SEISMIC DEMAND ON NON-STRUCTURAL ELEMENTS:
INFLUENCE OF MASONRY INFILLS ON FLOOR RESPONSE SPECTRA

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ABSTRACT

The seismic performance of non-structural elements is nowadays recognized to be a key issue in the seismic design as well as in the earthquake related losses estimation. The evaluation of the seismic demand on non-structural elements is of paramount importance for seismic design. For this reason, many modern building codes provide simplified relationships in order to define design inertia forces on acceleration sensitive non-structural elements. Researchers demonstrated that the formulations proposed by building codes are often not able to predict realistic accelerations on non-structural elements. In the last few years several design oriented simplified methodologies have been developed to define the floor response spectra in buildings of different typologies, including reinforced concrete (RC) frame buildings, considering the potential nonlinear behaviour of the structures. Despite these significant efforts, some issues are still open and require further investigations. Current methodologies generally neglect the influence of masonry infills in the evaluation of floor response spectra. In this paper, the influence of masonry infills on floor response spectra for masonry infilled RC buildings subjected to frequent (serviceability level) earthquakes is investigated. The study focuses on typical European RC buildings. The results demonstrate that masonry infills, in particular if the structures behave elastically, cannot be neglected in the evaluation of floor response spectra.

Keywords: Floor Response Spectra; Masonry Infills; Non-structural Elements; Seismic Demand

1. INTRODUCTION

A significant part of the observed earthquake related losses in recent earthquakes worldwide has been attributed to the damage to non-structural elements. The damage to non-structural elements could significantly affect the immediate functionality of buildings because they generally exhibit damage at low seismic intensity with respect to the structural elements. The damage observed following the 2010 Chile earthquake demonstrated the importance of non-structural damage on the post-earthquake functionality of critical facilities. Following this earthquake, the Santiago International Airport was closed for several days because of the damage to the pressurized fire suppression sprinkler piping systems interacting with ceiling systems, while the structure did not suffer significant damage (Miranda et al. 2014). During the same earthquake, four hospitals completely lost their functionality and over 10 more lost at least 75% of their functionality due to damage to sprinkler piping systems (Miranda et al. 2014). During the 2009 L’Aquila earthquake in Italy, one of the most common non-structural element failures was related to partition walls experiencing large in-plane inter-storey drifts (Salvatore et al. 2009, Ricci et al. 2011). Significant damage to non-structural elements has been also observed during the 2012 Emilia earthquake in Italy. In this seismic event, industrial facilities reported large economical losses often related to the failure of rack systems (Ercolino et al. 2012). Miranda and Taghavi (2003)

