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DESIGN-ORIENTED APPROACH FOR SEISMIC NONLINEAR ANALYSIS OF NONSTRUCTURAL COMPONENTS

Roberto VILLAVERDE[1]

SUMMARY

A design-oriented simplified method is proposed for the seismic design of nonlinear nonstructural components attached to nonlinear building structures. The method is based on a previously developed simplified procedure for linear systems and a technique that is analogous to the reduction of response spectrum ordinates by a ductility factor and involves the use of reduced natural frequencies and augmented damping ratios to linearise the nonlin-ear systems. Its application requires only the use of the geometric characteristics, weights, and ductility factors of the nonstructural component and its supporting structure, as well as the elastic design spectra specified by building codes for the design of the structure. Presented also are a numerical example that illustrates the appli-cation of the method and the results of a comparative numerical study carried out with three nonstructural com-ponents alternatively attached to two multi-storey buildings. Based on the simplicity and rationality of the resulting formulas and the results of this comparative study, it is concluded that the proposed method represents a simple but effective procedure for the design of nonstructural components in buildings.

INTRODUCTION

In recognition of their vulnerability to the effects of earthquakes and the importance of their survivability from a safety and an economic point of view, many methods have been proposed during the last few decades for the seismic analysis of nonstructural components attached to building structures. For the most part, however, these methods have been derived specifically for linear nonstructural components mounted on linear structures. These methods, therefore, cannot be used directly to estimate the maximum response of nonstructural components under an extreme seismic event since, by design, their supporting structures are supposed to incur into their nonlinear range of behaviour in such a case. These methods cannot take into account, either, the fact that many nonstructural components or their anchors are capable of resisting large inelastic deformations. Since, as pointed out by Lin and Mahin (1985), Sewell *et al*. (1989), and Schroeder and Backman (1994), the nonlinear behaviour of a building and a nonstructural component may affect the response of the nonstructural component either in the form of a significant reduction or a substantial amplification over the corresponding linear response, the use of linear methods in the analysis of nonstructural components may therefore lead to unrealistic designs.

A simple and well-know method to take into account the nonlinearity of a structural system is that of reducing the ordinates of the response spectrum that defines the input to the system by a ductility factor that is representative of its overall capability to deform beyond its elastic limit. This method, however, cannot be applied to the analysis of a nonstructural component because its seismic response also depends on the response of its supporting structure, and because the ductility of a nonstructural component is, in general, different from the ductility of its supporting structure. Notwithstanding these complications, it is nevertheless possible to use a similar procedure in which, instead of reducing response spectrum ordinates by a ductility factor, the natural frequencies of the system are reduced and its damping ratios are augmented in a way that is consistent with this reduction in response spectrum ordinates. It is the purpose of this paper to present a simplified method that has been developed for the seismic design of nonstructural components in buildings on the basis of this approach.

[1] *Department of Civil Engineering, University of California, Irvine, CA 92697, USA*

EFFECTIVE NATURAL FREQUENCIES AND DAMPING RATIOS

Consider an elastoplastic single-degree-of-freedom system with initial stiffness K, mass M, and yield deformation u_y. The force that makes such a system yield may be expressed as

$$F_y = Ku_y \tag{1}$$

which in accordance with the conventional definition of ductility factor may also be expressed as

$$F_y = K(u_m)_{in} / \mu \tag{2}$$

where $(u_m)_{in}$ denotes the maximum (inelastic) displacement experienced by the system under a given excitation and μ is the corresponding ductility factor. However, in accordance to the rules suggested by Newmark and Hall (1982) to construct nonlinear response spectra from their linear counterparts, it may be assumed that

$$(u_m)_{in} = \lambda(u_m)_{el} \tag{3}$$

where $(u_m)_{el}$ denotes the maximum displacement that would be experienced by the system if it is considered to behave linearly under all excitation levels, and

