Sec. 6.2.10.1 of the *Provisions*, noting that horizontal racking displacements (i.e., 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. 6.2.10.1-2.

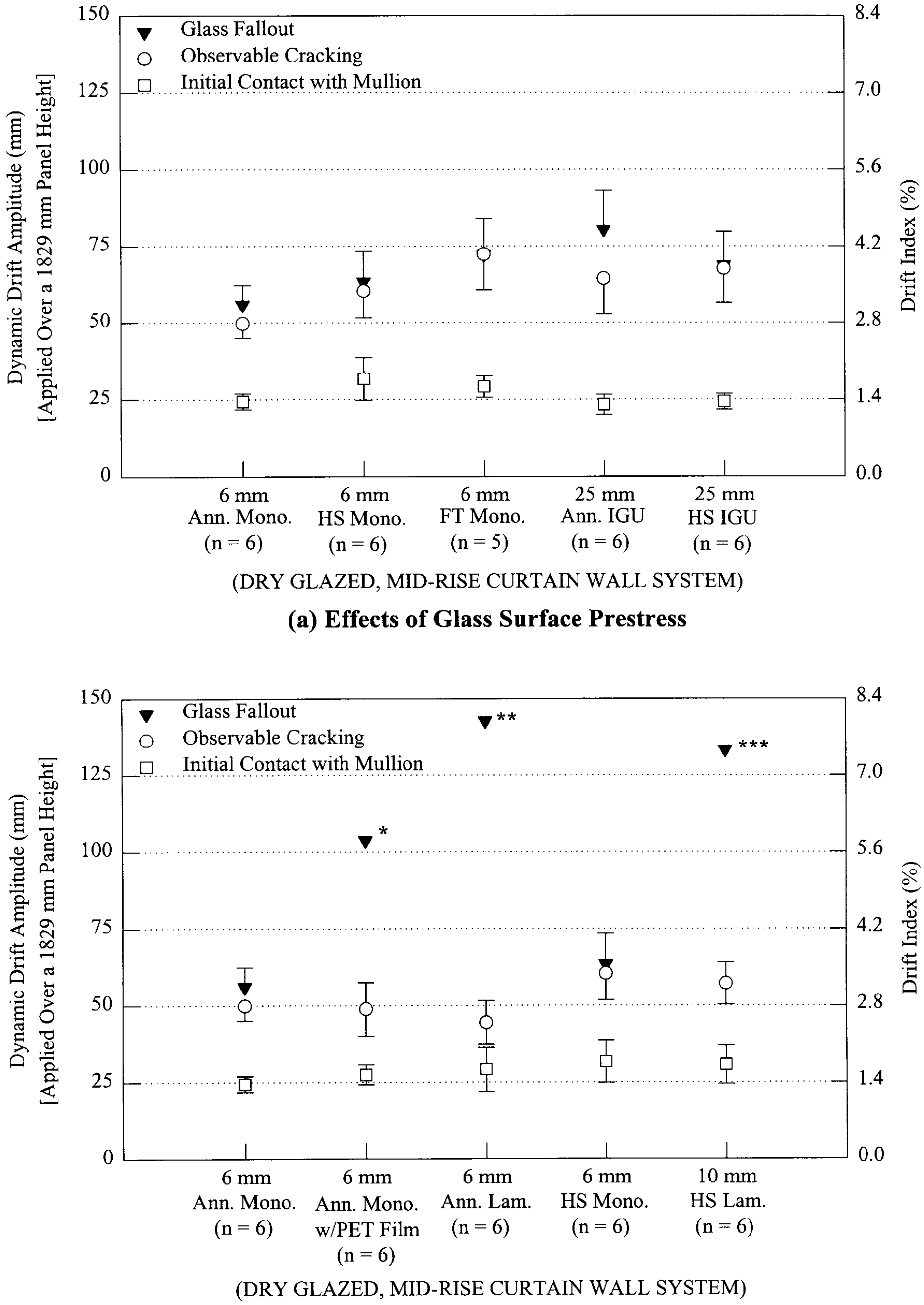
Glass Failure Patterns From Crescendo Tests

Glass failure patterns were recorded during each storefront test and mid-rise test. Annealed monolithic glass tended to fracture into sizeable 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 windows in Figure C6.2.10.2-2. Figure C6.2.10.2-2a shows the effects of glass surface prestress (i.e., annealed, heat-strengthened and fully tempered glass) on seismic drift limits; Figure C6.2.10.2-2b shows the effects of lamination (i.e., monolithic glass, monolithic glass with unanchored 0.1 mm PET film, and laminated glass); Figure C6.2.10.2-2c shows the effects of wall system type (i.e., lighter, more flexible, storefront wall system versus the same glass types tested in a heavier, stiffer, mid-rise wall system); and Figure C6.2.10.2-2d shows the effects of structural silicone glazing (i.e., dry glazing versus two-side structural silicone glazing). Each symbol plotted in Figure C6.2.10.2-2 is the mean value for specimens of a given glass type, along with ± 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 C6.2.10.2-2. (This ultimate drift limit, shown with a "▾" symbol in Figure C6.2.10.2-2, is related to the term "$\Delta_{fallout}$" in Sec. 6.2.10.1 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.).

