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Comparative study of the seismic demand estimation on acceleration-sensitive nonstructural elements

Wilson Carofilis[1], **Bryan Chalarca**[2]

[1] *Research fellow*, University School for Advanced Studies IUSS Pavia, *wilson.carofilis@iusspavia.it*

[2] *Ph.D. candidate*, University School for Advanced Studies IUSS Pavia, *bryan.chalarca@iusspavia.it*

Abstract

The latest middle and high intensity seismic events have demonstrated that the potential seismic risk of nonstructural elements compromises a building's overall seismic performance, affecting especially the building operability. Additionally, nonstructural elements account for a considerably large fraction of the total earthquake economic losses and total building costs. Several international building codes provide methods to approximate the seismic acceleration demand on nonstructural elements; however, the provided guidelines may not accurately estimate the actual seismic demand leading to unconservative designs of acceleration-sensitive nonstructural elements. This study compares the acceleration demand on nonstructural elements calculated by using international building codes and state-of-the-art estimation methodologies with actual floor acceleration response spectra from nonlinear time history analysis. Two moment-resisting steel frames of three and nine stories were selected as case-study buildings. The FEMA P-695 far-field ground motion set was scaled to an equivalent design intensity and it was used as the input seismic load. The floor absolute accelerations were recorded on the first and last stories of both buildings and the median floor absolute acceleration response spectra were calculated. The spectral floor accelerations were determined considering a wide range of nonstructural periods and the results were compared with the estimated floor spectral acceleration obtained from the building code provisions and the novel methodologies. The results point out that current building codes tend to mislead the design of nonstructural elements by underestimating the actual acceleration demand on these components. On the other hand, the state-of-the-art methodologies provide a better estimation of the acceleration demand on nonstructural elements without the need for conducting nonlinear time history analyses.

Key words: nonstructural elements, floor acceleration, seismic performance, seismic demand, spectral acceleration

1 Introduction

The last decade seismic events have demonstrated that nonstructural elements (NSEs) are still largely affected by seismic excitations representing a significant portion of the total economic losses and thus altering the overall seismic performance of a building [1, 2]. Most current building codes present a rough procedure to calculate the seismic demand on acceleration-sensitive NSEs. For instance, the ASCE 7-16 [3] and the Eurocode 8 [4] include a section dedicated to estimate the design force for NSEs, however, several studies have demonstrated that these approaches can miscalculate the seismic demand leading to non-conservative designs of NSEs [5-8]. Several authors have proposed methodologies to predict the floor acceleration spectra based on the structural dynamic properties without carrying out extensive and time consuming nonlinear time-history analyses [9-10]. These methods are better suitable to approximate the seismic demand on a large variety of NSEs, ensuring more accurate designs and an improvement of the overall building seismic performance. To assess the different methodologies for the estimation of the acceleration seismic demand on NSEs, two moment-resisting steel frames of three and nine stories were modeled and subjected to the 44 records of the FEMA P-695 far-field ground motion set [11]. The absolute acceleration floor response spectra were calculated at the first and top floor of each building, and the results were compared with the floor acceleration estimations obtained from the ASCE 7-16 [3] and Eurocode 8 [4] building codes, and the methodologies proposed by Vukobratović and Fajfar [9] and Merino *et al.* [10]. Four nonstructural periods (*i.e.,* 0.0, 0.5, 1.0 and 2.0 seconds) were used as reference points to cover a large range of NSE periods.

2 Case study buildings and seismic input

The results determined through building code procedures and simplified methodologies were compared and evaluated with respect to nonlinear time-history analyses, which were carried out in two moment-resisting steel frames of three and nine stories. The 44 records of the FEMA P-695 far-field ground motion set [11] were used. The records were scaled to an equivalent design intensity for the city of Los Angeles (US). The buildings were modeled by using the software OpenSees V3.2.2 [12], further details of the modeling assumptions can be found in [13].

2.1 Case-study buildings

Two 2-D moment-resisting steel frame buildings of three and nine stories were selected from the SAC Steel project [14] as case study buildings. These buildings are characterized by brittle beam-column joint connections typical of the pre-Northridge seismic designs. The three-story building was adapted from the FEMA 440 [15], the structure is composed of three bays in the north-south direction (selected for this study) and six bays in the east-west direction, the fundamental period is equal to 0.79 s. Fig. 1 illustrates the configuration of the three-story case study building. The nine-story building was adapted from the FEMA 440 [15]. The structure is composed of five bays in both directions, the fundamental period is equal to 1.89 s. Fig. 2 illustrates the configuration of the nine-story case study building. Further details about the case study buildings are reported in [13].

