Floor spectra for bare and infilled reinforced concrete frames designed according to Eurocodes

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Abstract
In this study, nonlinear time-history analyses are performed to assess the floor response spectra of bare and infilled reinforced concrete framed buildings with different number of stories and designed according to Eurocode provisions for different intensity levels of the seismic action. Infill walls are modeled by neglecting and by accounting for the effects of their out-of-plane response and of the in-plane/out-of-plane interaction. To this aim, a recent out-of-plane response model is updated and improved.

The results of the numerical analyses are compared in order to assess, first, the different floor spectra obtained for bare and infilled buildings and, second, the effect of the in-plane/out-of-plane interaction on the results obtained for infilled buildings. The main parameters influencing the shape and the amplitude of floor response spectra are investigated, namely higher vibration modes, structural nonlinearity, and damping of the secondary element. This is also performed with the support of the discussion and application of current code and literature formulations.

Based on the results of the numerical analyses, a simplified code-oriented formulation for the assessment of floor response spectra in bare and infilled reinforced concrete framed structures is proposed. The proposed formulation may be a useful tool for the seismic assessment and safety check of acceleration-sensitive nonstructural components.

KEYWORDS
floor response spectrum, formulation, infill wall, nonlinear analysis, out-of-plane, reinforced concrete building

1 | INTRODUCTION

In the framework of performance-based earthquake engineering, the seismic assessment of nonstructural components is a paramount issue: in fact, most of the earthquake-induced damage to buildings and the consequent economic losses is related to them. Nonetheless, the heavy damage of certain nonstructural components, such as infill walls, may even threaten human life safety. For these reasons, there is need for the development of robust and reliable models for the
assessmen of capacity and demand for their seismic safety assessment. This work is focused on the definition of the acceleration demand acting on them, that is, on floor response spectra.

As well known, a structure (hereinafter “primary structure”) subjected at ground to a certain acceleration time-history with a certain Peak Ground Acceleration (PGA) will experience a certain pseudo-spectral acceleration demand ($S_a$) different from PGA. The pseudo-spectral acceleration demand can be calculated from the response spectrum of the ground motion based on the structure modal (ie, the period $T$) and damping (ie, the damping ratio $\xi$) properties. At the same time, a certain floor of the structure will experience a certain acceleration time-history with a certain Peak Floor Acceleration (PFA). Consider now a nonstructural component (hereinafter “secondary element”) as a Single Degree of Freedom (SDOF) system—with vibration period $T_a$ and damping ratio $\xi_a$—supported by a certain floor of the primary structure. The maximum pseudo-spectral acceleration, herein identified as PSA, acting on it can be calculated through the pseudo-acceleration response spectrum of the acceleration time-history of the supporting floor. This is acceptable if the dynamic interaction between the secondary element and the primary structure is negligible. This pseudo-acceleration response spectrum is known as floor response spectrum. These concepts are summarized in Figure 1.

So, based on the above discussion, the seismic demand acting on a certain nonstructural component supported by a certain floor of the structure can be known if the PFA of that floor and the PSA spectral shape are known. In addition, based on the above discussion, it is possible to define some parameters that, intuitively, are expected to have an influence on the PFA and on the PSA spectral shape. Regarding the PFA, one can expect its dependence on the PGA value (the higher the PGA, the higher the PFA) and, due to the dynamic amplification of the fundamental vibration mode, on the height of the specific floor considered, $z$, potentially normalized with respect to the total building height, $H$. Regarding the spectral shape, one can expect that the PSA is amplified or de-amplified with respect to the PFA, with the maximum amplification expected due to resonance of the secondary element and of the primary structure, that is, when $T_a$ is closer to the fundamental vibration period(s) of the supporting structure. Also, the damping of the secondary element is expected to influence PSA maximum values. As will be shown later in this paper, further parameters may have a significant influence on both the PFA distribution along the building height and the floor spectral shape.

The study presented in this paper is aimed at defining code-oriented and simplified formulations for the assessment of floor response spectra in reinforced concrete framed buildings. First, a recall of current code and literature formulations is presented, mainly to define a systematic review of the geometric and mechanical structural parameters which are expected to influence the PFA distribution along the building height and the floor spectral shape. Second, nonlinear models of bare and infilled case-study reinforced concrete framed buildings with 2, 4, 6, and 8 stories designed for different PGA levels (0.05, 0.15, 0.25, and 0.35 g) according to Eurocodes are defined. Then, nonlinear time history analyses are performed on the case-study buildings to evaluate floor response spectra. For the sake of brevity, the results will be shown in this paper only for the extreme PGA levels (0.05 and 0.35 g) but all the results obtained are used for discussion and proposal purposes.

The numerical outcomes are compared to analyze the effect of the presence of infill walls on floor response spectra considering and not considering their out-of-plane response and the mutual interaction between their in-plane
and out-of-plane responses (ie, the so-called in-plane/out-of-plane interaction). To this aim, a recent out-of-plane and in-plane/out-of-plane interaction model proposed by the Authors of this study is updated to account for the potential softening and cyclic degradation of the out-of-plane response of infill walls. Note that modeling the out-of-plane response of infills and the in-plane/out-of-plane interaction is a novelty element for studies dedicated to floor spectra assessment, which rarely have been dedicated to infilled buildings in general. Finally, a selection of the outcomes of the analyses is compared with those expected based on code and literature formulations. This is done to assess the influence of different parameters on the analyses’ results and is preliminary to the proposal of a code-based simplified formulation for the evaluation of floor response spectra that may be useful for the safety check of acceleration-sensitive nonstructural components.

