Paper No. 643. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

A PROPOSED REPLACEMENT FOR THE SEISMIC CODE PROVISIONS FOR NONSTRUCTURAL COMPONENTS IN BUILDINGS

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ABSTRACT

The purpose of this paper is to propose a procedure to determine in a rational but simple way the lateral forces for the seismic design of nonstructural components attached to buildings. The procedure is derived using concepts from a previously developed dynamic theory for secondary systems mounted on a primary structure, and by introducing into this theory some simplifying assumptions. It takes into account the dynamic interaction between the structure and the nonstructural component, the level above the base of the structure of the point or points where the nonstructural component is attached to the structure, and the number of such attachment points. It uses, in addition, the design spectra specified by building codes for the design of the structure as the earthquake input to the nonstructural component. In a numerical study with various nonstructural components attached to a multistory frame building, a comparison is made between the shear force capacities of the elements of the nonstructural components when these capacities are determined with the proposed procedure, and the shear force demands on these same elements -- obtained through a time-history analysis of the building-component system -- when the base of the building is subjected to a critical ground motion. On the basis of this comparative study, it is demonstrated that the procedure is accurate enough for all practical purposes, more rational than the pertinent recommendations in current code provisions, and a suitable replacement for these recommendations.

KEYWORDS

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

INTRODUCTION

It has been widely recognized during the last two and a half decades that equipment, furniture, architectural elements, and other nonstructural components attached to the floors and walls of buildings are highly vulnerable to the effects of earthquakes. This vulnerability is owed to the fact that these nonstructural components are subjected to large ground motion amplifications when their natural frequencies are close to the natural frequencies of the structure that supports them, and to the fact that, because of their relatively small masses and stiffnesses, this closeness between natural frequencies may be possible in many cases. Notwithstanding, the seismic design of nonstructural components in buildings has not been given the attention it deserves, building codes continue to recommend irrational and oversimplified specifications for their seismic design, and failures of penthouses, signboards, water tanks, pipelines, bookcases, and storage racks are still a common occurrence earthquake after earthquake.

Much research work has been carried out during the last two decades on the dynamic behavior of secondary systems connected to a primary structure, and a vast amount of knowledge and understanding has been generated in this area (see state of the art reviews in Cheng and Soong, 1989, and Soong, 1994). This progress, however, has been generated as a result of an effort from the engineering profession to assure the survivability of critical installations, such as piping systems and control panels, in nuclear power plants. As such, rational methods have been developed for the design of these components, but the majority of these methods are still too complicated and too sophisticated for their application in the design of ordinary nonstructural components in ordinary buildings and for their incorporation into building codes. A notable exception is the recent work from Soong *et al.*(1993) and Singh *et al.* (1993), who in an effort to fill this gap have proposed design oriented simplified methods. The recommendations in the 1994 version of the NEHRP provisions for the seismic design of nonstructural components (BSSC, 1995) are based on this work by Soong *et al.*

The purpose of this paper is to introduce an uncomplicated procedure to determine equivalent static lateral forces for the seismic design of nonstructural components attached to buildings. The procedure is derived on the basis of a previously developed response spectrum method that uses the dynamic theory of secondary systems mounted on a primary structure, and by simplifying this response spectrum method using assumptions that are typical of those made in the development of building code recommendations. It takes into account the dynamic interaction between the structure and the nonstructural component, the level above the base of the structure of the point or points where the nonstructural component is connected to the structure, and the number of such connection points. The earthquake input is defined in terms of the design spectra specified by building codes for the design of the structure. Presented also is a numerical example that illustrates the application of the procedure and the results of a comparative study with three nonstructural components attached to a multi-story frame building, carried out to verify the validity of the procedure.