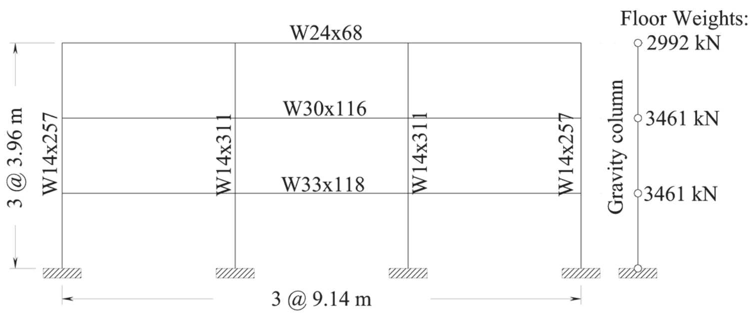

Figure 1. Three-story case study building

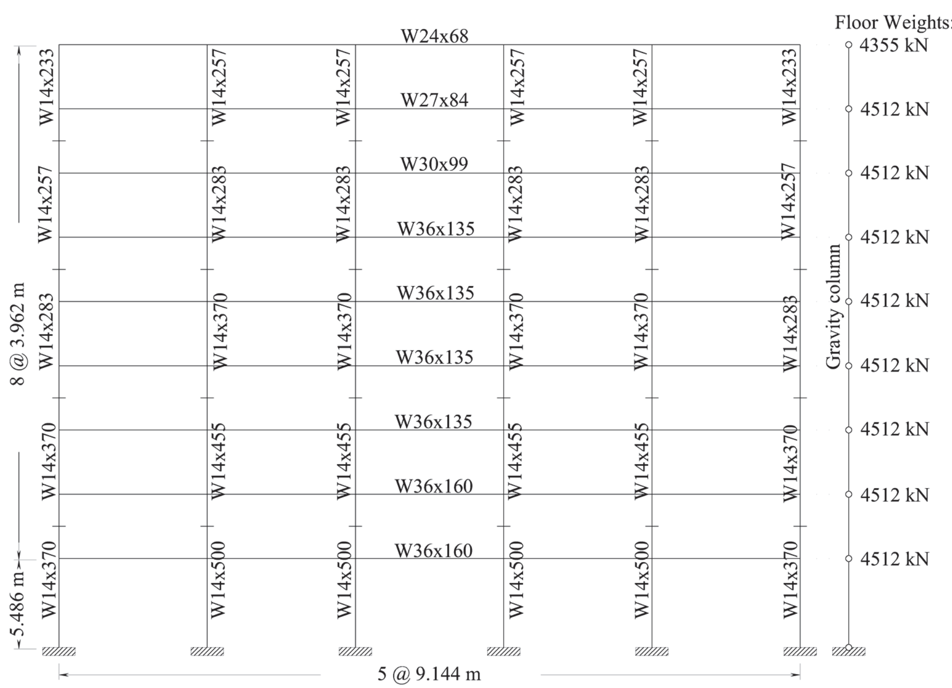

Figure 2. Nine-story case study building

2.2 FEMA P-695 far-field ground motion set

The FEMA P-695 far-field ground motion set [11] is composed of 22 pairs of historical horizontal ground motions (44 individual records) that represent the seismicity of the western United States. The records were scaled based on the median spectral acceleration at a period of one second matching the ASCE 7-16 [3] design spectrum for the city of Los Angeles (US) with a soil type D_{max}. Fig. 3 shows the acceleration spectra of the FEMA P-695 far-field ground motion set, and the target design spectrum used to scale the records.