2 | EXISTING CODE AND LITERATURE PROPOSALS

Several studies have been dedicated to floor response spectra in past and recent times. Comprehensive state-of-the-art reviews can be found in Rodriguez et al., Vukobratović, Degli Abbatì et al., Wang et al. In this section, for the sake of conciseness, the discussion is focused on recent practice-oriented proposals available in the literature, as well as on main code prescriptions regarding this topic.

2.1 | Code proposals

Eurocode 8, in Section 4.3.5, proposes an expression, whose theoretical derivation is not clearly stated, for the PSA as shown, with some manipulation in the nomenclature, in Equation 1.

\[
PSA = PGA \left[ \frac{3(1 + z/H)}{1 + (1 - T_a/T_1)^2} - 0.5 \right].
\] (1)

In Equation 1, \(T_a\) is the nonstructural element vibration period and \(T_1\) is the fundamental vibration period of the structure. If \(T_a\) equals zero, a linear PFA distribution along the building height is obtained at varying \(z/H\) ratio. PFA ranges from \(PGA\) (at \(z\) equal to zero) to 2.5\(PGA\) (at \(z\) equal to \(H\)). On the other hand, the maximum PSA is obtained if \(T_a\) equals \(T_1\), even at the ground floor, where the maximum PSA should depend on ground motion characteristics. If \(T_a\) equals \(T_1\), the maximum PSA ranges from 2.5\(PGA\) (at \(z\) equal to zero) to 5.5\(PGA\) (at \(z\) equal to \(H\)). It should be noted that this formulation returns a maximum PSA/PFA ratio different for each floor. This will be observed also from the numerical analyses performed in this study.

ASCE/SEI 7-10, in Section 13.3.1, proposes an expression for the PSA as shown, with some manipulation in the nomenclature, in Equation 2.

\[
PSA = PGA \left( 1 + \frac{2z}{H} \right) a_p.
\] (2)

In Equation 2, \(a_p\) is a factor accounting for the amplification of acceleration due to the deformability of the nonstructural element reported in Table 13.5-1 of the code. It is worth noting that the American approach, with this factor, simplifies the calculation of the seismic demand acting on a nonstructural element, as there is no need of a more or less detailed dynamic characterization to determine its period, \(T_a\), which enters Eurocode 8 formulation.

Also according to the ASCE/SEI 7-10 approach, the PFA varies linearly along the building height. It ranges from \(PGA\) (at \(z\) equal to zero) to 3\(PGA\) (at \(z\) equal to \(H\)). The maximum value of \(a_p\) reported in Table 13.5-1 of the code is equal to 2.5. Hence, the maximum possible PSA value is always equal to 2.5\(PFA\), independently on the floor considered.

The New Zealand code NZSEE2017, in section C7.6.2, refers to the loading code NZS 1170.5 for the calculation of floor response spectra. NZS 1170.5, in Section 8.5.1, provides the formulation reported, with some manipulation in the nomenclature, in Equation 3.

\[
PSA = PGAC_{Hi}C_i(T_a).
\] (3)
$C_{Hi}$ is calculated through the formulations reported in Equation 4 and defines the PFA distribution along the building height which is, in this case, multilinear. The PFA ranges from PGA (at $z$ equal to zero) to 3PGA (at $z$ equal to $H$).

\[ C_{Hi} = \left(1 + \frac{z}{6}\right) \text{ for all } z < 12\text{m}, \]  
\[ (4a) \]

\[ C_{Hi} = \left(1 + 10 \frac{z}{H}\right) \text{ for } z < 0.2H, \]  
\[ (4b) \]

\[ C_{Hi} = 3.0 \text{ for } z \geq 0.2H. \]  
\[ (4c) \]

In Equation 3, $C_i(T_a)$ is the spectral shape coefficient. $C_i(T_a)$ is expressed as a function of the period $T_a$ and ranges from 0.5, for $T_a$ higher than 1.50 seconds, to 2.0, for $T_a$ lower than 0.75 seconds. Hence, the maximum possible PSA value is always equal to 2PFA, independently on the floor considered. It is worth noting that, in this case, the maximum PSA value depends on the “absolute” value of $T_a$, that is, it does not depend on $T_a/T_1$ ratio.

As above shown, the previous code prescriptions generally relate the PFA linear or multilinear distribution along the building height to the floor height normalized with respect to the building height (ie, the higher $z/H$, the higher the PFA) and the PSA spectral shape to the ratio between the $T_a$ and $T_1$. From both these features, it can be assumed that they relate the floor response spectra to the elastic response of the structure to its first vibration mode.

The commentary (Circolare)\(^9\) to the current Italian building code\(^10\) proposes both a “rigorous” and simplified approaches for the assessment of floor response spectra. Both account for multimodal contributions and for the effect on floor response spectra of structural nonlinearity. In fact, structural nonlinearity limits the maximum force acting on a structure; hence, it also may limit the maximum acceleration acting at its floors. Both topics have been also highlighted in recent literature, as will be shown in the next subsection.