RESPONSE SPECTRUM METHOD

The response spectrum method on which the procedure herein being proposed is based is described in detail in a recent paper (Villaverde, 1991a). This method is derived on the basis of the application of the response spectrum technique to the combined system that a light secondary element forms with the structure to which it is attached. In it, however, under the assumption that the masses, stiffnesses, and damping constants of the secondary element are much smaller than those of the structure, the modal analysis is made in terms of the independent dynamic properties of the two subsystems. The purpose is to, first of all, avoid the computational difficulties associated with the calculation of the natural frequencies and mode shapes of a system whose masses, stiffness, and damping coefficients are of different orders of magnitude; and secondly, to be able to analyze the response of the secondary element in terms of the independent dynamic properties of the structure, which is information that is normally readily available from the structure's design team. This is achieved by deriving the dynamic properties of the combined primary-secondary system through a modal synthesis. Likewise, the modal analysis is carried out in terms of the complex mode shapes and natural frequencies of such a combined system since the damping matrix of a system with damping constants of different orders of magnitude cannot be assumed, without a significant error, of the Rayleigh type. Once the modal properties of the combined system are obtained, the seismic response of the secondary element is determined by: (1) deriving simplified equations to determine in terms of ordinates from a ground response spectrum the maximum response of the combined system, (2) using these equations to write explicit relationships for the maximum response of the secondary element, and (3) further simplifying and integrating all these relationships into a series of a few simple expressions. In the application of the method to a nonstructural component with N_s degrees of freedom attached to a supporting structure with N_p degrees of freedom, it is considered that the combined system is a system with $N_p + N_s$ degrees of freedom whose natural frequencies are those of its independent components. These combined system modes are considered as resonant if the associated natural frequency is common to both independent components, and as a nonresonant otherwise. Then, the maximum response of the nonstructural component in each of such modes is calculated using a set of given expressions, and combined in a proper way to estimate the nonstructural component's maximum response. As with any other application of the response spectrum technique, only a few of such modes need to be considered to attain a good estimate of the nonstructural component's maximum response.

The expressions presented in the aforementioned paper and discussed above are derived specifically for linear systems and considering linear response spectra. As such, they cannot be used directly to estimate the maximum forces on a nonstructural component under an extreme seismic event since by design, as is well known, the structure is supposed to incur into its nonlinear range of behavior in such a case. Since the proposed procedure is intended for design purposes, and since the design of a nonstructural component should necessarily involve such an extreme case, these expressions need therefore to be modified to take into account the effect of the nonlinear behavior of the structure on the response of the nonstructural component. In the spirit of keeping the simplicity of the desired formulation even at the cost of a reduced accuracy, this effect may be accounted for in the same way it is considered in the design of the structure; that is, by reducing the ordinates in the linear response spectrum by a factor that depends on the expected ductility of the structure. It is important to keep in mind, however, that this reduction factor cannot be the same one that is ordinarily used for the design of the structure. The reason is that, as pointed by Toro *et al.* (1989), the yield level of the structure has a pronounced effect in the response of the secondary system-- usually in terms of a significant reduction, and that the actual yield level of the structure is in most cases markedly higher than the nominal value considered in its design. To err on the safe side, it is thus wise to consider reduction factors that are less than those used for the design of the structure.

To incorporate the reduction in the response of the nonstructural component induced by the nonlinear behavior of the structure in the proposed procedure, the seismic forces determined on the basis of the response spectrum method discussed above will be here reduced by a factor R_p, which may be considered equal to a fraction of the R_w reduction factor specified for the design of the structure. Notice, however, that R_p should never be considered equal to R_w since, in addition to the aforementioned difference between the actual and the nominal lateral forces that make the structure yield, the factor R_w reduces the seismic forces on the structure to allowable stress levels, not to ultimate strength or yield levels. Pending a detailed study in this regard, it is recommended to consider for the time being R_p equal to $R_w/2$.