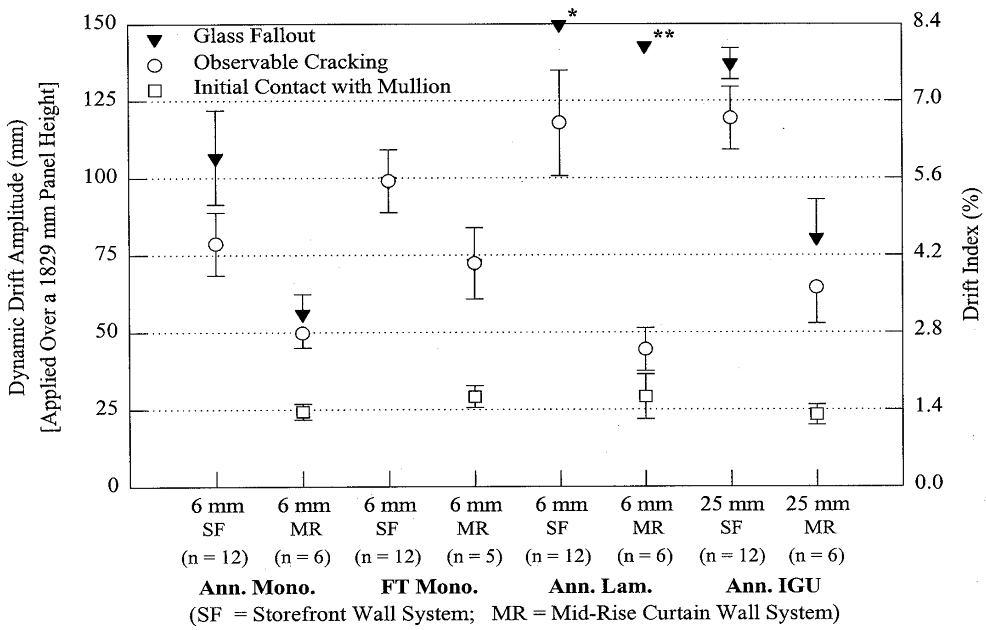
The ±152 mm (± 6 in.) racking limit of the test facility, when applied over the 1829 mm (72 in.) height of glazing panel specimens represents a severe interstory drift index of over 8 percent. This 8 percent drift index exceeds, by a significant margin, provisions in Sec. 5.2.8 (Table 5.2.8) that set 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 5.2.8 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 interstory drifts exhibited by the primary building structure in order to provide an acceptable safety margin against glass fallout.



