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EARTHQUAKE RESISTANT DESIGN OF SECONDARY STRUCTURES: A REPORT ON THE STATE OF THE ART

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ABSTRACT

An account is made of developments in the last three decades in the area of seismic design of mechanical and electrical equipment, architectural components, and other secondary structures and nonstructural components attached to the floors and walls of buildings. A description is made of what constitutes a secondary structure, its importance, and their performance during past earthquakes. A description is also made of the characteristics that make these systems particularly vulnerable to the effects of earthquakes, and their know response characteristics. In addition, a description is made from a historical point of view of the methods of analysis that have been proposed and are available to improve and simplify their analysis, the methods that are currently used in practice as reflected by various building codes and seismic provisions, and the experimental studies and field observations that have been carried out to further investigate some of their dynamic properties and response characteristics. The review closes with a summary of what research is still needed to advance the efforts to protect these systems against the effects of earthquakes and to develop methods and techniques to achieve this goal in a practical and economical way.

KEYWORDS

Secondary systems; nonstructural components, seismic equipment design, architectural elements; building contents; building attachments.

INTRODUCTION

Secondary structures are those systems and elements housed or attached to the floors and walls of a building or industrial facility which are not part of the main or intended load-bearing structural system for the building or industrial facility, but may also be subjected to large seismic forces and depend on their own structural characteristics to resist these seismic forces. In general, these secondary structures may be classified into three broad categories: (a) architectural components, (b) mechanical and electrical equipment, and (c) building contents. Examples in the first category are: elevator penthouses, stairways, partitions, parapets, heliports, cladding systems, signboards, lighting systems, and suspended ceilings. Some in the second are: storage tanks, pressure vessels, piping systems, ducts, escalators, smokestacks, antennas, cranes, radars and object tracking devices, computer and data acquisition systems, control panels, transformers, switchgears, emergency power systems, fire protection systems, boilers, heat exchangers, chillers, cooling towers, and machinery such as pumps, turbines, generators, engines and motors. And some of those that can be considered in the third are: bookshelves, file cabinets, storage racks, decorative items, and any other piece of furniture commonly found in office buildings and warehouses. Alternative names by which these systems are also known are "nonstructural components", "nonstructural elements," "building attachments", "architectural , mechanical, and electrical elements," "secondary systems and "secondary structural elements." The name that in this writer's opinion best describe their nature is the last one, since it reflects the fact that they are not part of the main structure but must possess, nevertheless, structural properties to maintain their own integrity.

In spite of their name, secondary structures are far from being secondary in importance. It is nowadays widely recognized that the survival of secondary structures is essential to provide emergency service in the aftermath of an earthquake. Experience from past earthquakes has shown that the failure of equipment and the debris caused by falling objects and overturned furniture may critically affect the performance of fire and police stations, emergency command centers, communication facilities, power stations, water supply and treatment plants, and hospitals. For example, during the 1994 Northridge earthquake in Los Angeles, California, area, several major hospitals had to be evacuated, not because of structural damage, but because of water damage caused by the failure of water lines and water supply tanks; the failure of emergency power systems; and heating, ventilation, and air conditioning units; damage to suspended ceilings and light fixtures; and some broken windows (Hall, 1994). Along the same lines, it is now recognized that damage to secondary structures represents a threat to life safety, may seriously impair a building 's function, and result in major direct and indirect economic losses. Understandably, the collapse of suspended light fixtures, hung ceilings, or partition walls; the fall of cladding components, parapets, signboards, ornaments, or pieces of broken glass; the overturning of heavy equipment, bookshelves, storage racks; and pieces of furniture; and the rupture of pipes and containers with toxic materials are all capable of causing serious injury or death. A most unfortunate demonstration that this may indeed happen is the death of a student who during the 1987 Whittier Narrows earthquake in California was struck by a falling precast panel while walking out of a parking structure (Taly, 1988). Similarly, it is easy to see that the normal activities that take place in a building may be critically disrupted when some essential equipment fails or when debris from failed architectural components gets in the way. Typical examples which illustrate the consequences of such an event are the unwanted solidification of a melted metal in an industrial facility, the inaccessibility of financial records in a timely manner in a banking institution, and the failure to fill pending orders in a manufacturing plant. In regard to the economic impact caused by the failure of nonstructural components, there is plenty of evidence that shows that, because of the loss of the nonstructural components themselves, loss of inventory, and loss of business income, the cost of such failures may easily exceed the replacement cost of the building (EERI, 1984). And in today's high-tech environment, this cost may be even further as a result of the widespread use of electronic and computer equipment and dependence of industry in this type of equipment. (See Fig. 1). It is clear, thus, that secondary structures should be the subject of a rational and careful seismic design in much the same way as their supporting structures are, and be the continuing object of a careful assessment of their performance after a strong earthquake.