LATERAL FORCE PROCEDURE IN BUILDING CODE FORMAT

A simplified expression to determine the design lateral seismic forces at the masses of a nonstructural component attached to a building structure has been derived by introducing into the formulas to determine a secondary system's response in the resonant modes of its combined primary-secondary system, given in conjunction with the response spectrum method summarized above, approximations of the kind that are common in current seismic code provisions, and by rewriting the resulting equations into the format used in the 1994 version of the Uniform Building Code (ICBO, 1994). The expression obtained is of the form:

$$F_{pj} = \frac{w_{pj}\, l_j}{\sum\limits_{=1}^{N_s} w_{pj}\, l_j} V_p \tag{1}$$

where F_{pj} denotes the lateral force generated by the design earthquake at the center of mass of the nonstructural component's jth mass; w_{pj} is the weight of this jth mass; and l_j is the distance to this mass measured in the case of a component with a single attachment point from this attachment point (see Fig. 1a). In the case of a component with two attachment points, l_j is measured from its lower end if the mass is located below the point at which the component attains its maximum deflection when each mass is subjected to a lateral force equal to its own weight, and from its upper end otherwise (see Fig. 1b). In the case of a mass located directly at such point of maximum deflection, measure l_j from the support that is the farthest away from that mass. N_s represents the number of masses in the nonstructural component and V_p the component's base shear or the sum of the shears at its supports. V_p is given by

$$V_p = \frac{ZIC}{R_p} I_p\, C_p\, w_p \tag{2}$$

in which Z = peak ground acceleration corresponding to given seismic zone; I = structure importance factor; I_p = component importance factor (herein assumed equal to 1.0); C = ordinate in response spectrum specified for design of structure, normalized to a peak ground acceleration of 1.0 g; R_p = component response reduction factor (herein assumed equal to $R_w/2$, where R_w is the response modification factor specified for the structure); w_p = component total weight; and C_p = component amplification factor, given by

$$C_p = \frac{1}{2} \frac{\Phi_o \sqrt{W}}{\sqrt{|\Phi_o^2 w_p - 0.0025W|}} \tag{3}$$

where

$$\Phi_o = \frac{W h_{av}}{\sum_{i=1}^{N_p} W_i h_i} \tag{4}$$

in which W_i and h_i are the weight and elevation above ground of the ith building floor, respectively, W is the total weight of the building, h_{av} is the average of the elevations above ground of the points of the structure to which the component is attached, and N_p denotes the number of floors in the building. Note that these equations are only valid when the nonstructural component is attached to the structure at one or two points. However, a component with more than two attachment points may still be analyzed with these equations by simply dividing it into a series of subcomponents with one or two attachment points and by considering each of these subcomponents separately.

It is also important to note that the simplified procedure presented above is based on the assumption that the independent fundamental natural frequencies of the structure and the nonresonant component are in resonance; that is, their values are equal or are very close to one another. Although this assumption offers the advantage of not having to know the natural frequencies of the nonstructural component, it may be nonetheless overly conservative for those cases in which such natural frequencies are far apart. To provide, thus, a means to assess more accurately the response of those nonstructural components for which their natural frequencies are known but without sacrificing the simplicity of the procedure, it may be considered, as suggested by Soong *et al.* (1993) and shown in Fig. 2, that the component response amplification factor varies linearly with the natural period ratio T_p/T (T_p being the fundamental period of the component and T that of the structure) between the value calculated with the above formulas when this ratio is close to 1.0 and the value which represents no amplification when T_p/T is sufficiently different from 1.0.