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reported that non-structural elements represent most of the total investments in typical buildings. In hospital buildings, for example, the monetary investments in terms of non-structural elements are approximately 92% of the total building cost. In the light of these considerations, the seismic performance of non-structural elements is nowadays recognized to be a key issue in performance-based earthquake engineering (PBEE) in order to ensure a desired structural system performance for a given seismic intensity (FEMA 2012). In the loss estimation framework, the evaluation of the total losses, including those to non-structural elements, involves four main steps: 1) hazard and facility definition, 2) structural analysis, 3) damage analysis and 4) loss analysis. In terms of non-structural elements, the first step consists in the definition of a database in which all non-structural elements are classified, in this context the use of Building Information Modelling (BIM) could be very helpful (Perrone and Filiatrault 2017). The second step is of paramount importance both for structural and non-structural elements. In this phase of the procedure, the seismic demand on non-structural elements is evaluated. A structural model of the building is subjected to seismic excitations of various intensities in order to evaluate the maximum response in terms of displacements, forces and accelerations. During the third stage of the loss estimation procedure, the probability that a certain element will suffer damage for a given seismic intensity is established. Finally, the last stage of the procedure includes the computation of decision variables such as monetary loss due to repair costs, loss of use or the likelihood of injuries and/or fatalities. The accurate evaluation of floor acceleration response spectra (FRS) in buildings is of paramount importance for estimating the vulnerability and reduce the probability of failure of non-structural elements. Lin and Mahin (1985) and Sewell et al. (1988) carried out two of the first research studies on the evaluation of FRS. Medina et al. (2006) studied peak floor accelerations and FRS for light non-structural elements mounted in regular moment-resisting frames. Sankaranarayanan and Medina (2007) studied the main factors that caused amplification or reduction in FRS. Sankaranarayanan and Medina (2007) identified the following main parameters that could affect the FRS: the location of the non-structural element in the building, the ratio between the period of the non-structural element and of the building, the damping ratio of the non-structural element and the level of inelastic response exhibited by the supporting structure. Even if detailed methods are nowadays available in order to estimate accurately the seismic demand on both acceleration and displacement sensitive non-structural elements, it is often required to use simplified methodologies in design situations. Some code prescriptions provide simplified relationship to evaluate the seismic demand on non-structural elements (ASCE 2010, CEN 2004, NZS 2004). The research studies carried out in the last years emphasized the inaccuracy of the formulations proposed by the codes. For this reason, some authors proposed more accurate methodologies to predict FRS for single-degree-of-freedom (SDOF) and multi-degree-of-freedom (MDOF) systems, both for elastic and inelastic structures. Miranda and Taghavi (2003) proposed an approximate method to estimate floor acceleration demands in multi-story buildings responding elastically or quasi-elastically when subjected to earthquake ground motions (2005). More recently, Petrone et al. (2015) and Vukobratovic and Fajfar (2016) proposed two different methods to evaluate FRS for MDOF structures. The method proposed by Vukobratovic and Fajfar (2016) was calibrated both for elastic and inelastic primary structures. Similar considerations have been introduced in the methodology developed recently by Sullivan et al. (2013). This method was originally proposed for linear and nonlinear SDOF structures and was recently extended to linear elastic MDOF systems (Calvi and Sullivan 2014). The influence of masonry infills, which are often present in these structures, was generally neglected in the calibration of these simplified methodologies to predict the FRS. Despite this, many studies discussed the influence of masonry infills on the dynamic properties of RC reinforced concrete (RC) moment-resisting frames, in particular if the structures behave elastically (Ricci et al. 2011, Perrone et al. 2016). Lucchini et al. (2014) reported that masonry infills could significantly modify the profile of peak floor accelerations along the height of the building and the FRS both in linear and non-linear stage. In this paper, a numerical study is carried out to evaluate the seismic demand on acceleration-sensitive non-structural elements for frequent (serviceability level) earthquakes in RC frames.

2. CASE STUDY BUILDINGS

The influence of masonry infills on floor response spectra (FRS) of reinforced concrete (RC) moment-
resisting frames was studied for 10 case study buildings from the database developed by Crowley and Pinho (2004). Crowley and Pinho (2004) selected several buildings representative of the European context in order to analyze their seismic performance. The selected case study buildings were designed between 1930 and 1980 in five seismic prone European countries: Greece, Italy, Portugal, Romania and Bosnia (formally Yugoslavia). RC buildings designed for gravity loads were selected because of the higher influence of the masonry infills on the structural response. Due to the reduced dimensions of the columns, the influence of the masonry panel stiffness on the global seismic behaviour is more important. Table 1 summarizes the main characteristics of the case study buildings taken into account in this study. The reader could consult the references provided in the table for more detailed information on the case study buildings.

Table 1. Summary of case study buildings.

<table>
<thead>
<tr>
<th>No.</th>
<th>Country</th>
<th>Era</th>
<th>Height [m]</th>
<th>Main Geometrical Characteristics</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Italy</td>
<td>1970's</td>
<td>9</td>
<td>3 storeys, 3 bays</td>
<td>Moratti 2000</td>
</tr>
<tr>
<td>2</td>
<td>Italy</td>
<td>1960's</td>
<td>6</td>
<td>2 storeys, 5 bays</td>
<td>FIB 2003</td>
</tr>
<tr>
<td>3</td>
<td>Portugal</td>
<td>1960's</td>
<td>11</td>
<td>4 storeys, 3 bays</td>
<td>Carvalho et al. 1999</td>
</tr>
<tr>
<td>4</td>
<td>Romania</td>
<td>1930's</td>
<td>15</td>
<td>5 storeys, 5, bays</td>
<td>Bosteranu et al. 2003</td>
</tr>
<tr>
<td>5</td>
<td>Greece</td>
<td>1950's</td>
<td>15</td>
<td>5 storeys, 6 bays</td>
<td>Zeris et al. 2002</td>
</tr>
<tr>
<td>6</td>
<td>Greece</td>
<td>1950's</td>
<td>17</td>
<td>6 storeys, 6 bays</td>
<td>Zeris et al. 2002</td>
</tr>
<tr>
<td>7</td>
<td>Italy</td>
<td>1970's</td>
<td>18</td>
<td>6 storeys, 3 bays</td>
<td>Moratti 2000</td>
</tr>
<tr>
<td>8</td>
<td>Romania</td>
<td>1930's</td>
<td>18</td>
<td>6 storeys, 6 bays</td>
<td>Bosteranu et al. 2003</td>
</tr>
<tr>
<td>9</td>
<td>Bosnia</td>
<td>1980's</td>
<td>22</td>
<td>7 storeys, 2 bays</td>
<td>ACI 1984</td>
</tr>
<tr>
<td>10</td>
<td>Italy</td>
<td>1970's</td>
<td>24</td>
<td>8 storeys, 3 bays</td>
<td>Moratti 2000</td>
</tr>
</tbody>
</table>

