Fifth International Workshop on the Seismic Performance of Non-Structural Elements (SPONSE)

Analytical Studies in Support of an Improved Approach to the Design of Acceleration-Sensitive Nonstructural Elements

Athanasia K. Kazantzi[1], Eduardo Miranda[2], Dimitrios Vamvatsikos[1], Ahmed Elkady[3] and Dimitrios Lignos[4]

[1] School of Civil Engineering, National Technical Univ. of Athens, Athens 15780, Greece
e-mail: kazantzi@mail.ntua.gr

[2] Dept. of Civil and Environmental Engineering, Stanford Univ., Stanford, CA 94305, USA
e-mail: emiranda@stanford.edu

[3] Dept. of Civil, Maritime and Envir. Engineering, Univ. of Southampton, Southampton SO16 7QF, UK
e-mail: a.elkady@soton.ac.uk

[4] RESSLab, École Polytechnique Fédérale de Lausanne, Lausanne CH-1015, Switzerland
e-mail: dimitrios.lignos@epfl.ch

Abstract. This paper summarizes a series of analytical studies that were conducted in connection with an improved approach for the design of acceleration-sensitive nonstructural elements. In the new approach, bracing to secure nonstructural elements to the structure is designed and detailed to experience nonlinearities to limit forces acting not only in the nonstructural elements but also in the attachments to the structure and in the attachment(s) to the nonstructural element. The project was sponsored by the Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (SERA) and involved shake table testing at the University of Bristol to validate the proposed novel approach as well as analytical studies. Prior to the testing, a series of analytical studies were conducted to examine the feasibility of the proposed approach and for selecting motions to be used in the shake table tests. By using exclusively motions recorded in instrumented buildings in California it is shown that acceleration demands in nonstructural elements can easily exceed 2 or 3g, but that by allowing nonlinearity to occur in the bracing element, acceleration and forces can be greatly reduced even with small levels of nonlinearity. In particular, it is demonstrated that given the frequency content of floor motions, which correspond to ground motions amplified and filtered by the structure, the reductions in accelerations and forces are much larger than those that are produced under ground motions for similar levels of nonlinearity. Furthermore, it is shown that the proposed approach not only results in large reductions in forces and accelerations, especially for elements tuned to any of the modal frequencies of the supporting structure but, simultaneously, it can also achieve substantial reductions in lateral deformations with respect to those that would occur on nonstructural elements remaining elastic. Yet, another important advantage of the proposed approach is that force and deformation demands become far less sensitive to the period of vibration of the nonstructural element.

Keywords: Nonstructural elements, Component amplification, Effect of Yielding, Acceleration demands, Force demands, Displacement demands.

1. INTRODUCTION

It is now well-recognized that nonstructural elements represent most of the initial investment in buildings (Taghavi and Miranda, 2003; Filiatrault and Sullivan, 2014) and they play a key role in the functionality of buildings. It is then not surprising that their failure can lead to important consequences such as loss of functionality and large direct and indirect economic losses. A well-known example of the critical role of nonstructural elements on the functionality of buildings is the performance of the Sylmar County Hospital in the 1994 Northridge earthquake, in which despite the fact that the building did not suffer any apparent structural damages, it had to be evacuated and remained inoperable for several months due to extensive repair works required for its contents and nonstructural elements (Naeim, 2004). But additionally, in some cases, failure of the nonstructural elements could also lead to serious injuries and even loss of life. Examples of the latter occurred in the United States, U.S., during the Good Friday 1964 Alaska earthquakes (Ayres, 1973) and during the 1987 Whittier earthquake (Taly, 1988). In both cases, loss of life was attributed to the detachment and fall of architectural façade elements as a result of the earthquake.

Seismic provisions for nonstructural elements have been given much less attention than seismic provisions for the design of buildings and structures. This is mainly because, for many years, the primary goal of earthquake resistant design has been to avoid the collapse of the structure, with much less attention paid to the design of nonstructural elements. Unfortunately, this has led to seismic provisions for nonstructural elements that have many deficiencies. For instance, in the case of structures, there is consensus that local site conditions play a major role in the intensity and frequency content of ground motions and therefore on the level of response and level of seismic risk of structures built on different site conditions. This has led to the explicit incorporation of the effect of site conditions in the design of structures in most seismic codes since the 1970s. In contrast, the role of the supporting structure on the design of nonstructural components has largely been neglected or not properly accounted for. For example, in the U.S. the influence of the fundamental period of vibration of the supporting structure has not been considered in the calculation of design forces of nonstructural elements. Another problem in the field of nonstructural elements, is a fairly generalized misconception that paying attention to load path and providing a bracing element is enough to avoid earthquake damages and therefore design forces are not important. Apparently, this is not true, since practically any moderate earthquake that has struck an urban area has led to a large amount of nonstructural damage and many of this damage has occurred in elements that were braced and in which there was an apparent seismic design. Hence it is clear that simply installing a bracing element is by no means sufficient to secure an adequate seismic behaviour. The bracing elements require a certain strength, stiffness and deformation capacity to lead to an adequate performance. Figure 1 illustrates a couple of examples of this situation. The first example shows diagonal bracing elements, of which one failed as a result of having insufficient force capacity and/or ductility. The other example illustrates a roof mounted vibration-isolated equipment with attachments designed to resist lateral loads. Again, failure occurred as a result of insufficient force capacity and/or insufficient ductility in the elements bracing the equipment to the structure.