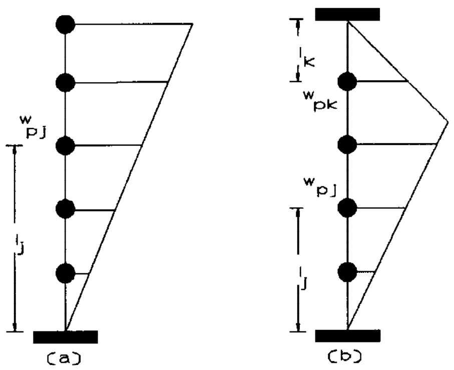

Fig. 1. Assumed mode shapes for nonstructural components

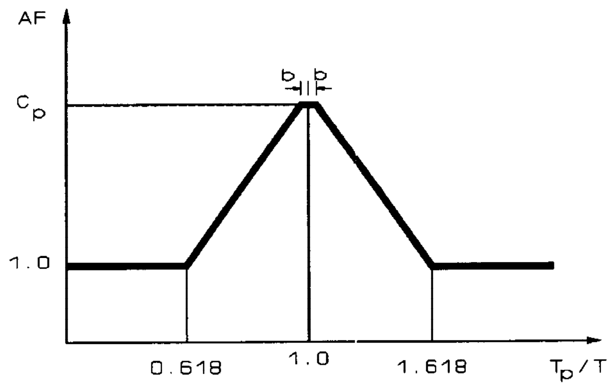

Fig. 2. Variation of component amplification factor (AF) with natural period ratio

Using, thus, the formulas from the response spectrum method described above to compute a component's response amplification factors in the case of nonresonant modes, it is found that the limits for the natural period ratio beyond which it may be considered that a nonstructural component undergoes no motion amplification are those indicated in Fig. 2. Similarly, it is found that the values of this ratio for which the fundamental natural periods of a nonstructural component and its supporting structure should be considered in resonance with one another, and for which the maximum value of C_p should be used (see Fig. 2), are those contained in the interval [1-b, 1+b], where

$$b = \frac{1}{2} \Phi_o \sqrt{w_p/W} \tag{5}$$

Consequently, when the fundamental natural periods of the component and its supporting structure are known and their values are not too close to one another, the diagram in Fig. 2 may be used to reduce in terms of the relationship between the two periods, the value of the amplification factor C_p obtained with Eq. 3 and, as a result, the component's design lateral forces.

ILLUSTRATIVE EXAMPLE

To clarify the proposed procedure, consider the 6-story office building and the three-mass nonstructural component shown schematically in Fig. 3. The building is located over a deposit of firm soil in the city of Irvine, California, and structured with steel moment resisting frames. The building's weight per floor is 2,200 kN and its total weight is thus equal to 13,200 kN. The nonstructural component has four equal segments with a length of 1.65 m each and is rigidly attached to the 4th and 6th floors of the building. Each of its masses weighs 4.4 kN, and thus its total weight is 13.2 kN; that is, 0.1 per cent of the total weight of the building. It is assumed at this stage of the design that the natural frequencies of the nonstructural component are unknown.

To determine the design lateral forces acting on the masses of the nonstructural component, consider first Eqs. 4 and 3, whereby one can compute, respectively, the average displacement amplitude of the component's attachment points and the component amplification factor. Accordingly, since in this case the average of the elevations above ground of the two attachment points equals 16.5 m, substitution into Eq. 4 of this value and the floor weights given above leads to a value for Φ_o of 1.43 m. Similarly, by substitution of this value of Φ_o and the specified component weights into Eq. 3, a value of 33.5 is obtained for C_p. To determine V_p, the sum of the shear forces at the component's supports, one can now follow the recommendations of the 1994 version of the Uniform Building Code to define Z, I, and C in Eq. 2 for the building and location under consideration. Accordingly, these values are $Z = 0.4$, $I = 1.0$, and, corresponding to a natural period of 0.8 sec. calculated with the code recommended formulas and a soil factor of 1.2, $C = 1.74$. Likewise, to define the component reduction factor R_p, one can consider the R_w factor of 12 recommended by the code for moment resisting steel frames, divided by two, to obtain $R_p = 6$. As a result, Eq. 2 yields $V_p = 51.3$ kN.