The rigorous approach accounts for multimodal contributions based on simple considerations related to structural dynamics. In fact, the PFA\(_{ij}\) acting at the $j$th floor of the primary structure associated with its $i$th vibration mode is determined by means of Equation 5.

\[ \text{PFA}_{ij} = \Gamma_i \varphi_{ij} S_a(T_i). \]  
\[ (5) \]

In Equation 5, $\Gamma_i$ is the modal participation factor of the $i$th vibration mode, $\varphi_{ij}$ is the modal displacement of the $j$th story for the $i$th vibration mode, $S_a(T_i)$ is the spectral acceleration of the structure associated with its $i$th vibration period, potentially reduced by means of the structure behavior factor. The PSA is obtained, for each floor and for each vibration mode contribution, by amplifying or de-amplifying the PFA through an $R_{ij}$ factor (which equals PSA\(_{ij}\)/PFA\(_{ij}\)) calculated by means of Equation 6.

\[ R_{ij} = \frac{\text{PSA}_{ij}}{\text{PFA}_{ij}} = \left( \frac{2\xi_a T_a}{T_i} + \frac{1 - T_a/T_1}{2} \right)^{-1} \]  
\[ (6) \]

Equation 6 returns a PSA\(_{ij}\) value equal to $1/(2\xi_a)$ times PFA\(_{ij}\) for $T_a = T_i$. The value of the maximum amplification is a classical result of the dynamic of the damped SDOF system,\(^{11}\) even if obtained with a slightly different and less rigorous formulation.

For each story, the floor spectrum can be obtained by combining multimodal contributions through the Square Root of Sum of Squares (SRSS) rule, thus resulting in a spectral shape with multiple peaks corresponding to the number of significant modes considered. Implicitly, also the PFA at the $j$th floor can be obtained through the SRSS combination of the different modal contributions, thus resulting in a potentially nonmonotonic distribution along the building height. Both circumstances will be observed also from the analyses results presented in this study. Regarding the PFA distribution, it is worth noting that Equation 6 may yield to a de-amplification of the PFA with respect to the PGA value, especially in bottom floors of high-rise structures. This circumstance has been observed by some authors,\(^1,12,13\) especially when analyzing elastic models, but not by other authors and never in the analyses performed in this study. A comment to this issue will be given in Section 4.2.
Regarding Equation 6, it may appear quite surprising that a theoretical formulation referring to a harmonic motion appears in a code. Also, due to its nature, Equation 6 is expected to provide significantly overestimated values of the real maximum PSA. In the authors’ opinion, this should be interpreted as a very conservative tool provided by the code if no one of the proposed simplified formulations (one specifically dedicated to reinforced concrete framed structures, the other validated for masonry structures and borrowed by the work by Degli Abbatit al.3) is applicable by the designer.

More specifically, the simplified approach proposed by the Italian regulation for framed reinforced concrete structures (which are the topic of this study) is borrowed by the work by Petrone et al.12 Since this proposal is based on numerical analyses on nonlinear models of reinforced concrete framed structures, it implicitly account for both multimodal contributions and structural nonlinearity effects. The proposed formulations are reported, with some manipulation in the nomenclature, in Equation 7.

\[
\text{PSA} = \begin{cases} 
\text{PGA} \left( 1 + \frac{z}{H} \right) \left( \frac{a_p}{1+(a_p-1)\left(1-\frac{T_a}{aT_1}\right)^2} \right) \geq \text{PGA} \text{ for } T_a < aT_1, \\
\text{PGA} \left( 1 + \frac{z}{H} \right) a_p \text{ for } aT_1 \leq T_a < bT_1, \\
\text{PGA} \left( 1 + \frac{z}{H} \right) \left( \frac{a_p}{1+(a_p-1)\left(1-\frac{T_a}{bT_1}\right)^2} \right) \geq \text{PGA} \text{ for } T_a \geq bT_1.
\end{cases}
\]

(7)

In Equation 7, \(a\), \(b\), and \(a_p\) are coefficients depending on \(T_1\) that account for higher modes effects (the coefficient \(a\)), resonance period elongation due to nonlinearity (the coefficient \(b\)) and effect of nonlinearity on the maximum PSA value (the coefficient \(a_p\)). If \(T_a\) equals zero, a linear PFA distribution along the building height is obtained. PFA ranges from PGA (at \(z\) equal to zero) to 2PGA (at \(z\) equal to \(H\)). On the other hand, the maximum PSA is obtained if \(T_a\) is between \(aT_1\) and \(bT_1\). In this case, the floor spectral acceleration ranges from 2.5PGA to 5PGA, dependently on \(a_p\) value.

Circolare9 also reports a second simplified formulation borrowed by the work by Degli Abbatit al.3 This proposal has rigorous basis and has been validated against the experimental results obtained for a mixed reinforced concrete/masonry structure as well as against the results of numerical analyses carried out on an existing masonry structure. Given that a specific formulation is provided in Circolare9 for reinforced concrete structures, the second simplified formulation is not further investigated in this study.