$$\lambda = 1 \text{ if } f \le 2Hz; \quad \lambda = \mu / \sqrt{2\mu - 1} \text{ if } 2 < f < 8Hz; \quad \lambda = (33 - f)\mu / 25\sqrt{2\mu - 1} \text{ if } 8 \le f \le 33Hz \tag{4}$$

in which f denotes the natural frequency of the system in Hz. Under such an assumption, the yield force in the elastoplastic system may therefore be approximately taken as

$$F_y = K(u_m)_{el} /(\mu / \lambda) = \tilde{K}(u_m)_{el} \tag{5}$$

where

$$\tilde{K} = K /(\mu / \lambda) \tag{6}$$

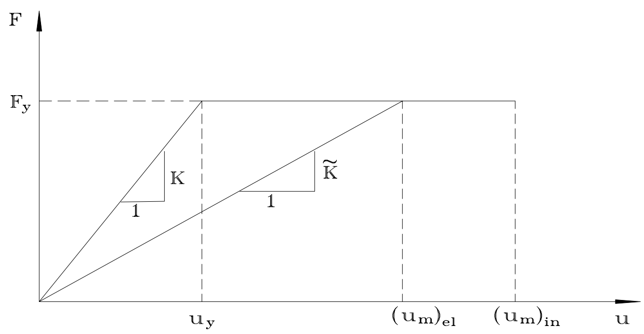

Figure 1. —Reduced stiffness of equivalent linear SDOF system

Observe thus that, as shown in Figure 1, the yield capacity of an elastoplastic single-degree-of-freedom system may be determined on the basis of a linear analysis, provided its original initial stiffness is reduced by the factor μ / λ as indicated by Eq. 6. Alternatively, this yield capacity may be calculated according to

$$F_y = \tilde{\omega}^2 M(u_m)_{el} \tag{7}$$

where $\tilde{\omega}$ is a reduced or effective natural frequency defined as

$$\tilde{\omega} = \sqrt{\tilde{K} / M} = \sqrt{K / M} / \sqrt{\mu/\lambda} = \omega / \sqrt{\mu/\lambda} \tag{8}$$

in which ω depicts the circular natural frequency of the original system before yield.

The damping ratio that should be considered with such an equivalent linear system may be obtained by considering that the damping mechanism in the linear and nonlinear systems is approximately the same, which is a conservative assumption. That is, one may assume that the damping constants of the two systems are approximately equal to each other. In this manner, if $\tilde{C}$ and $\tilde{\xi}$ respectively denote the damping constant and damping ratio of the equivalent linear system, and if C and ξ are the corresponding parameters of the original elastoplastic system, according to the definition of damping ratio one has that

$$\tilde{C} = 2\tilde{\xi}\tilde{\omega}M \tag{9}$$

and hence the damping ratio in the equivalent linear system may be written as

$$\tilde{\xi} = C / 2\tilde{\omega}M = 2\xi\omega M / 2\tilde{\omega}M = (\omega / \tilde{\omega})\xi \tag{10}$$

which in words means that to have the same damping mechanism in the equivalent linear system as in the original nonlinear one, it is necessary to augment the original damping ratio by a factor that is equal to the ratio of the original natural frequency of the system to the reduced one.

Finally, since the maximum elastic deformation experienced by the system when the system is subjected to a given excitation may be obtained from the displacement or acceleration response spectrum of this excitation, then the system's yield capacity may alternatively be expressed as

$$F_y = \tilde{\omega}^2 M (u_m)_{el} = \tilde{\omega}^2 M\, SD(\tilde{\omega}, \tilde{\xi}) = M\; SA(\tilde{\omega}, \tilde{\xi}) \tag{11}$$

where $SD(\tilde{\omega}, \tilde{\xi})$ and $SA(\tilde{\omega}, \tilde{\xi})$ respectively denote the spectral displacement and spectral acceleration corresponding to the effective natural frequency and effective damping ratio of the equivalent linear system.