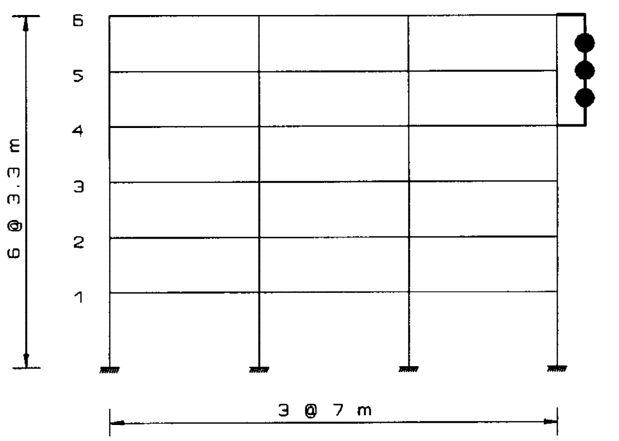

Fig. 3. Building and nonstructural component in illustrative example

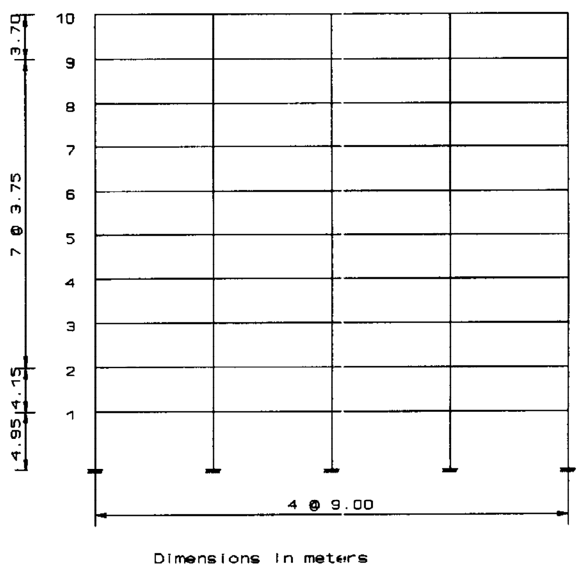

Fig. 4. 10-story building in comparative study

To distribute now this shear force of 51.3 kN among the three masses of the component, one needs to define first the component's point of maximum deflection and the distances l_j from its supports to each of its masses. Note, however, that since in this case the component is symmetric in mass and geometry, the point of maximum deflection is at its geometric center and thus $l_1 = l_3 = 1.65$ m and $l_2 = 3.3$ m, where l_1, l_2, and l_3 correspond, respectively, to the lower, middle, and upper masses. As a result, Eq. 1 gives the lateral forces $F_{p1} = 12.8$ kN, $F_{p2} = 25.7$ kN, and $F_{p3} = 12.8$ kN.

COMPARATIVE STUDY

To assess whether or not nonstructural components designed with the proposed procedure would survive a critical earthquake ground motion, a comparative analysis is performed with three different nonstructural components mounted on a multistory building. In this comparative analysis, the maximum shear forces in the elements of the nonstructural components are first calculated on the basis of the equivalent lateral forces obtained with the proposed procedure. Then, it is assumed that such elements are designed to resist these shear forces; that is, it is considered that these represent the shear force capacities of the elements. Thereafter, the shear force demands imposed on the elements when the aforementioned critical ground motion excites the base of the building are obtained by means of a time history analysis. In the final part of the analysis, the shear force demands are compared against the shear force capacities to investigate if in all cases the shear force demands are equal or less than the corresponding capacities.

The building considered is shown in Fig. 4. This building represents an actual reinforced concrete office building in Mexico City, located in the soft soil area of the city. The building, studied in detail by Villaverde (1991b), experienced significant damage during the September 19, 1985 earthquake, indicating thus that the structure incurred into its nonlinear range of behavior during this earthquake. The materials considered in its design were concrete with a nominal 28-day strength of 24 MPa and reinforcing steel with a nominal yield strength of 400 MPa. The dead load and live load for seismic design per floor for this building is 2031 kN for floors 1 to 9, and 1591 kN for the roof. The total building's weight is thus equal to 19,870 kN. The properties of its beams and columns are listed in Table 1. On the basis of the effective moments of inertia in this table and center-to-center lengths, its first three natural frequencies along its longitudinal direction are 0.542, 1.439, and 2.421 Hz. The horizontal displacement amplitudes for each of its floors in its first mode shape, after this mode shape is normalized to obtain a unit participation factor, are 0.141, 0.287, 0.424, 0.561, 0.723, 0.868, 1.026, 1.163, 1.300, and 1.381. The damping matrix of the building is assumed proportional to its stiffness matrix, with a damping ratio of 5 per cent of critical in its fundamental mode. The seismic design of the building (ultimate strength design) was carried out considering a response reduction factor of 4.