It is worth noting that none of the proposals discussed in this subsection explicitly refers to infilled buildings, as if the presence of infills may be considered only through \(T_1\) value, that is, as if the presence of infills only affects the resonance period at which the maximum PSA is attained, except for the simplified proposal of the Italian regulation, in which also the maximum PSA value depends on \(T_1\) and attains its maximum potential value for small values of the period (ie, \(a_p = 5\) when \(T_1\) is lower than 0.5 seconds). In other words, based on this approach, an infilled building is expected to experience an equal or higher maximum PSA value, at a certain floor, with respect to an identical but bare structure. This is also generally observed in the numerical analyses shown in this paper.

### 2.2 Literature proposals

Calvi and Sullivan14 propose an approach for the assessment of floor response spectra for SDOF systems extended to Multidegree of Freedom (MDOF) systems and validated against the results of numerical time-history analyses on reinforced concrete wall structures. The PSA value for each floor and for each modal contribution is calculated by means of Equation 8.

\[
\text{PSA}_{ij} = \Gamma_i \varphi_{ij} a_{m,j}.
\]

(8)

In Equation 8, \(a_{m,i}\) is a spectral shape function given by Equation 9.

\[
a_{m,i}(T_a) = \begin{cases} 
\frac{T_a}{T_i} \left[ a_{\max,j} (\text{DAF}_{i \max} - 1) \right] + a_{\max,j} \text{ for } T_a < T_i, \\
a_{\max,j} \text{DAF}_{i \max} \text{ for } T_i \leq T_a < T_{i,eff}, \\
a_{\max,j} \text{DAF}_{i} \text{ for } T_a \geq T_{i,eff}
\end{cases}
\]

(9)
In Equation 9, \( a_{\text{max}, i} \) is the minimum between \( S_a(T) \) and the \( S_a \) corresponding to global yielding of the structure \( (S_{ay}, \) potentially determined by means of a nonlinear static analysis), DAF is given by Equation 10, with DAF\(_{\text{max}} \) obtained for \( T_a = T_{i,\text{eff}}. \)

\[
\text{DAF}_i = \left( \frac{1}{\sqrt{1 - \frac{T_a}{T_{i, \text{eff}}}^2} + \xi_a} \right)^{-1}. \tag{10}
\]

In Equations 9 and 10, \( T_{i,\text{eff}} \) is the effective period of the structure (which is assumed potentially different from the elastic vibration period only for the first vibration mode), potentially evaluated through a nonlinear static analysis.

This approach accounts for structural nonlinearity effects in terms of resonance period elongation and limitation of the maximum PSA. The values of PSA\(_j\) determined at a certain \( j \)th floor for each \( i \)th vibration mode by means of Equation 8 can be combined through the SRSS rule to obtain a unique PSA\(_j\) value associated with the \( j \)th floor accounting for the contributions of the significant vibration modes. Once PSA\(_j\) has been obtained, by assuming \( T_a \) equal to zero, PFA\(_j\) can be determined. Hence, also PFA\(_j\) accounts for multiple vibration modes contributions. Note that the results may be influenced by the combination rule adopted. Also in this case, as highlighted when discussing the Italian regulation “rigorous” approach, the PFA value may result lower than PGA. Since this is only seldom observed from numerical analyses, Calvi and Sullivan suggest that the PFA value should never be assumed lower than PGA at bottom floors and that, at the same floors, the floor response spectrum should never return PSA values lower than those associated with the ground motion response spectrum.

Vukobratović and Fajfar\(^{15,16} \) propose a theoretical-based (except for the determination of the maximum PSA value, which is empirical) approach for the assessment of floor response spectra validated against the results of numerical time-history analyses on reinforced concrete frames. The PSA value for each floor and for each modal contribution is calculated by means of Equation 11.

\[
\text{PSA}_{ij} = \frac{\Gamma_i \varphi_{ij}}{\left( \frac{R_{\mu, i}}{T_a} \right)^2 - 1} \sqrt{\left( \frac{S_a(T_{i, \text{eff}})}{R_{\mu, i}} \right)^2 + \left( \frac{T_a}{T_{i, \text{eff}}} \right)^2 S_a(T_a) \leq \text{AMP}_i \Gamma_i \varphi_{ij} \frac{S_a(T_{i, \text{eff}})}{R_{\mu, i}}}. \tag{11}
\]

In Equation 11, \( T_{i,\text{eff}} \) is different from the elastic period only for the first vibration mode and can be calculated by means of a nonlinear static analysis; \( R_{\mu, i} \) is the reduction factor due to the nonlinear behavior of the structure and is equal to the maximum between 1 and the ratio between \( S_a(T_{i, \text{eff}}) \) and \( S_{ay} \) (potentially determined by means of a nonlinear static analysis) for the first vibration mode, while it is equal to 1 for higher vibration modes; AMP\(_i\) is the maximum PSA\(_{ij}/\text{PFA}_{ij}\) value and is expressed as a function of the damping ratio similarly to the approach proposed by Calvi and Sullivan.\(^{14} \) The effects of multiple modes are combined by applying the SRSS rule, except in the postresonance region in which modal effects are combined by algebraic summation. Also in this case, the PFA can be calculated by evaluating the PSA resulting from modal combination at \( T_a \) equal to zero. Also in this case, PFA may result lower than PGA. In Vukobratović and Ruggieri,\(^{17} \) based on physics, it is suggested to assume PFA never lower than PGA at bottom floors.