In order to be representative of the European building scenario, the height of the selected case study buildings varies between 6 m and 24 m. Based on the information available in the reference documents, the mechanical properties of the materials vary significantly. In particular, the unconfined concrete compressive strength ranges from 15 to 29 MPa, while the yield strength of the steel reinforcement typically varies from 200 to 380 MPa. Because the mechanical properties of the masonry infill were not provided in the reference studies, the authors refer to the study proposed by Bal et al. (2007) in which average mechanical properties of the masonry infills typically used in Europe is suggested. The Young Modulus of the masonry panels was assumed equal to 1200 MPa, while the shear strength was assumed equal to 0.2 MPa, these values were kept constant for all buildings.

2.1 Modelling

The two-dimensional numerical models of the case study buildings were developed with the OpenSees software (Mazzoni et al. 2006) assuming a lumped plasticity approach. The primary elements were modelled by elastic beam column elements with two plastic rotational hinges at both ends, as illustrated in Figure 1. The Pinching4 Uniaxial material available in OpenSees was used to simulate the hysteretic behavior of the plastic hinges, the yielding ($\gamma_Y$) and the ultimate ($\gamma_{MAX}$) rotation capacity were evaluated according to Eurocode 8 (CEN 2004). According to the reference studies, the floor slab was assumed rigid for all case study buildings.
The masonry panels were modelled by an equivalent diagonal axial strut approach. The width of each strut was calculated according to the relationship proposed by Mainstone (1971). The force-displacement relationship of each equivalent diagonal strut is based on the model proposed by Panagiotakos and Fardis (1996), which is meant to describe the failure mode of a masonry panel in shear.

2.2 Earthquake ground motions

In order to represent the seismic hazard of a medium-high region in the European context a preliminary analysis was carried out. Based on the results of this analysis a site close to the city of Cassino, in Italy, was selected to perform the ground motion selection. This site is characterized by a design peak ground acceleration on stiff soil equal to 0.21g for a 10% probability of exceedance in 50 years. The ground motion selection was carried out assuming the prescription provided by the Italian code in order to define the target spectrum (NTC 2008).

Figure 2. Mean response spectrum and acceleration response spectra of all considered ground motions for a conditional period equals to a) 0.2 sec, b) 0.5 sec and c) 1.0 sec.
A set of 20 ground motions were selected from the PEER NGA-West database (PEER) during a previous study carried out at the Eucentre Foundation (Iervolino 2016). Hazard-consistent record selection was based on spectral compatibility (matching of the geometric mean) with a conditional mean spectrum according to the methodology proposed by Jayaram et al. (2011). This approach considers the conditional variance given a return period of spectral acceleration at the vibration periods of interest selected (0.2, 0.5 and 1.0 sec). Figure 2 shows the mean response spectrum and all response spectra for the 20 considered ground motions for conditional periods equal to 0.2, 0.5 and 1.0 sec.

The ground motions were generated for different return periods. In this study, the ground motions selected for a return period equal to 95 years were used to be representative of the serviceability limit state.

Table 2 reports the elastic fundamental period computed during the eigenvalue analysis for all case study buildings with and without the influence of the masonry infills. The masonry panels significantly affect the dynamic behaviour of the analyzed frames. For the case study buildings, the reduction of the fundamental period due to the infill effects varies between 32% and 66%. The results of the eigenvalue analysis were also used for selecting the ground motion conditional period for each building, as indicated in the table.