Table 1. Properties of beams and columns in building in comparative study

Columns					Beams				
Story	b (m)	h (m)	I_{eff} (m^4)	M_u (kN-m)	Floor	b (m)	h (m)	I_{eff} (m^4)	M_u (kN-m)
1-2	0.5	0.9	0.01309	497.0	1-3	0.4	0.8	0.03151	780.9
3-4	0.5	0.8	0.00953	397.0	4-7	0.4	0.7	0.02549	627.6
5-6	0.5	0.7	0.00669	255.0	8-9	0.4	0.6	0.01742	463.4
7-8	0.5	0.6	0.00447	217.0	10	0.4	0.5	0.01330	210.9
9-10	0.5	0.5	0.00279	180.0					

Note: b=width; h=height; I_{eff} = effective moment of inertia; M_u= ultimate moment

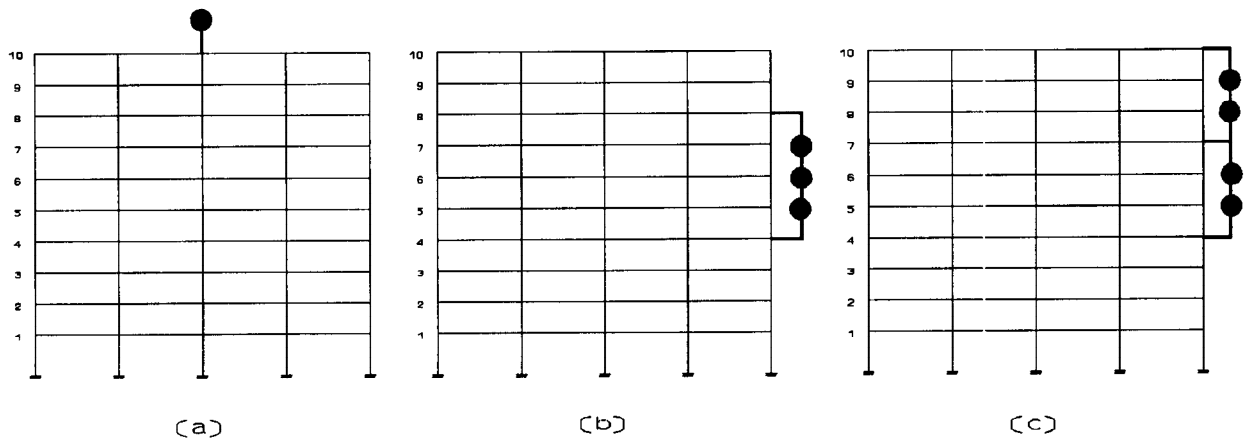

Fig. 5. Nonstructural components attached to building in comparative study

The three nonstructural components studied are: (a) a single-mass system with a single point of attachment; (b) a three-mass system with two points of attachment; and (c) four-mass system with three points of attachment. They are modeled as linear shear beams, rigidly attached to their supports, with the masses and stiffness in Table 2, and a length between masses of 3.75 m. They are considered mounted on the structure in the way shown schematically in Fig. 5. Their fundamental natural frequency when their ends that are connected to the structure are held fixed is equal to 0.5 Hz in all three cases. In all three cases, too, the damping ratio in their fundamental mode is assumed equal to 0.1 per cent of critical.

The ground motion selected for the analysis are the first 80 seconds of the acceleration time history which results from combining vectorially along the direction that maximizes the peak ground acceleration the two horizontal ground acceleration records obtained at the SCT station during the aforementioned 1985 Mexico City earthquake. Its peak ground acceleration, which occurs at a time of 60.34 sec, equals 0.188 g. This ground motion is considered a critical one for the building being considered and the nonstructural components mounted on it because it is representative of the motion that caused the multiple building failures during the 1985 earthquake in Mexico City. Consequently, it is likely that it will generate significant nonlinear deformations in the building and large stresses in the nonstructural components. In addition, its dominant frequency is around 0.5 Hz, which coincides with the natural frequency of the nonstructural components.