PERFORMANCE DURING PAST EARTHQUAKES

Damage to secondary structural elements has been observed during the earthquake of Alaska in 1964 (Ayres *et al.*, 1973); Venezuela in 1967 (Hanson and Degenkolb, 1976); San Fernando, California in 1971 (Jennings, 1971; Lew *et al.*, 1971; Ayres and Sun, 1973); Managua in 1972 (EERI, 1984); Friuli, Italy in 1976 (Stratta and Wyllie, 1979); Imperial County, California in 1979 (Brandow and Leeds, 1980); Livermore, California in 1980 (EERI, 1984); Coalinga, California in 1983 (Scoll and Stratta, 1984); Whittier Narrows, California in 1987 (Schiff, 1988; Taly, 1988); Michoacan, Mexico in 1985 (Eder and Swan, 1987; Goodno *et al.*, 1989; Lagorio, 1990); Loma Prieta, California in 1989 (Benuska, 1990); Northridge, California in 1994 (Hall, 1994); and Kobe, Japan in 1995 (Comartin *et al.*, 1995). For the most part, this damage has consisted of: (a) fallen ceiling panels (see Fig. 2); (b) fallen light fixtures (see Fig. 3); (c) damage to and fallen cladding components (see Fig. 4); (d) damage to and collapse of masonry partition walls; (e) collapse of penthouses and roof tanks (see Fig. 5); collapse of stadium scoreboard (see Fig. 6); (f) fallen ornaments (see Fig. 7); (g) broken and displaced pipes (see Fig. 8) and HVAC ducts; (h) overturned transformers, control panels and cabinets, storage racks, bookshelves (see Fig. 9), and computer equipment; (i) failure of pressurized gas storage tanks; (j) buckled crane rails; (k) sliding of unanchored equipment; (l) damage to equipment anchorages; (m) collapse of raised computer floors;

Fig. 1 Communication equipment on building

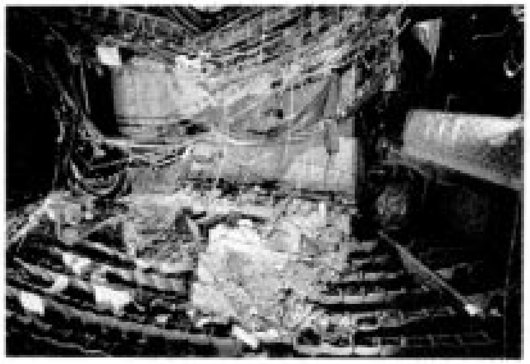

Fig. 2 Ceiling collapse in theater (From Benuska, 1990)

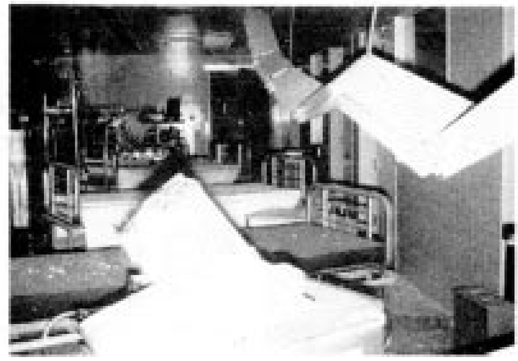

Fig. 3 Collapsed light fixtures (After Stratta, 1987)

and (n) fallen batteries from battery racks. The damage has been extensive during the Alaska, San Fernando, and Nicaragua, but has been consecutively less in recent years as a result of the lessons learned from these earthquakes and the growing awareness of architects, structural engineers, mechanical engineers, electrical engineers, contractors, equipment manufacturers, building officials, and owners about the seismic vulnerability of these elements. For example, measures are now taken to brace suspended ceilings and raised computer floors; to detached exterior panels, infill walls, partitions, and stairways from the structure; to connect cladding and curtain walls in a way that allows building distortions without failing (i.e., using slip joints); and to anchor or restrain equipment to the structure or to install it with isolation devices. Nevertheless, it is likely that damage will occur in future earthquakes given that many old buildings have not been retrofitted to modern standards and some inadequacies exist in the seismic provisions of current building codes.

Fig. 4 Damage to cladding components (After Lagorio, 1990)

Fig. 5 Penthouse failure

Fig. 6 Scoreboard collapse in stadium

Fig. 7 Fallen signboard (From Comartin et al. (1995)

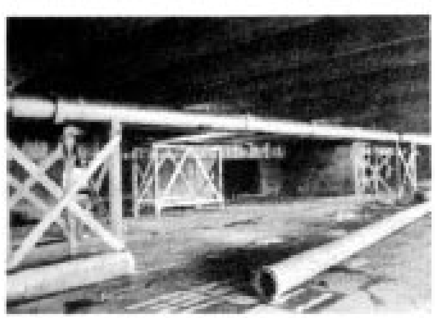

Fig. 8 Collapsed pipe in parking garage (After Rihal, 1990)

Fig. 9 Damaged bookshelves (After Taly, 1988)

GENERAL PHYSICAL CHARACTERISTICS

There are several physical characteristics of the secondary structural elements in buildings that make them particularly vulnerable to the effects of earthquakes. These are:

(1) They are usually attached to the elevated portions of the building and, thus, they are subjected not to the ground motion generated by the earthquake, but to the amplified motions generated by the dynamic response of the building.

(2) Their weight is light in comparison with the weight of the structure to which they are connected, and their stiffness is also much smaller that that of the structure as a whole. As a result it is likely that their natural frequencies are close to the natural frequencies of the structure and, hence, their dynamic response to the motion at their supports may be extraordinarily high.

(3) Their damping ratios may be quite low, much lower than those for the structure, and thus they do not possess the damping characteristics that are necessary for protection against sharp resonant motions.

(4) They may be connected to the structure at more than one point and therefore they may be subjected to the distortions induced by the differential motion of their supports.

(5) They are designed to perform a function other than to resist forces. As such, they are built with materials that are far from the ideal materials to resist seismic forces and they may possess parts that are sensitive to even the smallest level of vibration.