The two examples shown in Figure 1 also illustrate how challenging the seismic design of nonstructural components can be. Often nonstructural components are the end end of a long chain of aspects that affect the seismic demands of nonstructural components. Each of these aspects in this conceptual chain is subjected to large uncertainties. For example, there are very large uncertainities in the magnitude and location of future earthquakes. But even if the magnitude and location of a future earthquake were known, estimating the intensity of ground motions at a site is still highly uncertain. For example, for a given type of faulting mechanism, magnitude, distance and site conditions, spectral ordinates have logarighmic standard deviations in the order of 0.6 which means that, for a given earthquake (with known magnitude) at a given distance and site conditions, one could easily see changes in the level of intensity from one site to another at the same distance from the epicenter and in the same site conditions of a factor of four in the level of ground motion intensity. Furthermore, even if the median intensity at a site was to be known, it most likely


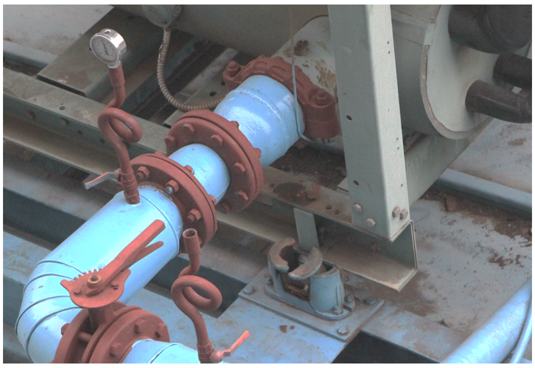

Figure 1. Examples of nonstructural elements that had bracing elements with seismic design but nevertheless experienced failure during an earthquake (Photos by Eduardo Miranda).

would have large variations in intensity with changes in direction due to directionality effects (e.g., Poulos and Miranda, 2022). The motion in the structure is influenced by soil structure interaction effects which depend on the local site conditions and type of foundation. The modifications include amplifications or deamplifications of the intensity of the ground motion as well as modification of the dynamic characteristics of the structure, such as periods of vibration, mode shapes and modal damping ratios with respect to those that would occur if the structure was fixed at the base. Depending on the fundamental period of vibration of the supporting structure, its lateral-force resisting systems, its modal damping ratios, the level of intensity and frequency content of the ground motion as well as the peak ground acceleration could be amplified by values in excess of six or be deamplified. Finally, the demands on the nonstructural element are influenced by the location of the element within the structure and by the mass, stiffness, strength and damping of the nonstructural component.

The main objective of this paper is to present a summary of a series of analytical studies conducted by the authors in connection with an experimental study to validate an new approach for the seismic design of nonstructural elements (Elkada et al., 2022; Miranda et al 2018a, 2018b). The proposed approach takes advantage of the unique characteristics of floor motions, which are characterized by ground motions that have been amplified and filtered by the supporting structure. In particular, floor motions are characterized by large amplifications at very specific frequencies that are equal or close to the modal frequencies of the supporting structure. Particular emphasis is placed on the selection of the recorded motions that were used in the shake tests. While the levels of amplifications are very large, it is shown that energy dissipation by means of viscous damping or hysteretic behaviour in a yielding element can produce significant reductions in acceleration and force demands that are much larger than those that would occur in nonstructural components at ground level, in other words, to components subjected to ground motions instead of floor motions. In the proposed approach, bracing elements that are located between the nonstructural elements and the structure are designed and detailed to yield during moderate and large earthquakes. Furthermore, they are designed to be the weakest element in the load path, allowing the design of the nonstructural element and the attachemets (anchors) of the bracing to the nonstructural element and to the structure to be designed for forces that can be estimated as a function of the capacity of the yielding element and therefore their seismic performance becomes more reliable.