The concepts introduced above can also be extended to the case of an elastoplastic multidegree-of-freedom system if it is assumed that all the resisting elements of the system have the same ductility capacity, that this ductility capacity is characterised by a ductility factor μ, that the damping matrix of the nonlinear system is equal to the damping matrix of its linear counterpart, and that both damping matrices are of the Rayleigh type. In such a case, it can be shown that, in similarity with the SDOF system case, the MDOF system's effective natural frequency, effective damping ratio, and yield capacities (i.e., the modal forces that would make the system's resisting elements yield) in its *rth* mode respectively result as

$$\tilde{\omega}_r = \omega_r / \sqrt{\mu / \lambda} \tag{12}$$

$$\tilde{\xi}_r = (\omega_r / \tilde{\omega}_r)\xi_r \tag{13}$$

$$\{F_y\} = [M]\{\varphi\}_r\, SA(\tilde{\omega}_r, \tilde{\xi}_r) \tag{14}$$

where ω_r and $\{\varphi\}_r$ respectively are the *rth* natural frequency and mode shape of the original system before yield, μ is the aforementioned common ductility factor, λ is defined by Eq. 4, ξ_r represents its *rth* damping ratio, and $[M]$ is its mass matrix. Observe, thus, that, as in the SDOF case, the yield capacity of the resisting elements of a MDOF system may be estimated on the basis of a linear analysis and a linear response spectrum, provided the system's original natural frequencies and damping ratios are substituted first by the corresponding effective ones.

DESIGN-ORIENTED SIMPLIFIED METHOD

A design-oriented simplified method to determine in a conservative but simple way equivalent static lateral forces for the seismic design of nonstructural components attached to buildings has been developed by incorporating the concepts introduced above into a procedure previously proposed by the author in a recent study (Villaverde, 1997). The major assumptions made in the derivation of the method are given in detail in this previous study. However, for the incorporation of the approach proposed in this paper to account for the nonlinearity of the two subsystems, two additional assumptions are made. First, it is assumed that both the structure and the nonstructural component possess elastoplastic behaviour and that this behaviour is characterised by their effective natural frequencies and damping ratios. Second, it is assumed that the input response spectrum for the combined structure-nonstructural component system is the design spectrum specified for the structure, adjusted to account for the increased damping ratio of the system when the structure and the nonstructural component are considered as nonlinear systems. This adjustment is based on the consideration of an increase in the damping ratio of the system from its original value to the effective one, and by assuming that the excitation is a harmonic one in resonance with the system; that is, by assuming that the amplification factor that defines the envelope to a response spectrum is equal to $1/2\xi$, where ξ is the damping ratio being considered.

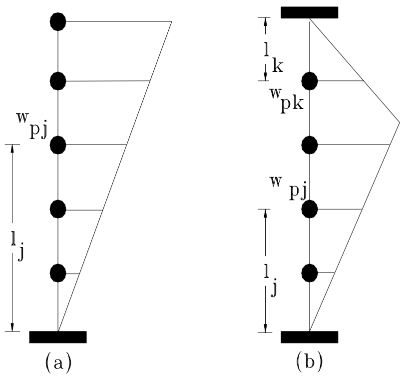

Figure 2. — Assumed mode shapes for components with (a) one and (b) two points of attachment

The method is intended to be valid for the design of nonstructural components connected to a building structure at one or two points. It may also be used, however, for a component with more than two attachment points by breaking the component up into a series of subcomponents with one or two attachment points each and by considering each of these subcomponents separately. It involves the calculation of equivalent static lateral forces whose value is intended to be greater than or equal to the maximum (ultimate) lateral forces that may be generated by a specified design earthquake on the masses of the nonstructural component. These forces are determined according to