The time-history analysis is performed using a computer program for the nonlinear analysis of plane frames developed by Hanna (1989), after its modification to be able to consider systems with different damping constants in different members of the system. In this analysis, the nonstructural component and its supporting structure are considered as one unit. It is assumed, further, that the behavior of the beams and columns of the building is bilinear with yield moments equal to the ultimate moments listed in Table 2 and a post-yield stiffness equal to 2 per cent of their stiffnesses before yield. The nonstructural components are considered to remain linear at all times. The influence of the axial force in the yielding of the columns is neglected.

In the calculation of the desired maximum shear force capacities, the lateral forces acting on the masses of each of the nonstructural components are determined first using the proposed formulas. Then, on the basis of these forces and using concepts from statics, the shear forces in each of the elements of the nonstructural components are determined. Note, however, that since the design spectrum specified by the Mexico City building code does not envelope the acceleration response spectrum for the ground motion herein being considered (Villaverde, 1994), for the sake of a meaningful comparison the approximate lateral forces are calculated using a value of ZIC equal to 1.14, which corresponds to the ordinate that envelopes the entire 5 per cent damping response spectrum for that ground motion. Similarly, since for the purpose of this comparison it is meaningless to consider safety factors and the possible deviations in the ductility factor for the structure, the reduction factor R_p in Eq. 2 is assumed equal to 4; i.e., the reduction factor assumed in the design of the structure. Note too that although in this case the fundamental natural frequencies of the building and the nonstructural components are known, no reduction in the value of the amplification factor C_p is considered since the values of these natural frequencies are sufficiently close to one another to justify the assumption of resonant frequencies.

Table 2. Masses and stiffnesses of nonstructural components in comparative study

Parameter	No.	System One-mass	System Three-mass	System Four-mass
Mass (Mg)	m_1	0.00450	0.00300	0.00150
	m_2		0.00150	0.00450
	m_3		0.00100	0.00450
	m_4			0.00150
Stiffness (kN/m)	k_1	0.04440	0.05584	0.00740
	k_2		0.03722	0.01110
	k_3		0.01861	0.06663
	k_4		0.00931	0.00663
	k_5			0.01110
	k_6			0.00740

Table 3. Shear force capacities and demands in elements of nonstructural components in comparative study

System	Element	Shear force capacity (kN)	Shear force demand (kN)
1-mass	1	0.226	0.212
3-mass	1	0.137	0.109
	2	0.064	0.052
	3	0.010	0.003
	4	0.034	0.032
4-mass	1	0.036	0.034
	2	0.032	0.034
	3	0.135	0.096
	4	0.202	0.085
	5	0.048	0.028
	6	0.055	0.029

The shear force demands and capacities obtained are listed in Table 3. As it may be seen from this table, the demands imposed by the selected earthquake ground motion in all the elements of the three nonstructural components considered in the analysis are equal or less than the shear capacities computed with the proposed simplified procedure. These results indicate thus that the proposed procedure is capable of leading to the safe design of nonstructural components in buildings.

CONCLUSIONS

A simplified procedure has been presented for the design of nonstructural components attached to building structures. The procedure takes into consideration the dynamic interaction between the structure and the nonstructural component, the level above the base of the structure of the point or points where the nonstructural component is attached to the structure, and the number of such attachment points. It utilizes the design spectra specified by building codes for the design of the structure to define the earthquake forces exciting the nonstructural component. The procedure, notwithstanding, is uncomplicated and with a format similar to the one used to determine the lateral seismic forces for the design of buildings. In a numerical comparative study with various nonstructural components mounted on a multistory building, it is shown that nonstructural components designed with it are capable of sustaining a critical ground motion without failing. Considering the basis of its derivation, its simplicity, and the results from this comparative study, it may be thus concluded that the procedure is suitable for the development of rational recommendations for the safe design of nonstructural components in buildings and its incorporation into building code provisions.

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