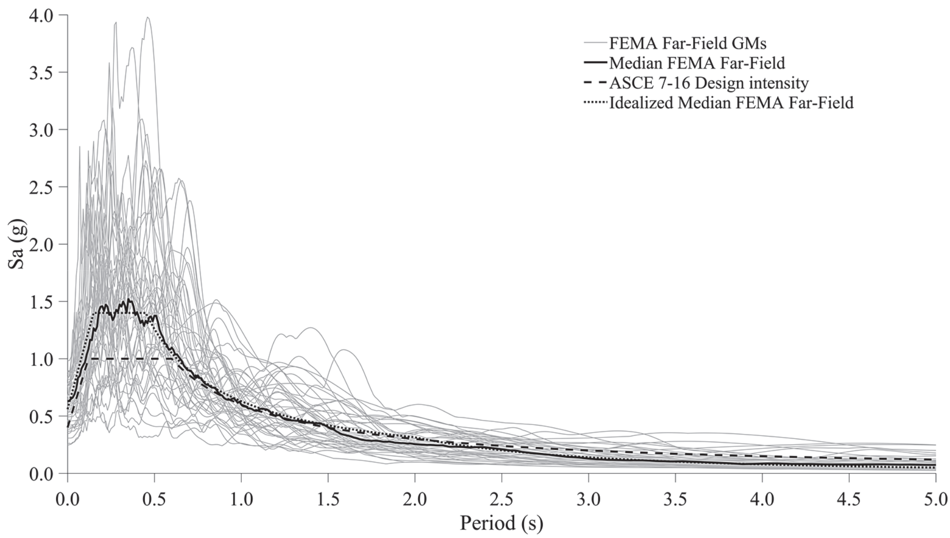

Figure 3. 5 % damping FEMA P-695 far-field and ASCE 7-16 design acceleration spectra

3 Estimation of acceleration demand

The estimation of the acceleration demand for the case study buildings was carried out through three methodologies. The first one involves two international building codes, the ASCE 7-16 [3] and the Eurocode 8 [4], which are commonly used as a reference for the seismic design of NSEs, especially in countries without official requirements. The second approach includes the studies of Vukobratović and Fajfar [9] and Merino *et al.* [10], who proposed state-of-the-art methods to estimate floor acceleration spectra. The last methodology calculates the floor response spectra from floor acceleration time-histories obtained after carrying out nonlinear time-history (NLTH) analyses.

3.1 International building codes

Generally, building codes address the seismic design of acceleration-sensitive NSEs by estimating the design force to be applied to the component. The ASCE 7-16 [3] in its chapter 13 describes how to calculate the seismic design force. However, it is not necessary to compute the component weight, hence, the calculations can be expressed in terms of spectral acceleration. The estimated spectral acceleration depends mainly on the site conditions (*i.e.,* maximum spectral acceleration), and on the relative location of the NSE regarding the total height of the building. Additionally, chapter C13 [3] proposes an amplification factor according to the period of vibration of the NSE and the supporting structure. Likewise, ASCE 7-16 [3] limits the floor spectral acceleration to an upper and lower bound.

Eurocode 8 [4] specifies a procedure to estimate the seismic design force. Yet, similarly to the ASCE method, the NSE weight can be omitted to obtain the equivalent spectral acceleration. The method takes into account the seismicity of the site (*i.e.,* peak ground acceleration) and the relative location of the NSE with respect to the total height of the building. However, the formulation directly integrates the fundamental vibration period of the NSE and the supporting structure. In this way, the absolute acceleration floor response spectra are obtained. In addition, the spectral acceleration is conditioned to be at least equal to the peak ground acceleration. It is noteworthy that although both building codes use the seismic input and the relative position of the NSEs, they do not consider either the effects of higher modes or the inelastic behavior of the supporting structure.

3.2 Current approximate methodologies

Simplified methodologies can better estimate the floor spectral acceleration demand by integrating not only higher mode effects but also the nonlinear behavior of the supporting structure. The procedures proposed by Vukobratović and Fajfar [9] and Merino *et al.* [10] are based on four factors that have a significant impact on the floor response spectrum [10]: 1. The dynamic interaction between the NSE and the supporting structure, in which the dynamic properties of both systems can increase or decrease the seismic acceleration demand. 2. The influence of the nonstructural damping that highly affects the acceleration demand on NSEs. 3. The influence of the nonlinear response of the supporting structure, and 4. The response of the NSE that can also present an inelastic response. Due to length limitations, both procedures are not explained herein, the authors highly recommend revising the original publications for a complete description and better comprehension of the methodologies.

3.3 Nonlinear time-history analysis

The case study buildings were subjected to NLTH analysis using the 44 records of the FEMA P-695 far-field ground motion set [11]. The absolute horizontal floor acceleration time-histories were recorded and the 5 % damping elastic absolute acceleration floor response spectra (AAFRS) were calculated for the first and top floors. Finally, the median spectrum was obtained to be used as a reference value for the previously described methodologies.