$$F_{pj} = \frac{w_{pj}\, l_j}{\sum_{j=1}^{n} w_{pj}\, l_j} V_p \tag{15}$$

where F_{pj} is a force acting at the *jth* mass of the nonstructural component; w_{pj} is the weight of this *jth* mass; and l_j is the distance to the same mass measured in the case of a single attachment point from this attachment point (see Figure 2a). In the case of a nonstructural 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 Figure 2b). In the case of a mass located directly at such point of maximum deflection, l_j is measured from the support that is the farthest away from that mass. In addition, n represents the total number of masses in the nonstructural component and V_p its base shear or the sum of the shears at its supports. V_p is given by

$$V_p = \frac{IC}{\sqrt{\mu}} I_p C_p w_p \tag{16}$$

in which I = structure importance factor; C = ordinate corresponding to the effective fundamental natural period of the structure in the acceleration response spectrum specified for the design of the structure; μ = ductility factor specified for the design of the structure; I_p = nonstructural component importance factor (specified by a local building code or arbitrarily selected by the designer); w_p = total weight of the nonstructural component; and C_p = a component amplification factor calculated according to

$$C_p = \frac{1}{2\sqrt{\left|\frac{w_p}{W} - \frac{0.0025\mu}{\Phi_0^2}\right|}} \leq 400\,\Phi_0 \tag{17}$$

where $$\Phi_0 = \frac{W\, h_{av}}{\sum_{i=1}^{N} W_i\, h_i} \tag{18}$$

in which W_i and h_i respectively denote the weight and elevation above ground of the building's *ith* floor; W is the total weight of the building; h_{av} is the average of the elevations above ground of the points of the building to which the nonstructural component is attached, and N denotes the number of floors in the building.

The derivation of the formulas established above is based on the assumption that the fundamental natural frequency of the nonstructural component is in resonance with the fundamental natural frequency of the structure; i.e., that the values of these two frequencies 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 to carry out its seismic design, it may be nonetheless overly conservative for those cases in which those two natural frequencies are not close to one another. As a means to reduce the conservatism involved in such cases, the amplification factor C_p may be replaced by a modified amplification factor C_m that varies linearly with the period ratio T_p / T between the maximum value, C_p, when this ratio is close to 1.0, and the minimum value, C_0, when such a ratio is significantly different from 1.0. In the definition of this period ratio, T_p represents the fundamental natural period of the nonstructural component and T the natural period of the structure which in addition to being a natural period in one of its lower modes is also the closest to T_p. It should be noted, however, that since the nonlinear behaviour of the structure and the nonstructural component is accounted for by using effective natural frequencies and damping ratios, this natural period ratio should determined in terms of effective natural periods.

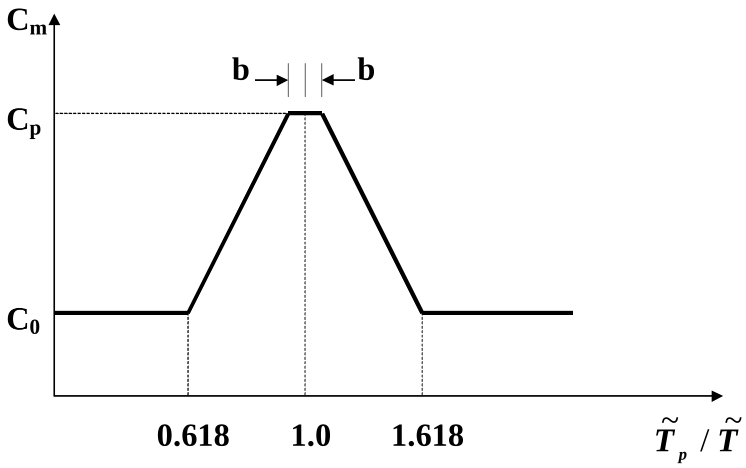

Figure 3. — Variation of modified amplification factor with natural period ratio

The variation of the proposed modified amplification factor C_m is shown in Figure 3, together with the limits of the period ratio beyond which C_m should be considered equal to C_0, and those that define the range for which the modes of the structure and the nonstructural component should be considered in resonance with one another. Note that in Figure 3 $\tilde{T}_p$ and $\tilde{T}$ are effective periods defined as