2. SELECTION OF INPUT FLOOR MOTIONS

Unlike most shake table tests of nonstructural elements that typically make use of artificial (synthetic) motions to match floor spectra, such as the AC156 floor spectra that was developed to match code provisions and not the characteristics of motions that occur in buildings during earthquake, in this investigation we made exclusive use of floor motions recorded in instrumented buildings in order to employ

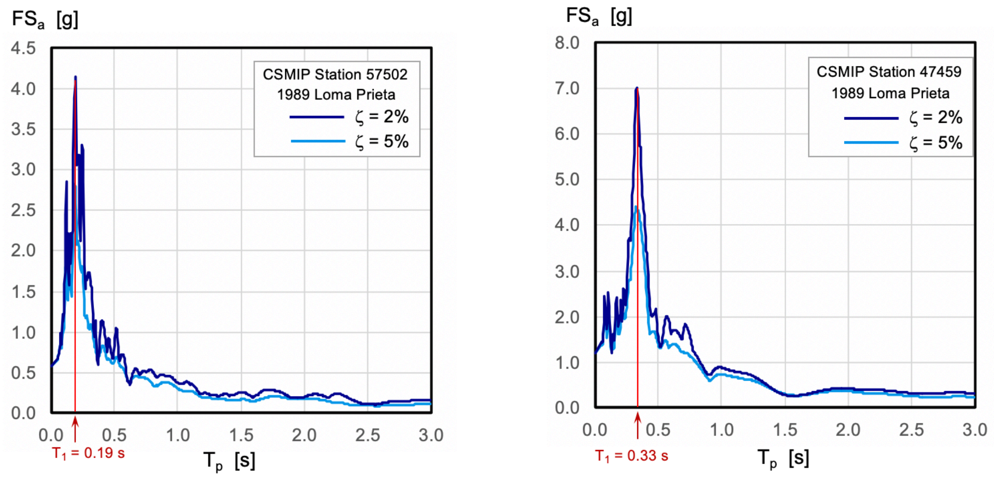

Figure 2. Floor spectra exhibiting large spectral ordinates at periods of vibration near the fundamental period of vibration of the supporting structure that were selected as input for the shake table tests.

motions with realistic amplitude and frequency content. Figure 2 shows the 2% and 5% damped floor response spectra computed for the floor motions recorded at the roof level on two instrumented buildings during the 1989 Loma Prieta earthquake.

The floor spectra on the left correspond to those at roof level of a two-story industrial building in the city of Milpitas, built in 1984, whose lateral-force resisting system comprises tilt-up walls. The peak ground acceleration in this direction was 0.14g which was amplified at roof level to 0.57g. This corresponds to an amplification of nearly four which is what would be expected to occur on average in lowrise buildings. This peak floor acceleration was subsequently amplified for periods of vibration smaller than about 0.5s and strongly amplified to experience accelerations in excess of 2g for periods close to the fundamental period of vibration of the supporting structure which is 0.19s. The floor spectra shown on the right correspond to those at roof level of a four-story reinforced-concrete shear wall commercial building in Watsonville. The peak ground acceleration in this direction was 0.36g which was amplified at roof level to 1.2g, corresponding to an amplification of about three. This peak floor acceleration was subsequently amplified for periods of vibration smaller than about 0.8s and deamplified for periods longer than 0.8s. The levels of acceleration were strongly amplified to experience accelerations in excess of 3g for periods close to the fundamental

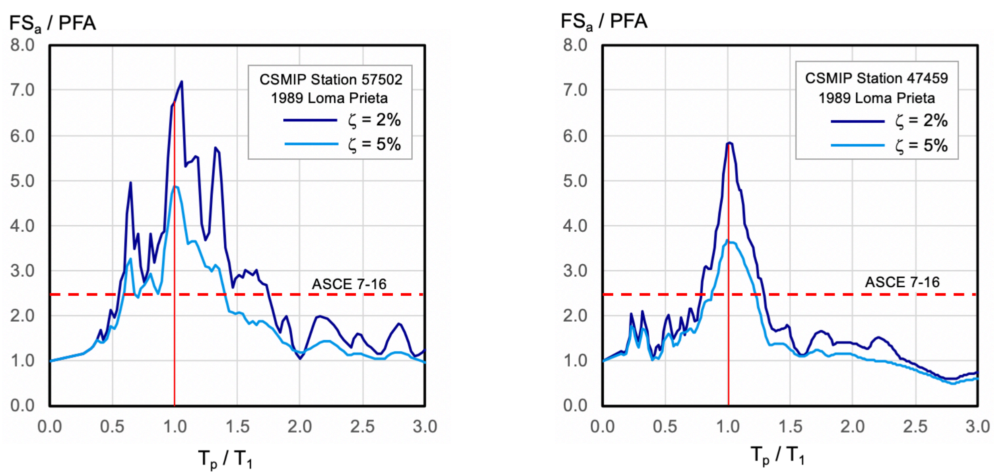

Figure 3. Binormalized floor spectra of recorded motions that were selected as input in the shake table tests which exhibit large amplifications of acceleration (i.e., in excess of four) at periods of vibration near the fundamental period of vibration of the supporting structure (i.e., at T_p/T_1 near one).