4 Analysis results

Fig. 4 illustrates the AAFRS for both case-study buildings at the first and top floors, calculated with the procedures detailed in section 3. In Table 1 is listed the comparison of the spectral accelerations for the selected four nonstructural periods normalized with respect to the NLTH results. The ASCE 7-16 and the Eurocode 8 procedures overestimate the spectral acceleration on the first floor, especially for periods higher than the fundamental period of vibration of the supporting structure (*i.e.*, 0.79 s and 1.89 s for the three-story and nine-story buildings, respectively). A better approximation is reported for the fundamental period of the three-story building on the top floor, in which the ASCE 7-16 provides a closer floor spectral acceleration to that of the NLTH analysis. To determine the AAFRS through the ASCE 7-16 procedure, it was necessary to assume a component response modification factor of 2.5, considered as a common factor for most of NSEs [3], since the ASCE 7-16 contemplates only inelastic spectral accelerations for the design of NSEs. Indeed, the limit imposed by ASCE 7-16 represents the maximum inelastic spectral acceleration, which is independent of the component response modification factor. Furthermore, Eurocode 8 provides closer values of AAFRS with respect to the NLTH results. As observed in Table 1, Eurocode 8 produces almost the same peak floor acceleration for the first floor in both case-study buildings. However, Eurocode 8 exaggerates the demand for very flexible NSEs and underestimates it for the case of rigid components. Additionally, the minimum floor spectral acceleration (*i.e.,* peak ground acceleration) imposed by the code overestimates the demand for NSEs with large periods. Although the code procedures can be easily implemented, the results show large divergences when compared to the NLTH. Consequently, the code results can mislead the seismic design of the NSEs.

The simplified methodologies (*i.e.,* Vukobratović and Fajfar [9] and Merino *et al.* [10]) are better suitable to approximate the shape of AAFRS obtained from the NLTH analysis. However, both methodologies tend to overestimate the spectral floor acceleration at the periods associated with the fundamental and second mode of vibration of the supporting structure and to underestimate the demand at the following higher modes. The last trend is especially evident for the methodology proposed by Vukobratović and Fajfar [9]. One of the main differences between both simplified methodologies is the fact that Vukobratović and Fajfar [9] generates large plateaus at the spectral peaks simulating the period elongation presented by the inelastic behavior of the supporting structure. Although this characteristic is desirable, it is not always representative of reality, particularly for steel structures [10]. In comparison, the procedure proposed by Merino *et al.* [10] allows the modification of the peaks plateaus to get a spectral shape closer to the expected response. Both methods tend to underestimate the peak floor acceleration at the first floor and show a good approximation at the top floor. In general, the methodology proposed by Merino *et al.* [10] is more conservative showing larger spectral accelerations than Vukobratović and Fajfar's [9] method. Even though the application of the described methodologies is not as simple and straightforward as implementing the code procedures, the accuracy of the results supports the extra steps to have a better estimation of the seismic demand.

Table 1. Normalized Spectral accelerations with respect to NLTH results

Nonstructural period [s]	**ASCE 7-16**	**Eurocode 8**	**Vukobratović and Fajfar**	**Merino *et al.***
Three-story first floor				
0.0	1.52	0.96	0.85	0.86
0.5	2.86	1.81	1.12	1.61
1.0	3.30	2.24	1.44	1.14
2.0	4.14	1.73	1.33	1.17
Three-story top floor				
0.0	2.51	1.46	1.02	0.97
0.5	1.47	1.47	1.05	1.17
1.0	1.31	1.40	1.62	1.04
2.0	5.41	1.63	1.74	0.85
Nine-story first floor				
0.0	1.34	0.88	0.61	0.72
0.5	0.77	0.72	0.88	1.11
1.0	1.99	2.04	0.69	1.18
2.0	8.74	5.52	1.14	1.27
Nine-story top floor				
0.0	2.67	1.55	0.96	0.93
0.5	1.25	0.98	1.08	1.36
1.0	2.21	2.04	1.04	0.94
2.0	1.97	2.26	1.17	1.14