$$\tilde{T}_p = \sqrt{\mu_p / \lambda_p}\, T_p \qquad \tilde{T} = \sqrt{\mu / \lambda}\, T \tag{19}$$

where T_p and T are as previously defined, μ_p and μ respectively denote the ductility factors selected for the design of the nonstructural component and the structure, and λ_p and λ are factors corresponding to the natural periods T_p and T, respectively, defined according to Eq. 4. C_0 and b are given by

$$b = \tfrac{1}{2}\Phi_0 \sqrt{w_p / W} \tag{20}$$

$$C_0 = \sqrt{\Phi_0^2 + (1+\Phi_0)^2} \tag{21}$$

ILLUSTRATIVE EXAMPLE

To illustrate the use of the proposed method, it will be employed to determine the design lateral forces for the three-mass nonstructural component shown in Figure 4, when the component is rigidly connected to the 4th and 6th stories of the 6-story office building shown in this same figure. The building is located over a deposit of stiff soil in the city of Irvine, California, and is structured with ordinary 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. Its first two natural periods are equal to 1.66 and 0.59 seconds. The nonstructural component is an ordinary architectural fixture for which one can consider an importance factor equal to 1.0. In addition, it may be modelled as a three-degree-of-freedom shear beam with four equal segments with a length of 1.65 m each. Each of its three masses weighs 4.4 kN, and hence its total weight is 13.2 kN. Its fundamental period is estimated to be equal to 0.5 seconds when its two ends are considered fixed. A ductility factor of 2 may be considered in its design. The 1997 version of the Uniform Building Code will be used to define the earthquake input to the building.

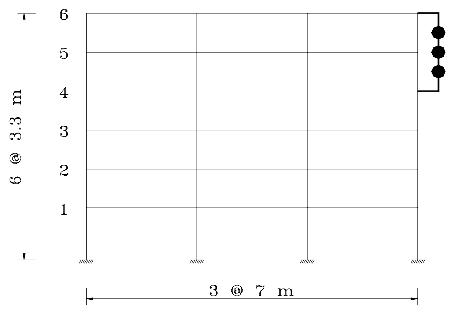

Figure 4. — Building and nonstructural component considered in illustrative example

For the calculation of the desired lateral forces, it is noted first that the 1997 version of the Uniform Building Code specifies a ductility factor of 4.5 for the design of this type of structure. Then, the Φ_0 value corresponding to the nonstructural component's two attachment points and the amplification factor C_p will be calculated next. For this purpose, note that the average of the elevations above ground of such two attachment points is 16.5 m. Hence, substitution into Eq. 18 of this value and the floor weights given above yields a Φ_0 value of 1.43. Similarly, by substitution into Eq. 17 of this Φ_0 value, the given weights of the structure and the nonstructural component, and μ = 4.5, one obtains a C_p value of 7.45. However, since in this case the fundamental natural frequency of the nonstructural component is known, one can reduce the value of this amplification factor using Figure 3. To this end, note that according to the specified ductility factors, the effective natural periods of the building and the nonstructural component result as $\tilde{T}_1 = 1.66\sqrt{4.5} = 3.52$ s; $\tilde{T}_2 = 0.59\sqrt{4.5} = 1.25$ s; $\tilde{T}_p = 2.0\sqrt{2} = 2.83$ s. It may be seen, thus, that in terms of these effective values the first structural period is the closest to the fundamental natural period of the nonstructural component. Consequently, the modified amplification factor for the nonstructural component will be calculated on the basis of these two periods; that is, for an effective period ratio of 0.80. Note too that in this case the value of the parameters C_0 and b that appear in Figure 3 are respectively equal to 2.82 and 0.023. Hence, the reduced amplification factor C_m results equal to 5.17. Also, since for the given building and from the recommendations of the 1997 version of the Uniform Building Code one has that I = 1.0 and C = 0.219, then by substitution into Eq. 16 of these values, the value of C_m determined above, μ = 4.5, and the given importance factor and total weight of the nonstructural component one finds that V_p = 7.05 kN.