period of vibration of the supporting structure, which in this direction is 0.33s. Figure 3 shows the same 2% and 5% damped floor spcetra but now binormalized. In these spectra, the periods of the secondary system, in this case the periods of the nonstructural components, T_p, have been normalized by the fundamental period of the building, T_1. This binormaliztion was first proposed by Miranda (1991) for characterizing seismic demands on structures built on soft soils, whose spectra is also characterized by being narrow banded. The normalization of the abscissas provides the opportunity to study seismic demands not as a function of the period but as function of how close or far a period of vibration is to the predominant period of the ground motion. More recently, Kazantzi et al (2020a, 2020b) used the same normalization to study the seismic demands on nonstructural elements. Meanwhile the normalization of the floor spectral ordinates by peak floor acceleration provides information on the level of amplification of accelerations as the period of vibration of the nonstructural element approaches or gets far from the fundamental period of vibration of the supporting structure. It can be seen that, for nonstructural elements with 5% damping, the amplification for elements tuned or nearly tuned to the first mode of vibration of the supporting structure exceeds four. On the other hand, for nonstructural elements with 2% damping, the amplification of acceleration for elements tuned or nearly tuned to the first mode of vibration of the supporting structure are in the order of 5 or six for the selected motions. Figure 3 also indicates the component amplification factor a_p = 2.5 that is used in ASCE 7-16 for flexible components. As can be seen, the amplifications computed from recorded floor motions for tuned or nearly tuned nonstructural elements greatly exceed those in the U.S. seismic provisions whereas there are other spectral regions where the provisions are very conservative. In the latest version of ASCE 7 (ASCE, 2022) the component amplification factor a_p has been replaced by the so-called component resonance ductility factor, C_{AR}, which varies depending on the type of nonstructural element and on whether the component is supported at or below grade, or is supported above grade by a building structure. The largest value is C_{AR} =2.8 which is assigned to architectural components above grade that are flexible with low-deformability materials and attachments as well as for some vibration isolated equipment above grade. It can be seen that the small increase from 2.5 to 2.8 still falls very short from the levels of amplification computed from recorded floor motions shown in Figure 3.

Examples of 2% and 5% damped floor spectra obtained from motions recorded at roof level of taller instrumented buildings are shown in Figure 4. These motions were selected as possible candidates to be used in the shake table tests as representative of cases in which the nonstructural component is tuned to higher modes of vibration. Station 24370 corresponds to a six-story commercial building whose lateral-force resisting system consists of steel moment resisting frames. The fundamental period of vibration in the NS direction is 1.27s and the second mode where large floor spectral ordinates in excess of 1g are produced is 0.43s. The second example is a thirteen-story office building in the city of Hayward with a fundamental period of vibration of 1.32s and with high acceleration demands for periods near 0.44s and 0.25s which correspond to the second and third translational period of vibration in the EW direction. The third example is a flexible nineteen story office building in Los Angeles whose lateral-force resisting systems consists of

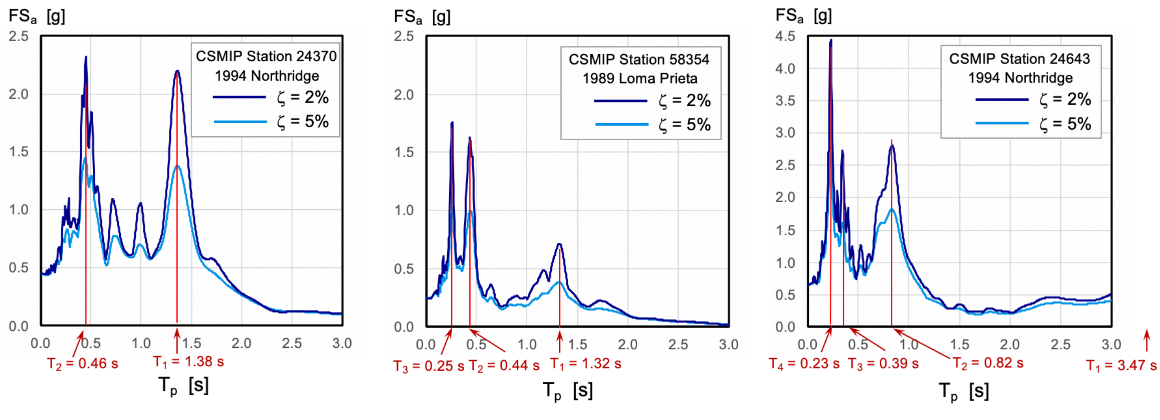

Figure 4. Examples of floor spectra exhibiting large spectral ordinates at periods of vibration near the higher modes of vibration of the supporting structure.