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

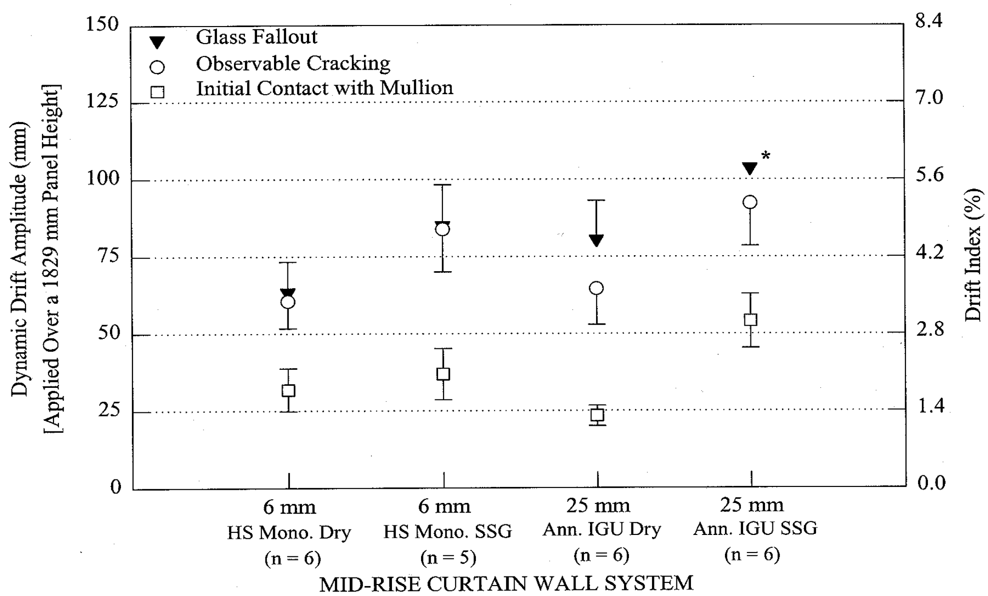
(b) Effects of Lamination

FIGURE C6.2.10.2 - 2 - Seismic Drift Limits from Crescendo Tests on Architectural Glass



*8 of 12 specimens did not fall out. **5 of 6 specimens did not fall out.

(c) Effects of Wall System Type



*1 of 6 specimens did not fall out.

(d) Effects of Structural Silicone Glazing

FIGURE C6.2.10.2 - 2 (continued) - Seismic Drift Limits from Crescendo Tests on Architectural Glass

Summary Observations From Figure C6.2.10.2-2:

(a) Effects of Glass Surface Prestress - Figure C6.2.10.2-2a 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.

(b) Effects of Lamination - Figure C6.2.10.2-2b 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 (i.e., Δ_{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.

(c) Effects of Wall System Type - Figure C6.2.10.2-2c 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.

(d) Effects of Structural Silicone Glazing - As shown in Figure C6.2.10.2-2d, 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 C6.2.10.2-2d 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.

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Acknowledgment

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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.

6.3 MECHANICAL AND ELECTRICAL COMPONENT DESIGN:

6.3.1 General: 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.

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 Glossary as "equipment supports" and "attachments." For the second safety level, failure of the mechanical or electrical equipment itself poses a significant hazard. In this case, failure could either be to a containment having hazardous contents or contents required after the earthquake or failure could be functional to 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; however, numerous seismic failures of large flat-bottom tanks and the hazard of a large fluid spill suggest that many, if not most, of these should be. Distinguishing between large and small, in this case, may require an assessment of potential damage caused by a spill of the fluid contents over and above the guidance offered in Sec. 6.3.9.

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 only affects structural integrity and stability directly. Function and operability of mechanical and electrical *components* may only indirectly be affected 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 concluded that if structural integrity and stability are maintained, function and operability after an earthquake will be reasonably provided for most 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 after the earthquake leaktightness is re-established. 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. 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.

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. When 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 ampleness of construction that renders such equipment 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 equipment ruggedness assessment then will determine the need for an appropriate method and extent of the 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 as it is to be expected that the developers have a greater familiarity with the expected failure modes of the *components* for which their design and construction rules are developed. 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* has sufficient strength (especially if constructed of ductile materials) to not break up or break away from its supports in such a way as to provide a life-safety hazard. Earthquake damage surveys confirm this.

Specific guidance for selected *components* or conditions is provided in Sec. 6.3.6 through 6.3.16.

6.3.2 Mechanical and Electrical Component Forces and Displacements: *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.

6.3.3 Mechanical and Electrical 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 (i.e., 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 for the highest fundamental period dominant mode of vibration to be excited by in-situ tests, relatively significant input levels of motion are required (i.e., the flexibility of the base and attachment needs to be exercised).

Many types of mechanical equipment *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 and narrow motor control centers and switch-boards lie in the upper end of this period range. Low and medium-voltage switchgear, transform-ers, 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.

6.3.4 Mechanical and Electrical Component 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 supports' conditions. However, *components* made of less ductile materials can only accommodate relative displacement effects by providing flexibility or flexible connections.

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 not so prone to damage and are not so likely to fracture and fall. This is valid if the supports can accommodate the imposed loads.

6.3.5 Component Supports: It is the intent of these requirements to ensure that all mechanical and electrical *component* supports, the means by which a *component* transfers seismic loads to the structure, 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.

6.3.6 Component Certification: It is intended that the certificate only be requested for *components* with an importance factor (I_p) greater than 1.00 and only if the *component* has a doubtful or uncertain seismic load path. This certificate should not be requested to validate functionality concerns.

In the context of the *Provisions*, seismic adequacy of the *component* is of concern only when the *component* is required to remain operational after an earthquake or contains material that can pose a significant hazard if released. Meeting the requirements of this section shall be considered as an acceptable demonstration of the seismic adequacy of a *component*.