GENERAL RESPONSE CHARACTERISTICS

The special physical characteristics described above make secondary structural elements not only susceptible to earthquake damage, but also make their response to earthquake ground motions peculiar and different from that of a building structure. That is, the response of a secondary structural element exhibits characteristics that are not common in the response of a structure. Some of these characteristics are the following:

(1) The response of a secondary element depends on the response of the structure to which is connected, and thus it depends not only on the characteristics of the ground motion that excites the base of the structure, but also on the dynamic characteristic of the structure.

(2) The response of a secondary element depends on its location within the structure. As a result, identical elements respond differently to the effects of an earthquake if they are located at different levels of the structure.

(3) There may be a significant interaction between a secondary element and its supporting structure. That is, the motion of the secondary element may modify the motion of its supporting structure, and vice versa. In such cases, therefore, one can not predict the response of the secondary element without knowing in advance the dynamic properties of both the secondary element and the structure.

(4) When the secondary element is connected to the structure at more than one point, then each of the element's supports is excited by a motion that is different and out of phase from the motions that excite the other.

(5) Since the damping in the secondary element is much lower than the damping in the structure, the damping in the system formed by the structure and its secondary elements, which is the system that characterizes the response of the element, is not uniform. This means that the response of the secondary element is governed by the response of a system whose natural frequencies and mode shapes are complex valued. That is, the response of a system without classical modes of vibration.

(6) Since, as mentioned above, it is likely that some of the natural frequencies of a secondary element may be close in value to those of its supporting structure, the combined structure-secondary element system may result in a system with closely spaced natural frequencies. As such, the response of a secondary element may be controlled by its response in two or more of its modes of vibration, as opposed to by a single mode which is the common case for structures.

(7) The response of a secondary structural element is affected by its own yielding as well as the yielding of its supporting structure.

METHODS OF ANALYSIS: HISTORICAL PERSPECTIVE

A great deal of research effort has been devoted over the last thirty years to develop rational methods for the seismic analysis of secondary structures. For the most part, however, this effort has been fueled by the need to guarantee the survivability of critical equipment, such as piping and control systems, in nuclear power plants. Therefore, these methods have been successfully applied in the analysis of these systems, but have not been used extensively for the analysis of ordinary secondary elements in ordinary structures. Notwithstanding, many methods of analysis have been proposed as a result of this research effort, some of them with a strong empirical base and others based on rigorous principles of structural dynamics.

In the development of methods for the analysis of secondary structures, it is generally recognized that they are difficult to analyze accurately and efficiently. It is always possible to consider them in conjunction with the analysis of their supporting primary structures, but a combined primary-secondary system generally results in a system with an excessive number of degrees of freedom and a large difference in the values of its various masses, stiffnesses and damping constants. Such characteristics usually render any conventional method of analysis expensive, inaccurate, and inefficient. For example, a modal analysis presents difficulties in the computation of natural frequencies and mode shapes, a step-by-step integration method becomes extraordinarily sensitive to the selected integration time step, and a random vibration approach turns out to be particularly susceptible to the assumptions made about stationarity and earthquake duration in the probabilistic model adopted. Furthermore, such an approach is impractical since, during the preliminary design of the secondary structure, the primary structure would have to be reanalyzed every time a change is introduced in some the parameters of the secondary one. Considering that normally primary and secondary structures are designed by different teams at different times, this would bring serious problems of schedule and efficiency. Thus, most of the proposed methods have been the result of an effort to avoid the analysis of a combined primary-secondary system and overcome the aforementioned difficulties.

One of the first simplified methods used in the analysis of secondary structures is the so called systems-in-cascade or floor response spectrum method. In this method, the acceleration time history of the point or floor of the structure to which the secondary structure is attached is determined by means of a step-by step integration with a synthetic time history consistent with a given ground response spectrum. Then, this time history is used to generate a response spectrum-- that is, a floor response spectrum, which in turn is used to carry out a response spectrum analysis of the secondary system in. the same way a primary structure may be analyzed using a ground response spectrum. Although simple in concept and somewhat rational, it was quickly recognized that this method is impractical since it requires lengthy numerical integrations. For this reason, several methods were proposed to generate floor response spectra directly form a specified ground response spectrum or a design spectrum without utilizing a time-history analysis. These methods use as input a specified ground response spectrum and the dynamic properties of the structure. Examples are those proposed by Biggs and Roesset (1970), Amin *et al.* (1971), Kapur and Shao (1973), Peters *et al.* (1977), Vanmarcke (1977), Atalik (1978), and Singh (1980).

Floor response spectrum methods have been proven accurate for secondary elements whose masses are much smaller than the masses of the supporting structure and with natural frequencies that are not too close to the natural frequency of this supporting structure. However, these methods may yield overly conservative results for secondary elements that do not have these characteristics. The reason is that by considering a secondary element separately from its supporting structure, floor response spectrum methods neglect the dynamic interaction between a primary and a secondary system. That is, they do not account for the fact that the response of the secondary system may affect the response of the supporting structure and vice versa.. An additional reason is that floor response spectrum methods cannot take into consideration the fact that the masses of the primary and secondary systems vibrate out of phase as a result from the fact that, in general, a combined primary-secondary system does not possess, and cannot be assumed to do so without introducing a significant error, classical modes of vibration. There is nowadays plenty of evidence that demonstrates that ignoring these two effects may lead to gross errors in the calculation of some secondary systems' response (refer, for example, to Igusa and Der Kiureghian, 1985a, and Chen and Soong, 1988)).