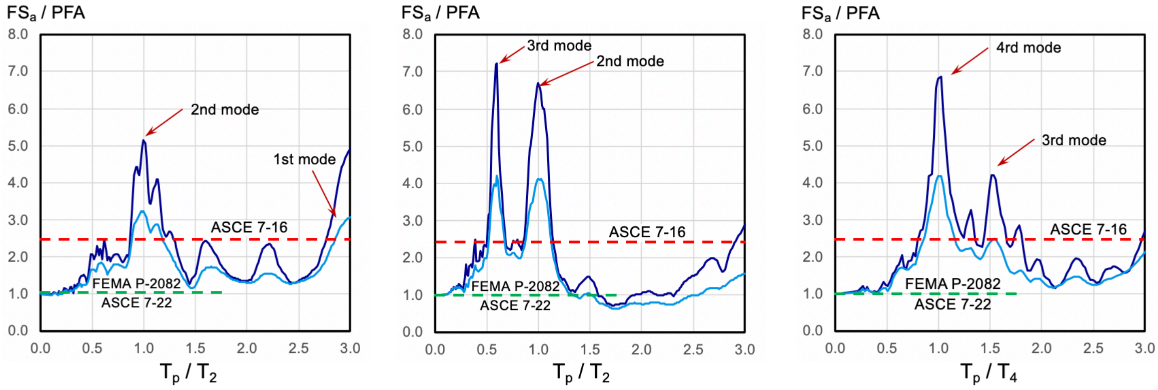

Figure 5. Examples of binormalized floor spectra exhibiting large amplifications of acceleration (i.e., in excess of four) at periods of vibration near higher mode periods of vibration of the supporting structure.

steel moment resisting frames. The fundamental period of vibration is 3.47s and high acceleration demands appear at periods near 0.82s, 0.39s and 0.23s that correspond to the second, third and fourth translational period of vibration in the NS direction of the building. As shown in this figure, unlike the new seismic provisions for nonstructural components stating that large amplifications (referred to in the provisions as "resonance") are unlikely to occur if the period of vibration is less than half of the fundamental period of vibration of the building, this is clearly not the case and in all three examples accelerations in excess of 1g are produced even in periods less than half of the fundamental period of vibration of the buildings. Figure 5 shows the same floor spectra but now in a binormalized form. As can be inferred from Figure 5, contrary to the new U.S. seismic provisions that consider the resonance as unlikely as to stipulate a component resonance ductility factor, C_{AR}=1.0—meaning an acceleration equal to the peak floor acceleration—nonstructural components tuned to higher modes could be subjected to amplifications of acceleration larger than four, suggesting that reducing design forces in this spectral region was not a step in the right direction. The motion recorded at roof level in CSMIP station 24370 at Burbank was selected to be used in the shake table tests at Bristol as a floor motion representative of one that can generate very large amplifications of acceleration for nonstructural elements with periods close to the second mode of vibration of the supporting structure.

Figures 3 and 5 show that the effect of damping of the nonstructural element has very different results depending on how close or far is the period of vibration of the component to one of the modal periods of the supporting structure. As shown in these figures, damping produces much larger reductions in seismic demands for nonstructural elements that are tuned to one of the modal periods of the supporting structure. This is consistent with previous observations by Kazantzi et al. (2020b) who conducted a study on the effect of damping on floor spectra.

Following the preliminary selection of some recorded floor motions, it was needed to verify that these motions were fairly representative of seismic demands that nonstructural elements can be subjected to. In other words, it was necessary to verify that pre-selected motions did not produce unusually low or unusually high amplifications. For this purpose, binormalized floor spectra were compared to statistical studies previously conducted by the first three authors (Kazantzi et al. 2020b). Figure 6 illustrates 113 binormalized floor motions along with their mean, median and 16th and 84th percentiles. The figure on the left corresponds to recorded motions in which the large amplifications occur at a period equal or close to the fundamental period of vibration of the supporting structure while the figure on the right corresponds to recorded motions in which the large amplifications occur at a period equal or close to periods of higher modes of vibration of the supporting structure. As can be inferred by inspecting Figure 6, nonstructural elements with damping ratios of 2% that are tuned to the fundamental period of vibration of the supporting structure are subjected to strong amplifications, 70% of which are between 5.8 and 9.5 with an average amplification of 7.4. The amplifications of the two binormalized floor spectra shown in Figure 3 for 2% damping have peak amplifications of 6.7 and 5.8 indicating that these high levels of accelerations in these example records are by no means unusual but are actually slightly smaller than mean amplifications that have been observed in motions recorded on instrumented buildings in California.

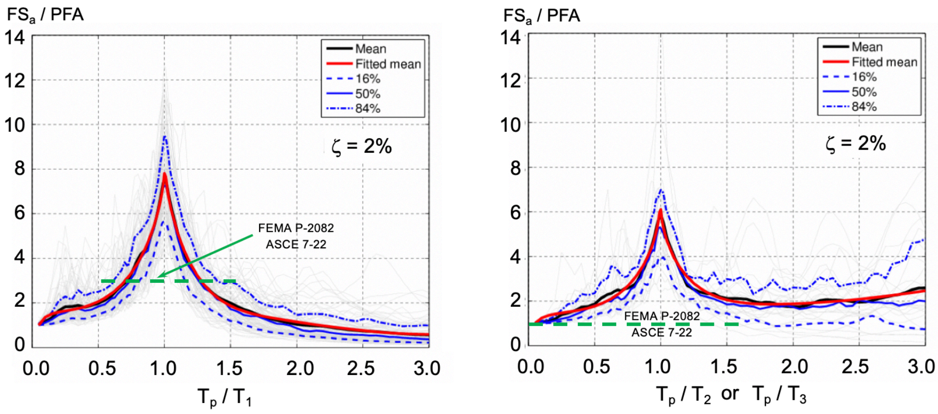

Figure 6. Statistical studies of amplifications of accelerations for flexible components on the left when periods are normalized by the first (fundamental) period of the supporting structure and on the right when normalized to the second or third mode of vibration of the supporting structure.