To distribute now this force of 7.05 kN among the three masses of the nonstructural component, one needs to obtain first its point of maximum deflection under lateral forces equal to the weight of its masses and define the distances l_j in Eq. 15. Note, however, that the nonstructural component under consideration is symmetric in mass and geometry and therefore such a point of maximum deflection is located at its geometric centre. By inspection, it can be determined thus that $l_1 = l_3$ = 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. 15 yields $F_{p1} = F_{p3}$ = 1.76 kN and F_{p2} = 3.53 kN.

COMPARATIVE STUDY

To assess whether or not nonstructural components designed with the proposed method would survive a critical earthquake ground motion, a comparative analysis is performed with three different nonstructural components mounted alternatively on a 10-story building and a 13-story one. In this analysis, the shear force capacities and design ductilities of the nonstructural components' resisting elements are compared against the shear force and deformation ductility demands that are imposed on these elements when the base of the buildings is excited by a critical ground motion. For this purpose, it is assumed that the aforementioned shear force capacities are equal to the shear forces that act on the resisting elements of the components when their masses are subjected to the equivalent lateral forces obtained with the proposed method. Two different cases are considered in the determination of these shear capacities. In the first, the components are assumed able to resist deformation ductility factors of up to 2, while in the second they are assumed able to resist deformation ductility factors of up to 8. The shear force and ductility demands are obtained by means of a nonlinear time history analysis in which a nonstructural component and its supporting structure are considered together as a single unit. The beams and columns of both buildings are assumed to possess a bilinear behavior with yield moments defined by their ultimate moments and a post-yield stiffness equal to 2 per cent of their stiffness before yield. Similarly, it is assumed that the nonstructural components behave as elastoplastic shear beams rigidly attached to their supports and that their resisting elements' yield shear strength is equal to their shear capacity. Both buildings are assumed with a damping matrix proportional to their respective stiffness matrices and a damping ratio of five per cent in their fundamental modes. The seismic design of the 10-story building is carried out considering a ductility factor of 4, whereas that of the 13-story building considering a ductility factor of 6. In each case, a component's design is considered satisfactory if the ductility demands imposed on the component's resisting elements by the considered ground motion are equal or less than the ductility factor assumed in the component's design.

The characteristics of the two buildings and the three nonstructural components are described in detail elsewhere (Villaverde, 1997, 1998). The 10-story building represents an actual reinforced concrete office building in Mexico City, located in the soft soil area of the city. The 13-story one corresponds to an existing reinforced concrete commercial building located in Sherman Oaks, California. Both buildings experienced significant damage during the Mexico City and Northridge, respectively, so an incursion into their nonlinear range of behaviour during these earthquakes was likely. The three nonstructural components studied were: (a) a single-mass system with a single point of attachment; (b) a three-mass system with two points of attachment; and (c) a four-mass system with three points of attachment. The single-mass component is considered mounted on the roof in both buildings. The three-mass component is assumed attached to the 4th and 8th floors of the 10-story building and 8th and 13th floors of the 13-story one. The four-mass component is considered attached to the 4th, 7th, and 10th floors in the case of the 10-story building and 6th, 8th, and 10th floors in the case of the 13-story building. The ground motions selected for the analysis correspond, respectively for the 10- and 13-story buildings, to the acceleration time histories recorded at the SCT station during the 1985 Mexico City earthquake and at the base of the Union Bank building during the 1994 Northridge earthquake.