Another problem with floor response spectrum methods is that they cannot be rationally applied for the analysis of secondary structures with multiple points of attachment (Wang *et al.*, 1983). This is so because these methods cannot realistically take into account the fact that each of its supports is subjected to a different and out-of-phase motion.. Attempts have been made to overcome this problem, but for the most part these attempts have been in the form of empirical or ad-hoc procedures. For example, it has been proposed to determine the maximum response of a multiply supported secondary system by calculating first its maximum response on the basis of each of the floor spectra (one at the time) that correspond to each of its supports. Then, these maximum responses are combined in an empirical way to estimate the system's true maximum response (Shaw, 1975; Thailer, 1976). Common among these empirical procedures is the selection of the largest of all the maxima, or the combination of them on the basis of the square root of the sum of their squares. Other techniques use a spectrum obtained from enveloping all the floor spectra corresponding to secondary system's supports, or that of including a "pseudo-static" component of the response, determined in terms of the difference between the peak displacements at the various attachment points. It is nevertheless recognized nowadays that these techniques are irrational and may lead to overconservative results.

In view of the limitations of the floor response spectrum methods and the impracticality of a direct analysis of a combined primary-secondary system, several alternative methods have been developed which not only take into account the aforementioned effects, but also overcome the problems of practicality associated with a direct analysis of the combined system. In general two approaches have been followed. In one of these, in recognition of the convenience and flexibility of the floor response spectrum method., and its wide use in the nuclear power industry, corrections are introduced into this method to account in an approximate manner for the interaction between the two subsystems and the out-of-phase support motions. Examples of these methods are those proposed by Lee and Penzien (1983), Igusa and Der Kiureghian (1985b), Singh and Sharma (1985), Asfura and Der Kiureghian (1986), Gupta (1986), Burdisso and Singh(1987), and Singh and Suarez (1987). In the other approach, the response of the secondary structure is obtained on the basis of an approximate modal or random vibration analysis of the combined primary-secondary system but using through a modal synthesis the dynamic properties of its separate components. This approach eliminates the main source of error inherent in the floor response spectrum method since, by considering the two subsystems together as a single unit, their interaction between the two subsystems and the different and out-phase support motions are implicitly taken into account. It is also a practical approach. By formulating the analysis in terms of the dynamic properties of independent primary and secondary systems, one also avoids the numerical difficulties of the conventional methods of analysis associated with the large difference in the values of the parameters of the primary and secondary systems. Furthermore, one avoids the solution of large eigenvalue problems, the need to generate floor spectra since the earthquake input is defined at the ground level, or the need to reanalyze the structure every time the parameters of the secondary system are changed.

Conceptually, the idea of determining the response of a secondary element in terms of an analysis of the compound system it forms with its supporting structure, but utilizing only the properties of the individual components, is a simple one. Its implementation, however, is not free of complications and difficulties. For example, if one wants to analyze such a compound system by means of the response spectrum method, one need to define first its natural frequencies, mode shapes, damping ratios, and maximum modal responses. Then, one needs to combine these modal responses using a modal combination rule. However, the system that results from combining two structures with such a drastic difference in the values of their masses, stiffnesses, and damping constants is a system without classical modes of vibration and with closely spaced natural frequencies. This means that the natural frequencies and mode shapes of the system are complex valued and that the combination of its modal responses requires, and highly depends on, an accurate rule to combine the modal responses of systems with non-classical damping and closely spaced natural frequencies. Notwithstanding such difficulties and complications, several methods that use this technique have been proposed throughout the years, methods whose basic difference is the way the dynamic properties of the components are synthesized to obtain the dynamic properties of the combined system and on the assumptions made to simplify the procedure. In chronological order, some of these methods are those suggested by Newmark (1971), Sackman and Kelly (1979), Newmark and Villaverde (1980), Der Kiureghian *et al.* (1983), Hernried and Sackman (1984), Gupta (1984), Igusa and Der Kiureghian (1985c), Villaverde (1986a, 1986b), Suarez and Singh (1987), Muscolino (1990), and Villaverde (1991).

All the methods referred to above have been derived specifically for linear secondary systems mounted on linear primary structures, without due consideration to the fact that most of the structures to which secondary systems are attached are designed to yield under the effect of a strong earthquake, and that the secondary systems themselves or their anchors are also capable of resisting large inelastic deformations. However, as pointed out by Lin and Mahin (1985), Aziz and Ghobarah (1988), Toro *et al.* (1989), Sewell *et al.* (1989), Igusa (1990), and Schroeder and Backman (1994), the nonlinear behavior of the structure and the secondary system may significantly affect the behavior of the latter, usually in the form of a significant reduction over its linear response. These methods, therefore, may lead to uneconomical and unrealistic designs. Recognizing that it may economically advantageous, a few investigators have made an effort to derive simplified methods which include such nonlinear behavior. Given, however, the difficulties in obtaining explicit solutions for such a complex problem, most of the effort has been directed towards the development of amplification factors by which a linear floor spectrum may be affected to approximately take into account the nonlinearity of a supporting structure (Kawakatsu *et al.* 1979, Lin & Mahin 1985, Viti *et al.* 1981). The exception are the works of Villaverde (1987) and Igusa (1990). Villaverde (1987) develops a method based on the use of nonlinear ground response spectra for the analysis of linear multidegree-of-freedom secondary systems mounted on elastoplastic multidegree-of-freedom primary structure. Igusa(1990) derives an analytical solution for the response of a two-degree-of-freedom primary-secondary system with small nonlinearities using random vibration theory and equivalent linearization techniques.