Surana et al\(^{13} \) propose an empirical approach (based on numerical analyses on infilled reinforced concrete structures) for the assessment of floor response spectra. This approach accounts for the effects of the first two vibration modes as well as for the effect of structural nonlinearity, which is associated, differently from the previous approaches, also to the second vibration mode. This is observed also from the structural analyses carried out for this study. The formulations proposed by Surana et al\(^{13} \) present a quite complex form; moreover, their results, differently from what occurs when dealing with the other code and literature models, are significantly different from those obtained by means of the numerical analyses herein presented. Hence, for the sake of conciseness, these formulations are not shown in detail in this section.

### 2.3 Final remarks

In the previous subsections, it has been observed that, generally, floor response spectra are determined by two parts: a distribution of the PFA along the building height and a spectral shape function amplifying or deamplifying the PFA to obtain
TABLE 1 Summary of code and literature proposals

| Proposal | PFA profile shape | Floor-dependent spectral shape | Higher modes | Structural nonlinearity |
|----------|------------------|-------------------------------|--------------|------------------------|
| Eurocode 8 | Linear | ✓ | | |
| ASCE-SEI 7/106 | Linear | | | |
| NZSEE 2017 | Multilinear | | | |
| Circolare “rigorous” | Combination of mode shapes | ✓ | ✓ | ✓ |
| Circolare simplified/Petrone et al | Linear | ✓ | ✓ | ✓ |
| Circolare simplified/Degli Abbatì et al | Combination of mode shapes | ✓ | ✓ | ✓ |
| Calvi and Sullivan | Combination of mode shapes | ✓ | ✓ | ✓ |
| Vukobratović and Fajfar | Combination of mode shapes | ✓ | ✓ | ✓ |
| Surana et al | Multilinear | ✓ | ✓ | ✓ |

the PSA. This spectral shape can be fixed or different, in terms of maximum amplification and resonance period, from floor to floor. The maximum PSA value, attained in case of resonance, is usually determined empirically. There are different significant parameters influencing both the PFA and the spectral shape function, above all the effect of higher vibration mode and structural nonlinearity. Not all the above listed proposals account for all these parameters, as summarized in Table 1.

3 | NONLINEAR ANALYSIS PROCEDURE

3.1 | Design and modeling of RC frames

The 2-, 4-, 6-, and 8-story case-study reinforced concrete moment resisting frames have regular rectangular plan defined by five and three bays in the longitudinal and transverse direction, respectively. Beams are 4.5 m long; columns are 3.0 m high. The buildings were designed with a Response Spectrum Analysis (RSA) according to Eurocode 2 and Eurocode 8 for four different design PGA values at Life Safety Limit State (0.05, 0.15, 0.25, and 0.35 g) in “High” Ductility Class (DCH). The materials used for design are class C28/35 concrete with characteristic compressive strength of the cylinder equal to 28 N/mm² and steel rebars with characteristic yielding stress equal to 450 N/mm². It is assumed that floor slabs have a diaphragm behavior. Further details on the design of the case-study reinforced concrete frames are available in Di Domenico. For the analyses, the reinforced concrete elements’ nonlinearity is modeled by adopting a lumped-plasticity approach in OpenSees by using ModIMKPeakOriented Material with response parameters determined according to Haselton et al and with the introduction of the cracking point. In the modeling process, average material properties are used, namely a compressive strength for concrete equal to 36 N/mm² determined according to Eurocode 2 and a steel yielding stress equal to 517.5 N/mm² determined according to Fardis et al.

3.2 | Modeling of infill walls

Two different unreinforced masonry infill layouts are considered. The first is constituted by a two-leaf (thickness: 80+120 mm) “weak” infill wall (weak layout, WL), the second is constituted by a one-leaf (thickness: 300 mm) “strong” infill wall (strong layout, SL). Note that the two-leaf infills are constituted by independent noninteracting panels. In other words, the in-plane and the out-of-plane responses of each leaf are determined (and modeled) independently on those associated with the other leaf.

The mechanical properties of infills are those calculated for the masonry wallets tested by Calvi and Bolognini for the WL and those by Guidi et al for the SL. The mass of infill panels is obtained by multiplying the panel nominal volume times the density of masonry proposed by the Italian regulation equal to 800 kg/m³. Each infill leaf is introduced in the structural model by using a couple of equivalent no-tension struts.
The in-plane nonlinear behavior is modeled, separately for each leaf, based on the proposal by Panagiotakos and Fardis. The two layouts analyzed are characterized by similar elastic in-plane stiffness but by different in-plane and out-of-plane strength capacity. In other words, buildings with WL and SL infills have similar elastic period, but those with WL infills are more likely to experience nonlinearity due to infills’ cracking. Further details regarding infill walls’ modeling are available in Ricci et al.26

Regarding the out-of-plane response, the modeling strategy proposed by Ricci et al 27 is adopted in order to account for the in-plane/out-of-plane interaction effects, that is, the degradation of the out-of-plane strength and stiffness due to in-plane damage and of the in-plane strength and stiffness due to out-of-plane damage. The equations adopted for modeling the out-of-plane response are those proposed in Ricci et al 26 for infills in which the out-of-plane response is governed by two-way arching strength mechanism, except for those associated with some response parameters. In fact, for this study, the out-of-plane response model is improved and updated.