### Table 2. Computed elastic fundamental period for bare and infilled frames [sec].

<table>
<thead>
<tr>
<th>Frame Configuration</th>
<th>Period (sec)</th>
<th>Building No.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare Frame</td>
<td></td>
<td></td>
<td>0.64</td>
<td>0.44</td>
<td>0.69</td>
<td>0.57</td>
<td>0.95</td>
<td>0.92</td>
<td>0.97</td>
<td>0.74</td>
<td>1.96</td>
<td>1.16</td>
</tr>
<tr>
<td></td>
<td>Period</td>
<td></td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Conditional period for ground motions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Infilled Frame</td>
<td></td>
<td></td>
<td>0.25</td>
<td>0.22</td>
<td>0.41</td>
<td>0.39</td>
<td>0.36</td>
<td>0.37</td>
<td>0.45</td>
<td>0.51</td>
<td>0.98</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td>Period</td>
<td></td>
<td>0.20</td>
<td>0.20</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>0.50</td>
<td>1.00</td>
<td>0.50</td>
<td>0.50</td>
<td></td>
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<tr>
<td></td>
<td>Conditional period for ground motions</td>
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</tr>
</tbody>
</table>

3. ANALYSIS RESULTS

In this section, the results of the non-linear time-history (NLTH) analyses on the case study buildings are presented and discussed in terms of peak floor acceleration (PFA) and floor response spectra (FRS).

3.1 Peak floor accelerations

The ratio of median peak floor acceleration to peak ground acceleration (PFA/PGA) as a function of the relative height ratio (z/H) for the bare and infilled frames of each case study building is compared in Figure 3. Also shown in the figure are PFA amplification formula in three different international building codes (Eurocode 8, ASCE7 and New Zealand Standard). The PFA/PGA distributions along the building’s height are almost similar for all case study buildings. The acceleration amplifications tend to increase linearly and assume the highest values at the top floor. A linear distribution of floor accelerations with building’s height indicates a strong influence of the fundamental period. A more complex profile of PFA/PGA can be attributed to higher mode effects (e.g. Figure 3 bare frames for Buildings No. 5 and 6). Observing the behaviour of Building No. 9 (Figure 3i) is interesting. For this case, the median peak floor accelerations for the bare case are lower than the peak ground accelerations (PFA/PGA < 1). This behaviour is directly related to the nonlinear response of the frame. The introduction of the infill panels increases the global strengths of the structures and reduces the nonlinear response of their primary structures. As a result, the median PFA/PGA ratios are larger than unity but, at the same time, the acceleration profiles are nonlinear with height, indicating a stronger influence of higher modes. The influence of higher modes for Building No. 9 on the MFRS was also observed in Figure 3i. Comparing the median PFA/PGA ratios for bare and infilled frames, it can be clearly observed that the presence of masonry infill causes significant increases in peak floor accelerations compared to that of bare frames. For all case study buildings, with the exception of frame No. 9, the PFA distribution is more regular in the infilled frames because the participating masses associated to the fundamental period are higher than that of the bare frames. The uniform distribution of the infill panels along the height of the frames tends to reduce the vertical stiffness variation.
The median PFA/PGA ratios for the bare frames are generally overestimated by the code acceleration amplification formulas. Different considerations can by pointed out for the masonry infilled RC frames, in which amplification higher than 3.0 are often observed (with the exception of frames No. 9 and No. 10). The amplification factors proposed by the codes are limited to 2.5 for the Eurocode 8 and to 3.0 for the ASCE7 and New Zealand Standard. For this reason, the acceleration amplifications are often underestimated if the structures behave elastically.

The results presented in this section clearly demonstrates that the influence of the masonry infills cannot be neglected in the seismic design/assessment of non-structural elements.
3.2 Floor response spectra

The acceleration time-histories were recorded during the non-linear time history analysis to evaluate, though numerical techniques (Chopra 2001), the floor response spectra (FRS) for both bare and infilled RC frames of the case study buildings. The 5% damped median FRS (MFRS) taken across the ground motion ensemble was evaluated for each floor of the case study buildings. Figure 4 reports the MFRS for bare (solid lines) and infilled (dashed lines) frames. Three MFRS are reported for each frame, these MFRS are representative of the lower floors, mid-height and top floor of the case study buildings. The peak median spectral accelerations are generally recorded for periods close to the fundamental period of the structures due to the filtering action of the primary structures.

Looking at the MFRS is also possible to evaluate the median response in terms of elastic or inelastic response of the frames across the ensemble of ground motions (Figure 4). In taller frames, as well as in the frames responding in the inelastic range, the influence of the higher modes was often observed (Buildings No. 7, 9 and 10). The contribution of higher modes is more evident at the lower floors in which the median peak spectral accelerations at shorter periods are lower than that at longer periods (Buildings No. 5, 6, 7, 8, 9 and 10). This result can be attributed to the larger displacements associated with the modal shapes of the higher modes at the lower levels. Even if only a serviceability-level seismic intensity is considered in this study, some of the case study buildings experienced an inelastic response due to their lateral strength deficiencies. If the structural response is essentially elastic, the MFRS are generally characterized by large and narrow spectral peaks corresponding to the elastic natural periods of the structure (Sullivan et al. 2013). If the supporting structures respond in the inelastic range, the spectral peaks are wider because of period elongations.