Current methods of analysis also include those which, albeit in a limited way, properly account for the influence of a secondary system in the response of another secondary system supported by the same structure but at a different location, and for the torsional response of the structure in the case of asymmetric structures. Observing that when a building has more than one secondary system attached to it, each secondary system may influence the response of the structure and thus, indirectly, the response of all the other secondary systems, Suarez and Singh (1989) have proposed a procedure to calculate the modal properties of a primary structure that supports two secondary systems Similarly, recognizing that the torsional response of the structure may be an important factor which may significantly increase the response of a secondary systems if this is connected to a structure with significant torsional modes, Yang and Huang (1993) have proposed a simplified method to compute the seismic response of a secondary system in such a case. Their method, however, is limited to linear primary-secondary systems with classical damping and floor eccentricities in only one direction. Finally, it is worth mentioning that just recently three code and design oriented simplified methods have been proposed by Soong *et al.* (1993), Singh *et al.* (1993) and Villaverde (1996).

For a detailed description of the difference between the methods of analysis cited above, the reader is referred to the excellent state-of-the-art reviews made by Chen and Soong (1988), Singh (1990), and Soong (1994).

DESIGN PROVISIONS IN BUILDING CODES

Overview

Several building codes and seismic provisions give recommendations for the seismic design of equipment and other secondary structural elements. In the United States, some of these are the Uniform Building Code, issued by the International Conference of Building Officials (ICBO, 1994); the NEHRP (National Earthquake Hazard Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, issued by the Building Seismic Safety Council under contract with the Federal Emergency Management Agency (BSSC, 1995); the

Recommended Lateral Force Requirements and Commentary, issued by the Structural Engineers Association of California (SEAOC, 1990); and the ASME (American Society of Mechanical Engineers) Boiler and Pressure Vessel Code (ASME, 1993). Discussed here will be the provisions in the UBC and NEHRP codes. The Uniform Building Code is selected for the discussion because this code is well know worldwide and the NEHRP provisions because they have recently incorporated a set of modern and comprehensive recommendations that are more rational than those in most other codes. The recommendations given by SEAOC will not be discussed because the seismic provisions in this document and those in the UBC are essentially the same. This result from the fact that SEOAC is the organization that develops these provisions and that traditionally UBC simply adopts them with only minor modifications. Similarly, the ASME recommendations will not be covered because these recommendations are only pertinent for the seismic design of equipment. It is recognized that there are some other codes around the world who also address the design of secondary structural elements, but for the most part the recommendations in these other codes are very similar to those herein being considered.

Uniform Building Code

The Uniform Building Code requires that elements of structures (i.e., infill walls, penthouses, diaphragms), permanent nonstructural components, and the attachments (i.e., connections and anchorages) for permanent equipment supported by a structure be designed to resist at least the lateral seismic force calculated with the following equation:

$$F_p = Z\, I_p C_p W_p$$

In this equation, Z is a zone factor, which essentially represents the peak ground acceleration expected at the site under consideration in an average recurrence interval of 475 years; I_p is an equipment importance factor which is set equal to 1.0 for ordinary components and to 1.5 for critical ones; and W_p is the total weight of the component. C_p is a coefficient specified by the code, which varies, depending on the type of component or equipment, from 0.75 to 2.0, and is intended to account for the dynamic amplification of the ground motion by the building for items located above grade. The equation is supposed to be used in conjunction with working stress design principles.

The values of C_p that are explicitly given by the code are supposed to be for rigid elements and components and rigid or rigidly supported equipment. For that purpose, the code defines a rigid component or element and a rigid or rigidly supported equipment as those having a fundamental period less than or equal to 0.6 seconds. In the absence of a dynamic analysis or empirical data, the value of C_p for nonrigid components or flexibly supported equipment located above grade is supposed to be taken as twice the specified values, but need not exceed 2.0. Another exception is for ductile piping, ducting, and conduit systems for which the listed values may be used.

The code also requires that in the design of equipment, structural elements, and nonstructural elements the relative motion between the points of their attachment to the structure be considered. It does not specifies, however, how this effect should be accounted for.

As it can be seen, the approach used by the Uniform Building Code for the seismic design of secondary structural elements is largely empirical and judgmental, and rational to a lesser extent. The specified values of C_p are set primarily by (a) examining the performance of nonstructural components in past earthquakes; (b) the results of some amplification analyses with linear multistory buildings, which show that the ground acceleration amplification with height usually falls in the range between 1.6 and 2.3; and (c) the consideration of inherent inelastic behavior as a reserve capacity (Porush, 1990). It does not involve the dynamic interaction between the component and its supporting structure, its location within the building, the way it is connected to the building, its tuning or detuning to the natural frequencies of the structure, the differential motion between its supports, and the yielding of the structure. Thus, the emphasis is in life safety and not in damage prevention. Functionality is only indirectly addressed by the use of an importance factor. Emphasis is also in simplicity so that the recommendations may be interpreted correctly by the average engineer and understood and easily enforced by building officials, who in most cases are not specialist in the subject. It does not incorporate the current level of understanding about their seismic behavior accumulated during the last twenty years.

1994 NEHRP Provisions

The NEHRP provisions are based on ultimate strength design principles and establish, as the UBC does, minimum design criteria for architectural, mechanical, and electrical systems, nonstructural components and elements permanently attached to buildings, and their supporting systems and attachments. These design criteria are in terms of a required minimum equivalent static force and a minimum relative displacement demand when the component is connected to the structure at multiple points.