Meanwhile, nonstructural elements with damping ratios of 2% that are tuned to periods corresponding to higher modes of vibration of the supporting structure are also subjected to strong amplifications, 70% of which are between five and seven with an average amplification of 5.3.

3. EFFECT ON NONLINEARITY IN THE SECONDARY COMPONENT

Figure 7 shows force reduction factors computed from floor motion recorded in instrumented buildings. These reduction factors correspond to relatively small values of nonlinearity as measured by displacement ductility ratios of 1.5 and 2.0. It can be seen that these reduction factors are very different from those computed from ground motions recorded on rock or firm soils. In particular, they are characterized by having large force reductions in secondary systems for approximately the same periods for which large amplifications are produced (i.e., those shown in Figure 6). This means that by allowing only relatively small levels of nonlinearity to take place in the bracing of nonstructural elements that are tuned or nearly tuned to modal periods of the supporting structure it is possible to design for significantly smaller forces than those necessary to keep these elements elastic. For example, by allowing a ductility demand of only 1.5 to take place in components tuned to the first mode, it is possible to design for forces 3.6 times smaller than those necessary to maintain them elastic or 2.4 higher than those that on average could be used for broadband motions for the same level of nonlinearity. If the allowed level of nonlinearity is increased to a ductility of two, the design forces become 6.2 smaller than those necessary to maintain them elastic or 3.2 smaller than those that on average are produced in motions with broadband spectra for the same level of nonlinearity. Hence, the proposed novel design approach is particularly effective in reducing acceleration and force demands in components that are tuned or nearly tuned to modes of vibration of the supporting structure with strong contribution to the response of the structure. For more information on reduction factors for secondary systems, the reader is refered to Kazantzi et al. (2020c).

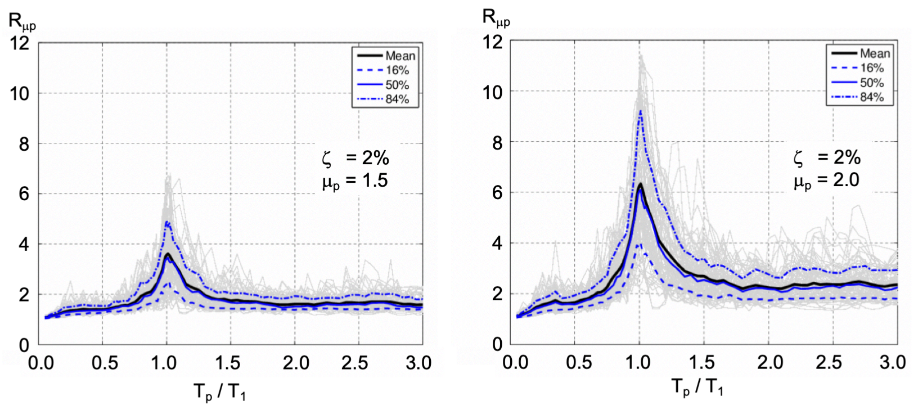

Figure 7. Statistical studies of the effect of level of nonlinearity on force reduction factors of nonstructural elements.

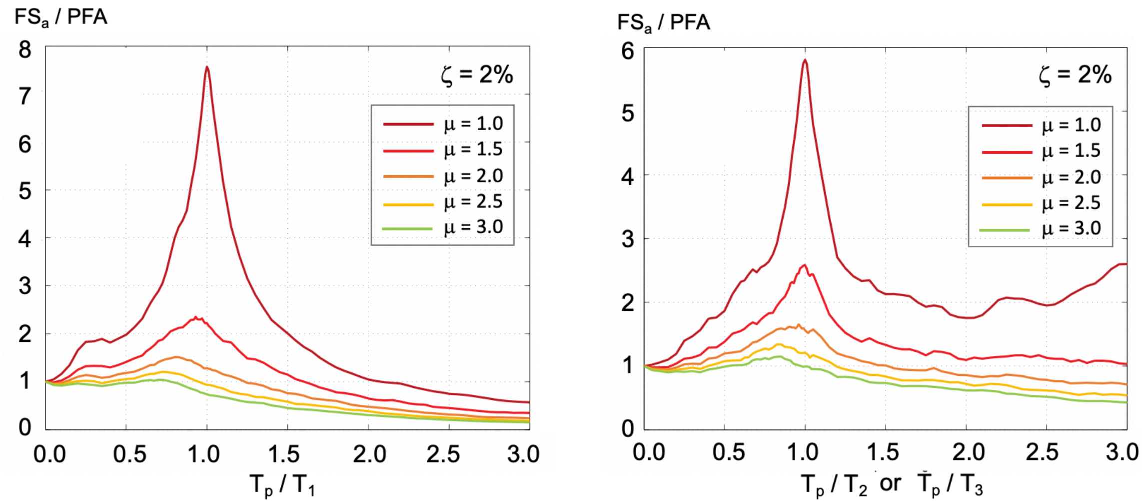

Figure 8. Statistical studies of the effect of level of nonlinearity on levels of amplification of accelerations on nonstructural elements.