The results reported in Figure 4 emphasized two main differences in the MFRS for the bare and infilled frames due to the influence of the masonry panels: 1) the peak spectral accelerations are shifted to shorter periods corresponding to the natural periods of the buildings (see Table 2) and, 2) the peak spectral accelerations are significantly amplified in infilled frames. These two effects were observed for all floors of each case study building. The presence of the masonry infills significantly affects also the shape of
the MFRS. The MFRS with masonry infills are generally characterized by a well-defined single peak that lies at a period close to the fundamental period of the structure with steep drops off for other periods. This behavior can be associated to the stiffness and strength increase caused by the incorporation of the infilled frames.

Note also that the MFRS were obtained for 5% of critical damping. As reported by Sullivan et al. (2013), the acceleration amplification can be significantly affected by the damping ratio.
4. CONSIDERATIONS ON SIMPLIFIED APPROACH TO DEFINE FLOOR RESPONSE SPECTRA

As pointed out in the Introduction, significant efforts have been made in recent years to develop simple methodologies able to estimate, with reasonable accuracy, floor response spectra (FRS) in reinforced concrete (RC). The more advanced methodologies available in the literature generally account for three main parameters to predict the FRS in RC structures:

1. Influence of dynamic filtering offered by the vibration modes of the supporting structure.
2. Influence of damping characteristics of the non-structural elements.
3. Influence of inelastic response of the supporting structure.

In order to provide an example of the effectiveness of the methodologies available in the literature to predict FRS for bare and masonry infilled RC frames, the Sullivan et al. (2013) methodology (modified by Calvi et al. (2014) for MDOF structures) was applied to all floors of the case study buildings. Figures 5 and 6 report the comparison between the 5% damped median roof FRS obtained from the NTLH analysis (solid lines) and the FRS predicted by the Sullivan et al. methodology (dashed lines) for case study Buildings No. 2 and No. 4 (see Table 1). Similar comparisons were observed for all considered case study buildings. The Sullivan et al. (2013) methodology provides an effective means of predicting median FRS for bare frames as well as the peak spectral accelerations and the general shape of the spectra are well predicted. As expected, if the masonry infills are not considered in the numerical models used to evaluate the elastic periods and the modal shapes necessary to apply the Sullivan et al. methodology, the FRS for masonry infilled RC frames are not well predicted (Figure 6b and 7b). The two main issues are related to the value of the peak spectral accelerations and the period at which these accelerations arise. Including in the numerical models the effects of the masonry infills, the periods at which the
maximum spectral accelerations arise are well predicted by the Sullivan et al. methodology, but the peak spectral accelerations are still poorly predicted (Figure 6b and 7b).

Figure 6. Comparison of the 5% damped median FRS at the top floor obtained from the NLTH analysis and FRS predicted by the Sullivan et al. methodology for case study Building No. 2

Figure 7. Comparison of the 5% damped median FRS at the top floor obtained from the NLTH analysis and FRS predicted by Sullivan et al. methodology for case study Building No. 4.

Based on these considerations, even if the masonry infills are considered in the numerical models one correction factor should still be defined to predict the FRS of masonry infilled RC frames by means of the Sullivan et al. methodology (2013). This parameter should be introduced to amplify the peak spectral accelerations.

5. CONCLUSIONS

In this paper, the influence of masonry infills on floor response spectra (FRS) in reinforced concrete buildings subjected to frequent (serviceability level) earthquakes was investigated. The FRS for bare and infilled frames, as well as the distribution of the peak floor accelerations, were compared. The results of the NLTH analyses demonstrated that the masonry infills significantly affect the evaluation of the FRS, in particular if the structures behave elastically under serviceability level ground motions. The presence of the masonry infills causes a shift of the FRS peaks to shorter periods and gives rise to a significant amplification of the corresponding peak spectral accelerations. The obtained results were compared to the FRS predicted by a simplified methodology available in the literature. The main methodologies available in literature were not developed to explicitly account for the presence of the masonry infills, for this reason further investigations are required to improve these methodologies in order to consider the influence of the masonry infills. The main issue that should be investigated is related to the definition of an amplification parameter in order to account for the highest peak spectral accelerations.
accelerations observed in masonry infilled reinforced concrete frames.

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NZS, Structural design actions - Part 5 Earthquake Actions, NZS 1170.5, New Zealand Standards, New Zealand.


PEER NGA-West database, available on-line: http://peer.berkeley.edu/ngawest