To determine the required minimum static force, the provisions provide two alternate equations. One of these is the default equation, which is conservative, but simple and easy to apply. This equation is

$$F_p = 4.0\, C_a I_p W_p$$

The other is more complex, as it takes more factors into account, but it generally leads to smaller forces. It is given by

$$F_p = a_p A_p I_p W_p / R_p > 0.5\, C_a I_p W_p$$

where

$$A_p = C_a + (A_r - C_a)(x/h)$$

in which $A_r = 2.0\, A_s \leq 4.0\, C_a$.

In the equations above, F_p = seismic design force applied at the component's center of gravity vertically, laterally, or longitudinally in combination with the dead and live loads acting on the component; a_p = component amplification factor specified in the provisions according to component type (varies between 1.0 and 2.5); A_p = acceleration expressed as a fraction of gravity at point of attachment to the structure; I_p = component importance factor specified in the provisions according to component type (equal to either 1.0 or 1.5); W_p = component operating weight; R_p = component response modification factor specified according to component type (varies between 1.5 and 6.0); C_a = seismic coefficient specified as a fraction of gravity for structural design (i.e., effective peak ground acceleration); A_r = acceleration, expressed as a fraction of gravity, at structure roof level; A_s = structural response acceleration coefficient (i.e., ground response spectrum ordinate), expressed as a fraction of gravity, given by

$$A_s = 1.2\, C_v / T^{2/3} \leq 2.5\, C_a$$

in which C_v = velocity-related effective ground acceleration specified as a fraction for structural design, and T= effective fundamental period of the structure.

To determine the required minimum relative displacement between the two connections points of a nonstructural component with multiple connection points, such as cladding, stairwells, windows, ducts and piping systems, the provisions recommend to use the smaller of the values obtained with the following two equations:

$$D_p = \delta_{xA} - \delta_{yA}$$

$$D_p = (X-Y)\, \Delta_{aA} / h_{sx}$$

For nonstructural components with connection points on separate structures or buildings, the corresponding two equation are:

$$D_p = |\delta_{xA}| + |\delta_{yB}|$$

$$D_p = X\,\Delta_{aA}/h_{sx} + Y\,\Delta_{aB}/h_{sx}$$

In the equations above, D_p = relative seismic displacement between supports which a component should be able to accommodate; $\delta_{xA}, \delta_{yA}, \delta_{xB}, \delta_{yB}$ = deflection of building under design forces, multiplied by an amplification factor to account for inelastic deformations, of building level x, y of building A, B; X ,Y = height above grade of component support at level x, y; Δ_{aA}, Δ_{aB} = allowable story drift for Building A, B; and h_{sx} = story height. The equations in terms of the allowable interstory drifts are provided in recognition that the building displacements may not be available at the time the component is designed. A component is supposed to be designed to be able to resist the displacements obtained with these equations when they are combined with the effects of other displacements such as those generated by thermal and static loads.

The equations presented above take into account the amplification of a ground motion at the points of a structure that are above grade, the location of the component within the structure, the amplification of floor motion by the dynamic characteristics of the component, the ductility and energy absorption capabilities of the component, and the performance expectations of the component. As such, they incorporate many of the factors that may influence the seismic behavior of nonstructural components on buildings and are, thus, more rational than, and a significant improvement over, their counterparts in other building codes, the Uniform Building Code included. Notwithstanding, the NEHRP provisions still have some limitations. For example, the given equations are in terms of two separate and independent amplification factors. One is to account for the ground motion amplification by the structure at the level of the point where the component is attached to the structure, and the other the amplification by the component of the motion at this attachment point motion. Consequently, the equations do not fully consider the interaction between the structure and the component. Similarly, the provisions give two separate sets of equations, one to compute the maximum forces and the other to compute the maximum relative displacements between the component's attachment points. The effect of these forces and that of these relative displacements on the structure are supposed to be added directly. This implies that these two maximum effects occur at the same time, which is an assumption that in most cases leads to overly conservative results. Another limitation is the recommended amplification factors. A factor of 2.0 is proposed for the structural amplification and a maximum of 2.5 for the component amplification. These are ad-hoc factors which are justified on the basis of limited experimental results and observations from past earthquakes (Soong *et al*, 1993), but lack a theoretical base. Furthermore, combined these factors give a maximum amplification factor of 5.0, which when compared with those that are theoretically possible (Villaverde, 1996) may not be large enough to cover all cases. This may be particularly true when it is considered that the provisions explicitly account for component yielding to reduce the magnitude of a component's design forces, and thus one cannot count on the inelastic behavior of the component to resist earthquakes forces that exceed these design forces. Finally, the recommended equations do not account for the yielding of the structure. Although it is recognized that in many cases the design of a structure is governed by drift limits or other loads, and that it is difficult to define for the purpose of nonstructural component design the magnitude of the forces that in actuality make the structure yield, it is also recognized that structural yielding may significantly reduce the seismic forces on a component (Toro *et al.*, 1989), and that localized yielding will always occur whenever the structure is subjected to an earthquake of a size that is comparable to the size of the earthquakes for which it was designed. Structural yielding is, therefore, an important parameter that should be considered explicitly in the determination of the design seismic forces for nonstructural components.

EXPERIMENTAL STUDIES AND FIELD OBSERVATIONS

In contrast to the vast analytical work, experimental tests and field observations of secondary structural elements seem to be scarce. Many experimental tests have been conducted and reported to qualify equipment and other nonstructural elements, but only a few have been performed to either further investigate their seismic behavior when mounted on a structure, or to verify the findings from analytical imvestigations. In general, the experimental work that has been reported in the literature consists of either the testing of secondary elements mounted on a primary structure to analyze the behavior of the secondary elements, or the testing of the secondary elements by themselves with the purpose of investigating their dynamic properties and load capacity.