6.3.7 Utility and Service Lines at Structure Interfaces: 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. 6.3.7 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. 5.2.2 and should be in accordance with *Provisions* Sec. 6.1.4.

Consideration may be necessary for nonessential piping carrying quantities of materials that could, if the piping is ruptured, damage essential utilities.

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 tend to be 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 the device 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.

6.3.9 Storage Tanks: Storage tanks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.7.3.

6.3.10 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 HVAC, SMACNA Rectangular, SMACNA Restraint 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 bending stress cycles, should be avoided. Some ductwork systems carry hazardous materials or must remain operational during and after an earthquake. These ductwork system would be designated as having an I_p greater than 1.0. A detailed engineering analysis for these systems should be performed.

All equipment (e.g., fans, humidifiers, and heat exchangers) attached to the ducts and weighing more than 75 lb (334 N) should be braced independently of the duct. Unbraced in-line equipment can damage the duct by swinging and impacting it during an earthquake. Items (e.g., dampers, louvers, and air diffusers) attached to the duct 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.

6.3.11 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 standard shave 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 demand is equal to or exceeds the requirements of Sec. 6.1.3 and 6.1.4 and provisions are 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* ASHRAE CH 50-95 *Seismic Restraint Design Piping,* as used herein, are assemblies of pipe, tubing, valves, fittings, and other in-line fluid containing *components,* excluding their attachments and supports.

6.3.12 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 demand is equal to or exceeds the requirements of Sec. 6.1.3 and 6.1.4. Until such nationally recognized codes incorporate force and displacement requirements comparable to the requirements of Sec. 6.1.3 and 6.1.4, it is nonetheless the intention to use the design acceptance criteria and construction practices of those codes.

Boilers and pressure vessels as used herein are fired or unfired containments, including their internal and external appurtenances and internal assemblies of pipe, tubing, and fittings, and other fluid containing *components*, excluding their attachments and supports.

6.3.13 Mechanical Equipment Attachments and Supports: Past earthquakes have demonstrated that most mechanical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the design of mechanical equipment items for operational and transportation loads typically envelopes loads due to earthquake. For this reason, the requirements primarily focus on equipment anchorage and attachments. It was felt, however, that mechanical equipment *components* required to maintain containment of flammable or hazardous materials should themselves be designed for seismic forces.
In addition, there liability of equipment operability after an earthquake can be increased if the following items are also considered in design:

a. Internal assemblies are attached with a sufficiency that eliminates the potential of impact with other internal assemblies and the equipment wall; and
b. Operators, motors, generators, and other such *components* functionally attached 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.

6.3.14 Electrical Equipment Attachments and Supports: Past earthquakes have demonstrated that most electrical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the design of electrical equipment items for operational and transportation loads typically envelopes loads due to earthquake. For this reason, the requirements primarily focus on equipment anchorage and attachments. However, reliability of equipment operability after an earthquake can be increased if the following items also are considered in design:

a. Internal assemblies are attached with a sufficiency that electrical subassemblies and contacts will not be subject to differential movement or impact between the assemblies, contacts, and the equipment enclosure.
b. Any ceramic or other nonductile *components* in the seismic load path should be specifically evaluated.
c. Adjacent electrical cabinets are bolted together and cabinet lineups are prevented from banging into adjacent structural members.

6.3.15 Alternate Seismic Qualification Methods: Testing is a well established alternative method of seismic qualification for small to medium size equipment. Several national standards, other than IEEE 344 (IEEE-344), have testing requirements adaptable for seismic qualification.

6.3.16 Elevator Design Requirements: 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*.

6.3.16.2 Elevator Machinery and Controller Supports and Attachments: The ASME *Safety Code for Elevators and Escalators* (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*.

6.3.16.3 Seismic Controls: The purpose of the seismic switch as used here is different from that provided under the ASME *Safety Code for Elevators and Escalators* (ASTM C635), which has incorporated several requirements to improve the seismic response of elevators (e.g., rope snag point guard, rope retainer guards, guide rail brackets) that do 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 directing 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.

6.3.16.4 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* (e.g., resilient mounting *components* or certain protecting devices) require maintenance to ensure their reliability and provide the 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 was not determined and, hence, requirements to cover this situation 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 6.2.2 and 6.3.2 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 being raised 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 integrated with the approach of the structural engineer to seismic safety if the design profession as a whole is to make any meaningful impact on the public conscience in this issue. Accordingly, considerable effort was spent in this area of concern. It is hoped that as the design profession gains more knowledge and sophistication in the use of seismic design, it will collectively be able to develop a more comprehensive approach to earthquake design requirements.

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