Figure 8 shows binormalized floor spectra for different levels of inelastic deformation in the secondary system. The figure on the left depicts the case in which the period is normalized by the first mode of vibration of the supporting structure, whereas the figure on the right corresponds to spectra where the period of the component is normalized by the second or third mode of vibration of the supporting structure. It can be seen that nonlinearity incurs large reductions in horizontal accelerations and equivalent static forces but additionaly acceleration and force demands becomes much less sensitive to changes in the normalized period. This is an important advantage because often the period of vibration of the nonstructural element is not known or is subjected to important uncertainties. Figure 9 shows reductions in forces and in displacements for nonstructural components that are perfectly tuned to the first mode or to higher modes of the supporting structure strongly contributing to the response. It can be seen that even fairly small levels of nonlinearity, such as 1.5 or 2.0, lead to large reductions in forces. However, in addition to large reductions in forces, the proposed approach also leads to important reductions displacement demands. It should be noted that in Figure 9 the displacement demands are normalized with respect to those that would occur in elastic systems showing that they can be reduced to half by allowing a relatively small level of nonlinearity.

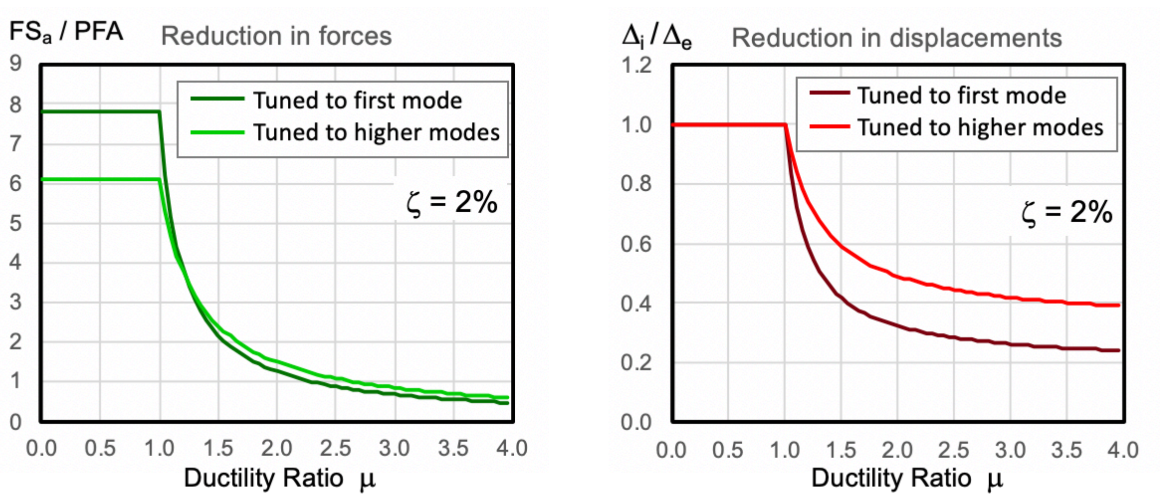

Figure 9. Reductions in forces (left) and in displacement (right) as a function of the level of nonlinearity allowed in nonstructural elements that are perfecty tuned to the first mode or to higher modes of the supporting structure.

4. SUMMARY AND CONCLUSIONS

The seismic design of nonstructural elements is challenging since, in general, there are large uncertainties in estimating force and deformation demands for nonstructural elements and their attachments to the structure in which they are mounted on or are suspended from. Current design provisions make use of oversimplified equations to estimate equivalent static forces that do not properly take into account the main factors controlling the intensity and other characteristics of seismic demands that may occur on nonstructural elements, and therefore, they may greatly overestimate demands leading to overly conservative designs, while in many other cases, they may greatly underestimate seismic demands leading to unconservative designs and nonsatisfactory seismic performance.

There is no doubt that it is possible to develop rational methods for design of nonstructural elements that adequately consider the characteristics of the ground motion, of the supporting structure (lateral strength, lateral stiffness and their spatial distribution in the structure, modal frequencies, damping, etc.) and of the nonstructural element (mass, stiffness, strength, modal frequencies, damping). However, nonstructural elements are typically not designed by structural engineers that are experts in seismic loading. Even, if structural engineers are asked to design these elements they mainly design their bracing and attachments to the structure. Furthermore, nonstructural elements are rarely included in the analytical model of the structure and, more importantly, very little information required to develop detailed models is typically available to the engineers in charge of designing bracing elements of nonstructural elements or their attachments to the structure and to the nonstructural component.