First, the empirical equation for the assessment of the out-of-plane first macro-cracking force, $F_{\text{crack}}$, is updated to account for recent experimental evidences proposed by De Risi et al 28 and by Di Domenico et al. 29 The new formulation is reported in Equation 12.

$$F_{\text{crack}} = 5.90 f_m v^{0.11} t^{0.83} w h.$$  

In Equation 12, forces are expressed in Newtons and lengths are expressed in millimeters. In addition, the formulations proposed by Di Domenico et al 29 are used to model the in-plane/out-of-plane interaction effects. In these formulations, the reduction of the out-of-plane strength and stiffness capacity is related not only to the in-plane interstory drift ratio demand and to the infill wall vertical slenderness (ie, the $h/t$ ratio), but also to the infill wall aspect ratio $w/h$.

However, the most important and significant improvement of the out-of-plane response model is the introduction of the softening branch in the out-of-plane response backbone, as shown in Figure 2A, and the assessment of the cyclic degradation of the response envelope. As shown in Figure 2A, the out-of-plane response backbone adopted in Ricci et al 26 was trilinear with a plastic branch after the peak load point up to the attainment of the displacement, equal to 0.30 times the infill thickness, corresponding, on average and based on experimental tests, to the 20% degradation of the maximum out-of-plane strength. In this study, the “conventional” ultimate point is retained at a displacement equal to 0.30 times the infill thickness. However, a bilinear softening branch is modeled. The first part of the softening branch goes from the peak load point to the “conventional” ultimate point. The second part goes from the “conventional” ultimate point to the “collapse displacement” point corresponding to zero out-of-plane strength capacity, that is, at vanishing of arching strength mechanism. As demonstrated in Angel et al 30 and further explained in Di Domenico et al. 31 vanishing of both vertical and horizontal arching effect is expected, based on geometrical considerations, at an out-of-plane central displacement of the infill wall equal to 0.80 times the infill thickness. So, the out-of-plane collapse displacement is set, in this study, to this value.

The quadrilinear response envelope shown in Figure 2A is modeled in OpenSees by adopting Pinching4 Material. Pinching4 Material also allows modeling the hysteretic degradation of strength, unloading and reloading stiffness, as well as the so-called “pinching” effect. Further details on the nature and meaning of the hysteretic parameters governing Pinching4 Material are available in Di Domenico et al. 29

\[ F_{\text{crack}} = 5.90 f_m v^{0.11} t^{0.83} w h. \]  

**Figure 2** Updating of the out-of-plane response model: (a) new response envelope and (b) calibration of Pinching4 Material hysteretic parameters based on the cyclic test performed at DIST-UNINA.
Material response are available in Lowes et al.\textsuperscript{32} A preliminary calibration of these parameters has been performed by carrying out at the laboratory of the Department of Structures for Engineering and Architecture of University of Naples Federico II (DIST-UNINA) a cyclic out-of-plane test on an unreinforced masonry infill with thickness equal to 120 mm and poor mechanical properties (i.e., nominally identical to those monotonically tested in Di Domenico et al.\textsuperscript{33} whose out-of-plane force ($F_{OOP}$)-displacement ($d_{OOP}$) response is shown in Figure 2B. Further details on this experimental test are available in Di Domenico.\textsuperscript{19}

The Pinching4 Material hysteretic parameters have been calibrated by setting damage degradation type to “energy” and to minimize the distance between the experimental and the predicted energy dissipation history during the entire experimental test. They have been calibrated by neglecting the force cyclic degradation, since the limited number of comparisons between monotonic and cyclic experimental tests available\textsuperscript{34,19} shows that no significant strength degradation is observed when comparing monotonic and cyclic out-of-plane tests. The calibrated parameters are $gK_1 = 2.00$, $gK_2 = 0.55$; $gK_3 = 1.50$; $gK_4 = 0$; $gD_1 = 0.45$; $gD_2 = 0.85$; $gD_3 = 0.45$; $gD_4 = 0.50$; $gF_1 = gF_2 = gF_3 = gF_4 = 0$; $gK_{Lim} = 1$; $gD_{Lim} = 1$; $gF_{Lim} = 0$; $gE = 100$; $rDispP = rDispN = -0.40$; $rForceP = rForceN = -1.00$; $uForceP = uForceN = -0.40$. As already highlighted, the number of cyclic out-of-plane experimental tests is very limited. Further experimentation is needed to achieve a more robust and reliable calibration of the hysteretic parameters of the out-of-plane response of unreinforced masonry infills, also characterized by different geometric and mechanical properties.

### 3.3 Analysis procedure

The nonlinear time-history analyses have been performed by applying to each case-study structure ten bidirectional ground motions selected in the European Strong Motion (ESM) Database.\textsuperscript{35} Some properties of the selected records, which were registered on stiff and horizontal soils (i.e., type A soils according to Eurocode 8)\textsuperscript{5} are reported in Table 2. Both components of the selected records were simultaneously matched (to ensure the condition of spectrum-compatibility) to Eurocode 5%-damped design spectrum at Life Safety Limit State by using the RspMatchBi software.\textsuperscript{36} Then, for each case-study building, each record was scaled in order to provide it with a PGA equal to the design PGA of the considered case-study building. The matched and scaled record component registered in the NS direction was applied along the longitudinal global direction, while the matched and scaled component registered in the EW direction was applied along the transverse global direction. Further details on record selection are available in Refs. (19) and (26).