Within the first category are the tests carried out by Kelly and Tsai, Japan's Building Research Institute, Nims and Kelly, Juhn *et al.*, and Japan's Nuclear Power Engineering Center. In the test carried out by Kelly and Tsai (1985), they investigate, among several other things, the response of light equipment in structures isolated using rubber bearings and compare it against the equipment's response in a fixed base system. For that purpose, three oscillators representing pieces of light equipment were attached to the fifth floor of a 1/3-scale, 5-story frame mounted on four rubber, or lead-rubber, isolators. The total weight of the structure and the added weights was 80 kips. Three isolators were used. Their weights were 80, 40, and 20 lb., and were tuned, respectively, to the fundamental natural frequency of the fixed frame, the second natural frequency of the base-isolated frame, and the third natural frequency of the base isolated frame. Four different earthquake records were used to excite the base of the frame. In the experiment performed by the Building Research Institute in Japan (conducted under the auspices of the U.S.-Japan Cooperative Research Program and described in detail by Wang, 1987), a full-scale three-dimensional frame with a full-scale cladding system was tested to observe the behavior of cladding systems and their connections. The frame had six stories, a total height of 22.38 m., and a 15 m x 15 m plan with two equal bays in each direction. The cladding system consisted of precast concrete and glass fiber reinforced concrete panels with a variety of sway type connections. The tests were carried out under static loading, free vibrations, and force vibrations. In the test under static load, the frame was deformed up to story drifts of 1/40. In the test conducted by Nims and Kelly (1990), a piping system was mounted on two independent full-scale steel frames and tested, together with the frames, on a shaking table. One of the frames had a single bay and three stories, with a total height of 17 ft. and 4 in. and a plane area of 12 x 6 ft. The other had three bays and four stories, with a total height of 14 ft. and a plane area of 18 x 6 ft. The piping system, configured to represent a typical one in nuclear power plants, was approximately 100 ft. long. It was made of pipes 2 and 3 in. in diameter, and mounted on seven rigid supports and five restraining devices. The test was performed to evaluate the performance of three types of restraining devices (snubbers, seismic stops, and energy dissipating restrains), but also served to study the interaction between the piping system and the frames. The test were conducted under synthetic and recorded earthquake ground motions. The test conducted by Juhn *et al.* (1990) was a shaking table test involving a secondary system in the form of an inverted pendulum attached to the second story of a quarter scale, three-story fixed-base steel frame. The secondary system had a mass ratio equal to 1/10 the mass of its supporting floor. Both tuned and detuned cases were considered. The excitations considered were time histories representing a white noise and a scaled version of an earthquake ground motion. The tests were conducted to obtain experimental evidence on the behavior of secondary systems mounted on a seismically excited structure and to verify the results of a proposed analytical method to generate floor response spectra. Finally, the tests carried out by Japan's Nuclear Power Engineering Center were a series of shaking table tests of full, or close to full, scale models of critical equipment in nuclear power plants. These tests, conducted at the 1000-ton shaking table at Tadotsu Engineering Laboratory in Shikoku Island, are part of a program started in 1982 to confirm the integrity of critical equipment in nuclear power plants and to validate the methods used for their seismic design. They are reported in a series of papers presented at the Tenth World Conference on Earthquake Engineering (see, for example, Ohtani *et al.*, 1992).

In the second category are the works of Craig and Goodno (1981), Rihal (1988), Chiba *et al.* (1992), Rihal (1994), and Pantelides and Behr(1994) and Behr *et al.* (1995). Craig and Goodno (1981) measured in the laboratory the natural frequencies, mode shapes, and damping ratios of the window in a full scale glass cladding panel. Their specimen consisted of a single story section of a cladding system and included the mullions, munitions, spandrel framing, glazing materials, and four double-pane vision lights 99 in. x 57 in. x 1 in. Rihal (1988) conducted cyclic in-plane racking tests of a precast concrete cladding panel with bearing connections at the bottom and threaded-rod lateral connections at the top. The objective of the test was to obtain quantitative data on the in-plane resistance and deformation capability of precast cladding panels. The tested specimen consisted of a solid precast concrete cladding panel 8 ft. wide, 10 ft. high , and 4-1/2 in. thick, with two threaded-rod lateral connections at the top of the panel and two bearing connections at the bottom. The bearing connections consisted of a steel angle assembly with four studs 5/8 in. in diameter welded to the back of the angle, and embedded in the cladding pane. The specimen was tested under cyclic displacements applied to the threaded rods of the lateral connections. Chiba *et al.* (1992) tested on a shaking table a three-dimensional piping system mounted on a combination of rigid restrains and elastoplastic dampers. The purpose of test was to investigate the dynamic behavior of a cracked pipe supported on elastoplastic dampers, and to clarify the effect of the stiffness of piping support on the crack growth. The piping system tested was 27.4 m in length and 165.2 mm in diameter. The elastoplastic dampers were made of three layered steel plates. Rihal (1994) conducted experimental in-plane and out-of-plane cyclic tests of loaded library shelving units, representative of those designed under current standards. The objective of the tests was to assess the adequacy of the provisions in current codes and standards governing the design and installation of cantilever library shelving systems in seismic zones. In the in-plane test, the shelving specimen consisted of three units 3-ft. wide and 7 ft.-6-in. high., each. In the out-of-plane test, the specimen consisted of only of two such units. Pantelides and Behr (1994) and Behr at al. (1995) conducted experimental tests to investigate the general glass breakage and glass fallout behavior during earthquakes of various types of architectural glass elements in dry-glazed curtain walls. A 4.56 m x 3.68 m section of a dry glazed curtain wall, containing three 1.52 x 1.84 m glass panels and a wide mullion, was employed in the tests. The type of glass tested included annealed, heat-strengthened, fully tempered glass in monolithic and laminated configurations and with different thicknesses. In the tests, the specimen was subjected to in-plane and out-of-plane dynamic motions.