A series of analytical studies have been presented that provide the basis for a new design approach for nonstructural elements in which bracing elements are designed and detailed to yield in the case of moderate and strong earthquakes. The analytical studies allowed the selection of several motions recorded in instrumented buildings that provide severe excitation to nonstructural elements that are representative to those that are expected to occur in nonstructural elements on buildings during moderate and strong earthquake ground motions. The proposed approach is particularly effective for nonstructural elements whose frequencies of vibration coincides with modal frequencies of the structure in which they are mounted on or suspended from. This is true whether the nonstructural element is tuned or nearly tuned to the fundamental mode of vibration or to higher modes of the supporting structure.

The proposed design approach has a number of important advantages with respect to current seismic provisions for the design on nonstructural elements. These advantages are: (1) It can be used with limited information about the supporting structure; (2) It allows to design for significantly lower acceleration and forces; (3) It significantly reduces uncertainties on the seismic forces acting on the nonstructural components, the bracings and attachments, as these forces now depend on the strength of the yielding bracing element which can be estimated with much smaller uncertainty; (4) For components tuned or nearly tuned to modes of vibration of the supporting structure, the proposed approach, in addition to reducing force demands it also leads to important reductions in lateral deformation demands to levels significantly smaller than those that would occur in tuned components responding elastically; and (5) Force and deformation demands become far less sensitive to the ratio of period of vibration of the component to modal periods of vibration of the supporting structure.

ACKNOWLEDGEMENTS

The project was sponsored by the Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (SERA). This funding is greatly appreciated. Special thanks are also given to Profs Anastasios Sextos and George Mylonakis, Dr. Raffaele De Risi and laboratory personnel at the University of Bristol for their assistance during the shaking table testing. We also thank an anonymous reviewer of this manuscript who provided several sugguestions to improve it.

REFERENCES

Ayers JM, Sun TY, Brown FR. Non-structural damage to buildings, the great Alaska earthquake of 1964: Engineering. National Academy of Science, Washington, DC. 1973.

Elkady A., Vamvatsikos D., Lignos D., Kazantzi A and Miranda E., Experimental study to validate an improved approach to design acceleration-sensitive nonstructural component, Proc. Fifth Intl. Workshop on the Seismic Performance of Nonstructural Elements, Stanford, California, December, 2022.

Filiatrault A, Sullivan T. Performance-based seismic design of nonstructural building components: The next frontier of earthquake engineering. Earthquake Engineering and Engineering Vibration. 2014 Aug;13(1):17-46.

Kazantzi A, Vamvatsikos D, Miranda E. Effect of yielding on the seismic demands of nonstructural elements. In *Proceedings of 16th European conference on earthquake engineering*, Thessaloniki, Greece 2018.

Kazantzi AK, Vamvatsikos D, Miranda E. The effect of damping on floor spectral accelerations as inferred from instrumented buildings. *Bulletin of Earthquake Engineering*. 2020a Mar;18(5):2149-64.

Kazantzi AK, Vamvatsikos D, Miranda E. Evaluation of seismic acceleration demands on building nonstructural elements. *Journal of Structural Engineering*. 2020b Jul 1;146(7):04020118.

Kazantzi AK, Miranda E, Vamvatsikos D. Strength-reduction factors for the design of light nonstructural elements in buildings. *Earthquake Engineering & Structural Dynamics*. 2020c Oct 25;49(13):1329-43.

Miranda E. Seismic evaluation and upgrading of existing buildings. *PhD Thesis*, University of California, Berkeley; 1991.

Miranda E, Kazantzi AK, Vamvatsikos D. New approach to the design of acceleration-sensitive non-structural elements in buildings. *In Proc. 16th European conference on earthquake engineering* 2018a (pp. 18-21).

Miranda E, Kazantzi AK, Vamvatsikos D. Towards a new approach to design acceleration-sensitive non-structural components. *In Proceedings of 11th national conference in earthquake engineering*, Earthquake Engineering Research Institute, Los Angeles, CA 2018b.

Naeim F. Impact of the 1994 Northridge earthquake on the art and practice of structural engineering. The Structural Design of Tall and Special Buildings. 2004 Dec 15;13(5):373-89.

NIST 2018. Recommendations for Improved Seismic Performance of Nonstructural Components, NIST GCR 18-917-43, National Institute of Standards and Technology, Gaithersburg, Virginia; 2018.

Poulos, A. and Miranda, E. Probabilistic characterization of the directionality of horizontal earthquake response spectra *Earthquake Engineering & Structural Dynamics*. 2022.

Taghavi, S., Miranda, E., [2003] "Response assessment of nonstructural building elements," *Report PEER 2003/05*, Pacific Earthquake Engineering Research Center (PEER Center), University of California, Berkeley.

Taly N. The Whittier Narrows, California Earthquake of October 1, 1987—Performance of Buildings at California State University, Los Angeles. Earthquake Spectra. 1988 May;4(2):277-317.