Nonlinear time-history analyses were performed on the Bare Frame (BF) models, on the infilled frame models with WL infills and SL infills but modeling only their in-plane response (WL and SL models), and on the infilled frame models with WL infills and SL infills with modeling both the out-of-plane response and the in-plane/out-of-plane interaction effects (WLOOP and SLOOP models). In the following, each case-study building is identified using an acronym, such as XYP_Z, in which X is the number of stories, Y the design PGA at Life Safety Limit State expressed in g/100, Z the model identifier (BF, WL, SL, WLOOP, and SLOOP).
3.4 | Expected nonlinear demand on the case-study buildings

As above stated, in this study floor spectra will be determined also by accounting for the effect on them of the nonlinear response of the supporting reinforced concrete structure. Since the case-study buildings were designed with an RSA for a certain PGA level, and since they are analyzed with nonlinear time-history analyses for the same PGA demand level, it is not expected that structural members will yield during nonlinear time-history analyses. Hence, nonlinearity is expected to be due only to elements’ cracking (remember that reinforced concrete elements are modeled by accounting also for the cracking point in the assigned moment-chord rotation response envelope) and to infills’ (if present) cracking.

This is confirmed by Figure 3, in which the pushover curves obtained by applying a first-mode-shaped lateral load pattern to both bare and infilled structures are shown. By normalizing the top displacement (\(\Delta_{TOP}\)) applied during static pushover (SPO) analyses with respect to the average top displacement demand registered during time-history analyses (\(TH\)), it is observed that this displacement demand is not able to produce, on average, elements’ yielding, but only tangent stiffness reduction due to members’/infills’ (if present) cracking. It should also be noted that the first significant nonlinearity in the pushover curve occurs for a \(\Delta_{TOP,SPO}/\Delta_{TOP,TH,average}\) value which is lower for buildings designed for higher PGA (ie, the stiffer and stronger ones). That being considered, Figure 3 shows that buildings designed and analyzed for higher PGA level experience a higher nonlinearity demand. All these outcomes will have an influence on the results in terms of floor response spectra that will be shown in the next section.

4 | TIME-HISTORY ANALYSIS RESULTS

In this section, the results of the time-history analyses are presented and discussed. Namely, the outcomes in terms of PFA/PGA profiles and PSA/PFA spectral shapes are shown. The spectral shapes are calculated by assuming a code-consistent value of the damping ratio equal to 5%. For each case-study building, the results shown are obtained as the average of the results obtained from the ten time-history analyses performed. For the sake of brevity, only some significant results are shown. These results can be considered representative of the general trends observed from the outcomes of all the numerical analyses. Then, these outcomes are compared with the results of the application of code and literature proposals described in Section 2.
The PFA/PGA profiles along the longitudinal direction for a selection of case-study buildings are shown in Figure 4. Some considerations can be drawn:

(i) The maximum value of PFA/PGA is generally between 3 and 4, it is registered at top floors and is lower for buildings designed and analyzed for higher PGA (see point v. for comments on this). Among code and literature formulations, NZSEE2017 approach returns a maximum PFA/PGA value equal to 3 at the top floors of buildings; in addition, a maximum PFA/PGA ratio equal to 4 was expected at top stories according to Uniform Building Code. Hence, the PFA/PGA maximum values obtained from the analyses can be deemed reasonable.

(ii) The shape of the profiles is roughly linear for 2-story buildings, while it becomes multilinear for the other buildings and nonmonotonic for taller (6- and 8-story) buildings. This is an effect of the influence of higher vibration modes, which, as it is well-known, is more significant in high-rise structures. This is expected based on the more refined code and literature proposals discussed in Section 2. The effect of higher modes appears also more visible and significant for infilled structures than for bare structures, as well as in presence of higher nonlinearity (ie, for higher PGA demand, see point v. for comments on this). This occurs because nonlinearity mainly affects the structural response to the first vibration mode, thus making more “visible” the contribution of higher vibration modes on PFA/PGA profiles.

(iii) Generally, at fixed building and at fixed floor, the maximum PFA/PGA value is registered for infilled buildings without modeling of out-of-plane response and in-plane/out-of-plane interaction, except for the 2-story buildings. This occurs due to the dependence of the PFA on $S_o(T_1)$. In fact, in 4-, 6-, and 8-story buildings, $S_o(T_1)$ is generally higher for infilled buildings, since $T_1$ is lower for infilled buildings and $T_1$ for bare frames is higher than the corner period $T_C$ equal to 0.40 seconds; on the contrary, $T_1$ for 2-story bare structures is lower than $T_C$ so, infilled buildings, which are always characterized by lower $T_1$ than bare buildings, are characterized, in this case, also by lower $S_o(T_1)$. The dependency of the PFA values on $S_o(T)$ is expected based on the more refined code and literature proposals discussed in Section 2.