As indicated earlier, the components' shear force capacities were determined on the basis of lateral forces calculated using the formulas introduced above. Consequently, these shear force capacities depend on the code-specified coefficient C, which is supposed to represent a spectral acceleration expressed as a fraction of the acceleration of gravity. It may be noted, however, that the design spectra specified by the Mexico City and California building codes do not envelope the elastic acceleration response spectra of the ground motions considered in this comparative analysis. Hence, it would be meaningless to compare the shear force demands imposed by these ground motions against the shear force capacities calculated on the basis of code-specified C values. For the sake of a meaningful comparison, therefore, the approximate lateral forces are calculated using instead C values equal to the ordinates corresponding to the effective fundamental natural periods of the buildings in the 5-percent-damping acceleration response spectra of the considered ground motions. For the buildings and ground motions under consideration, this corresponds to a C value of 1.14 for the design of the nonstructural components in the 10-story building and 0.5 for the design of those in the 13-story building. It is worthwhile to mention too that in each case the component amplification factor was determined by considering the reductions that can be obtained in terms of the relationship between the natural frequencies of the building and the nonstructural component. As established in the recommended procedure presented above, this relationship was defined in terms of the effective fundamental natural frequency of the component and the effective natural frequency of the building that turns out to be the closest in value to component's effective fundamental natural frequency.

The shear force capacities, shear force demands, and deformation ductilities obtained are listed in Tables 1 and 2, from which it can be seen that the ductility demands imposed by the selected ground motions on the resisting elements of the nonstructural components are in every case less than the ductilities considered in their design.

CONCLUSIONS

Based on the simplicity and rationality of the resulting formulas and the results of the performed comparative study, it is concluded that the proposed method represents a simple but effective procedure for the design of nonstructural components in buildings.

Table 1. — Shear force capacity, shear force demand, and deformation ductility demand in elements of nonstructural components in 10-story building

Compo-nent	Ele-ment	Components with design ductility of 2			Components with design ductility of 8		
		Shear force capacity (kN)	Shear force demand (kN)	Ductility demand	Shear force capacity (kN)	Shear force demand (kN)	Ductility demand
1-mass	1	0.138	0.138	1.04	0.103	0.103	1.05
3-mass	1	0.089	0.084	0.94	0.070	0.067	0.96
	2	0.041	0.041	1.02	0.032	0.032	1.09
	3	0.006	0.005	0.81	0.005	0.004	0.90
	4	0.022	0.022	1.14	0.017	0.017	1.20
4-mass	1	0.034	0.032	0.94	0.026	0.026	1.12
	2	0.030	0.030	1.09	0.023	0.023	1.16
	3	0.126	0.095	0.76	0.095	0.080	0.85
	4	0.126	0.085	0.67	0.095	0.079	0.83
	5	0.030	0.028	0.94	0.023	0.023	1.10
	6	0.034	0.029	0.82	0.026	0.026	0.97

Table 2. — Shear force capacity, shear force demand, and deformation ductility demand in elements of nonstructural components in 13-story building

Compo-nent	Ele-ment	Components with design ductility of 2			Components with design ductility of 8		
		Shear force capacity (kN)	Shear force demand (kN)	Ductility demand	Shear force capacity (kN)	Shear force demand (kN)	Ductility demand
1-mass	1	0.0391	0.0318	0.82	0.0391	0.0318	0.82
3-mass	1	0.0355	0.0259	0.73	0.0355	0.0259	0.73
	2	0.0165	0.0123	0.75	0.0165	0.0123	0.75
	3	0.0025	0.0012	0.48	0.0025	0.0012	0.48
	4	0.0088	0.0075	0.85	0.0088	0.0075	0.85
4-mass	1	0.0062	0.0055	0.94	0.0062	0.0055	0.94
	2	0.0038	0.0038	1.51	0.0038	0.0038	1.51
	3	0.0336	0.0171	0.61	0.0336	0.0171	0.61
	4	0.0336	0.0169	0.50	0.0336	0.0169	0.50
	5	0.0038	0.0038	1.31	0.0038	0.0038	1.31
	6	0.0062	0.0055	0.78	0.0062	0.0055	0.78

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