If experimental investigations are scarce, field observations from instrumented secondary structural elements are even more so. To the writer's knowledge, the only report of field instrumentation of equipment is that from Hiramatsu *et al.* (1988). In this report, Hiramatsu *et al.* describe a field observation system established in Japan in 1982 to monitor the seismic response of telecommunication equipment in a five-story telephone office building. In this system, accelerometers have been installed on the equipment in several floors of the building at 16 points, on a roof steel tower at 2 points, and on an antenna above the tower at one point. So far the system has recorded several earthquakes of moderate magnitude. Worthwhile to mention too in regard to field observations is the study conducted by Rihal (1992) to investigate the relationship between peak floor acceleration, frequency content, and inter-story drift and the nonstructural damage observed during an earthquake. In his study, Rihal uses the data recorded during the 1989 Loma Prieta earthquake in an instrumented building and the corresponding observed nonstructural damage.

RESEARCH NEEDS

As seen from the above review, much progress has been done towards the understanding of the seismic behavior of secondary structural elements, the development of simplified methods of analysis, and damage mitigation. Notwithstanding, it is also clear from this review and the damage to these elements observed during recent earthquake that the problem is a complex one and as such has not been completely solved. Thus, there are several aspects about secondary structural elements which demand further research.

An area of research which is urgently needed to advance the understanding of secondary structural elements and to derive simplified methods for their analysis is that related to the effect of the nonlinearity of their supporting structures and that of the secondary elements themselves on the response of the latter. All current evidence seems to indicate that the seismic response of a secondary element is significantly lower when the yielding of its supporting structure and the yielding of the element itself are taken into account than when it is assumed that both subsystems remain linear at all times. However, only a limited number of studies have been conducted to investigate this problem, and only a few simplified methods of analysis which take into account such nonlinearity have been proposed. Given that most primary structures are designed to yield under a strong earthquake, and given that most secondary elements or their anchorages may be capable of resisting some level of inelastic deformation, the inclusion of such nonlinear effects into the design of secondary structural elements could bring important economical benefits.

Research is also needed to study the influence of the torsional response of a supporting structure in the response of a secondary element attached to it. As mentioned earlier, this torsional response may significantly increase the response of a secondary system if this is connected to an asymmetric structure. Hence, if ignored, it may lead to unconservative designs. This problem has received little attention in the past, so presently there is not enough information available to assess how important this factor is and how to take it into account in the design of a secondary system. Work is thus needed to assess this importance and to develop simplified methods of analysis that take structural torsion into account. It is important to keep in mind, however, that future work in this area should consider such torsional response in conjunction with the nonlinear behavior of the structure. It is well know that under current practice structures resist large torsional motions by incurring into their nonlinear range of behavior. Consequently, a study which includes the torsional response of the structure but is based on the assumption of linear structural behavior would have not much meaning.

Another area of research that deserves full consideration from the research community is that related to the base isolation and structural control of secondary structures. Given their relatively small size and the high accelerations to which they can be subjected, secondary structures are ideal for the application of these techniques and important benefits may be realized from their use. Particular topics of interest in this regard are the influence of structural yielding in the effectiveness of these techniques when they are applied to secondary structures, and the development of simplified methods for the analysis of secondary structures which incorporate in their design any of these techniques.

Despite the high level of understanding that has been gained about the behavior of secondary structural elements mounted on a primary structure, and despite the numerous rational procedures that have been proposed over the last few years for the analysis of these secondary structural elements, the standards and specifications in current building codes for their design still do not reflect this level of understanding and have not yet incorporated any of these rational procedures. Undoubtedly, this has been so because these rational methods of analysis are too complicated or too cumbersome for the design of ordinary secondary elements in ordinary structures. Thus, as pointed out by Chen and Soong (1988), a great challenge for researchers is the development of methods of analysis that, on one hand, are rational and accurate, but on the other, are simple enough for their incorporation into building codes. As noted in the section on methods of analysis above, some progress has been made in this regard in the last couple of years, but still further work is needed. In particular, research is needed to develop simplified guidelines to account in a rational way for the effect of yielding in a structure on the response of the secondary elements attached to it.

Finally, an extensive program of experimental work and field instrumentation is needed to complement the ongoing analytical studies. Laboratory tests, mainly in the form of shaking table tests, are needed to verify the findings from the analytical studies; to define the stiffness, damping, ductility, and drift limits of specific secondary structural elements and their anchorages; to define the acceleration limits at which particular pieces of equipment cease to be operational; to test the suitability of current and new bracing methods and anchoring systems, and to test the effectiveness of base isolation and structural control schemes. The instrumentation of actual secondary systems on actual structures is needed to collect data about their performance during real earthquakes under actual conditions, and to contrast this performance against the results from analytical and experimental studies.

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