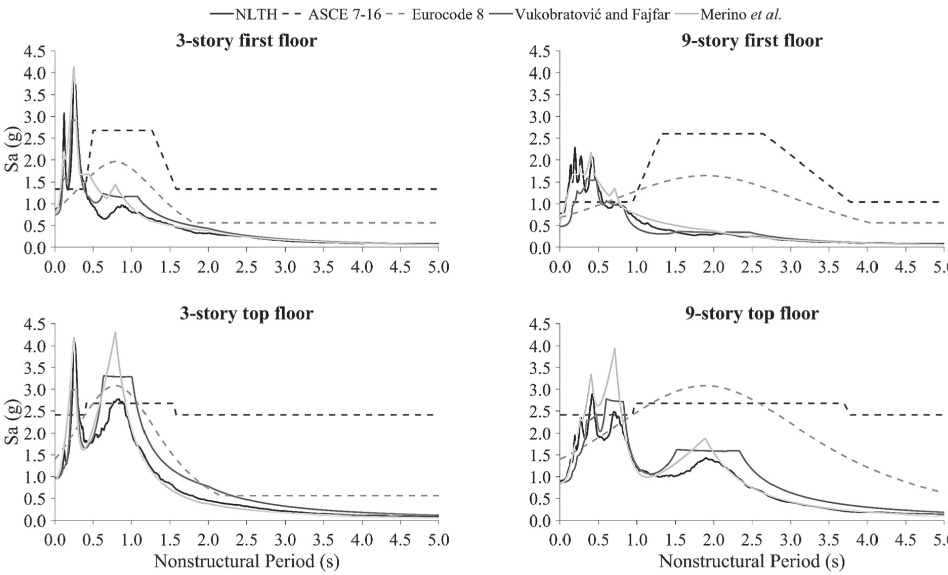

Figure 4. Comparison of the different 5 % AAFRS for both case-study buildings

5 Conclusions

The results obtained from the nonlinear time-history analysis show that the absolute acceleration floor response spectra are remarkably affected by higher modes, the first and top floor shows larger spectral accelerations at shorter periods, this trend can be explained by the inelastic response of the buildings which significantly influences the acceleration demand at the fundamental mode of vibration. The peak spectral accelerations are located at the higher modes of vibrations (up to 4.3 g) and the acceleration demand decreases rapidly after the fundamental period of the structure. The results obtained from the building code procedures show that the absolute acceleration floor response spectra are controlled by the fundamental mode, especially the one developed from the Eurocode 8 [4]. The approaches presented by the building codes (ASCE 7-16 [3] and Eurocode 8 [4]) account for neither higher modes effects nor inelastic behavior of the supporting structure. In general, the acceleration demand is highly overestimated from the fundamental to longer periods and underestimated at shorter periods. The ASCE 7-16 [3] and Eurocode 8 [4] provide a decent estimate of the floor spectral acceleration in the last floor of the three-story building when the nonstructural period and the supporting structure period are similar. Additionally, Eurocode 8 [4] shows a good prediction of the peak floor acceleration on the first floor of both case study buildings, especially for the three-story building.

The method proposed by Vukobratović and Fajfar [9] matches closely the spectral shape of the nonlinear time-history analysis results, coinciding with most of the peak positions. The method tends to be conservative at the fundamental and longer periods and slightly underestimated the spectral acceleration at shorter periods. On the other hand, the method of Merino *et al.* [10] shows a more conservative absolute acceleration floor response spectrum. The spectral shape matches fine the nonlinear time-history analysis results showing slightly larger peak spectral accelerations. Similar to Vukobratović and Fajfar [9], the spectral acceleration is slightly underestimated at very short nonstructural periods. Despite this, the peak floor acceleration is accurately predicted. It is noteworthy that the simplified methodologies show a better approximation of the absolute acceleration floor response spectra. Indeed, for some periods the simplified methodologies provided a perfect match with respect to the results from the nonlinear time-history analysis. Nevertheless, the approach of Merino *et al*. [10] shows a better estimation of the acceleration demand compared to Vukobratović and Fajfar [9]. In view of the outcomes, the simplified methodologies such as the ones proposed by Vukobratović and Fajfar [9] and Merino *et al.* [10] should be adopted to address the seismic design of acceleration-sensitive nonstructural elements since they produce closer estimations of the floor spectral accelerations when compared to the results from nonlinear time history analyses. Therefore, reliable floor spectral acceleration can be determined without the need for exhaustive calculations and detailed models required in such type of analysis.

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