(iv) Generally, at fixed building, the PFA/PGA profiles for infilled buildings with out-of-plane response of infills and in-plane/out-of-plane interaction modeled tend to approach the profile obtained for the bare structures. This is expected, since when the in-plane/out-of-plane interaction is modeled, the in-plane response of the infill deteriorates, in terms of both stiffness and strength capacity, due to the out-of-plane displacement demand. So, the general behaviour of the building appears intermediate between those of the bare and of the infilled (with in-plane response only of infills modeled) buildings.

(v) Generally, the trends described above are confirmed when passing from the buildings designed and analyzed for PGA equal to 0.05 g to those designed and analyzed for PGA equal to 0.35 g. However, they are sometimes disturbed due
FIGURE 5 PSA/PFA spectral shapes along the longitudinal direction of a selection of case-study buildings

Based on the above considerations, which are corroborated by the discussion of literature and code proposals presented in Section 2, in can be concluded that, with reference to the set of case-study buildings analyzed for this study, the significant parameters influencing the values of PFA/PGA ratios are the $S_a(T)$ acting on the structure and on the PGA value itself (which is representative of the level of expected nonlinearity in the structure), both referred to the ground motion. On the other hand, the shape of the PFA/PGA profile depends on the importance of higher modes, which is higher for taller buildings, for infilled buildings, and in presence of higher nonlinearity demand (ie, for higher PGA).

The PSA/PFA spectral shapes obtained along the longitudinal direction for a selection of floors of a selection of case-study buildings are shown in Figure 5. The value of $T_1$ used to normalize the period $T_a$ is the period of the elastic uncracked bare frame for WLOOP and SLOOP models: in fact, as previously discussed, due to the in-plane/out-of-plane interaction effects, the behavior of models in which the out-of-plane response of infills is considered is intermediate between the behavior of the bare structural model and that of the infilled structural model with only the in-plane response of infills

...to structural nonlinearity, which in general, yields to a reduction of the PFA/PGA demand. This is expected based on the more refined code and literature proposals discussed in Section 2. Remember that, as shown in Section 3.4, the reduction of PFA at increasing demand PGA and, as will be shown in the following, period elongation and PSA reduction, is mainly due to elements'/infill's' cracking. This has been observed also by Petrone et al. 12
considered; for bare and WL/SL models, the period adopted for normalization is the one calculated by means of modal analysis of the elastic uncracked bare models for BF models, of the elastic uncracked infilled models for WL/SL models.

Some considerations can be drawn:

(i) The peak values of PSA/PFA (especially those corresponding to the first vibration mode in high-rise buildings) can vary from floor to floor, as some of the code and literature models described in Section 2 show. In general, it appears dependent on the PFA/PGA value: the higher the PFA/PGA, the higher the peak value of PSA/PFA associated with the first vibration mode. Generally speaking, the maximum observed value of the PSA/PFA ratio is roughly equal to 5.5, which is consistent with the maximum value, equal to 5, predicted by the proposal by Petrone et al.\textsuperscript{12} adopted also by the Italian code.

(ii) The spectral shapes are characterized, in general, by multiple peaks corresponding to resonance with the multiple vibration modes of the primary structure. This is quite evident in taller buildings, in which the maximum value of PSA/PFA associated with the higher modes (i.e., \( T_a/T_1 \) significantly lower than the unit) may be even higher than that associated with the first vibration mode (i.e., \( T_a/T_1 \) similar to \( T_1 \)). In 2-story buildings, the effect of higher modes is visible only at the first floor, at which, actually, the maximum influence of the second vibration mode is expected. On the other hand, in the 8-story buildings, peaks of PSA/PGA associated with higher modes are visible at different \( T_a/T_1 \) values dependently on the floor considered. This occurs because the predominant effect among higher modes may be associated with different higher modes from floor to floor (e.g., the second or the third vibration mode of the structure).

(iii) In general, infilled buildings are characterized by higher values of the maximum PSA/PFA ratio associated with the first vibration mode with respect to bare buildings, except for the 2-story buildings. This occurs, most likely, because \( S_a(T_a = T_1) \) is higher for infilled buildings with respect to \( S_a(T_a = T_1) \) for bare buildings, since \( T_1 \) of the infilled building is lower than \( T_1 \) of the bare building, which, in tune, is higher than \( T_C \). As already explained when discussing maximum PFA/PGA trends, this is not true for 2-story buildings, since in this case \( S_a(T_a = T_1) \) is lower for infilled buildings than for bare buildings. This occurs because \( T_1 \) of the infilled building is still lower than \( T_1 \) of the bare building, but \( T_1 \) of the bare building is lower than \( T_C \). The dependence of the maximum PSA/PFA value on \( S_a(T_a = T_1) \) may be also expressed by relating it directly to the period \( T_1 \), as done by Petrone et al.\textsuperscript{12}

(iv) As also shown when discussing the PFA/PGA profiles, also the spectral shapes for WLOOP and SLOOP models are intermediate between those obtained for the BF models and those obtained for WL and SL models. The fact that for WLOOP and SLOOP models the peak of PSA/PFA due to resonance with the first vibration mode occurs at \( T_a/T_1 < 1 \) is due to the fact that, in this case, \( T_a \) is normalized with respect to \( T_1 \) of the bare frame, while the real resonance period is lower than it, being intermediate between the bare and the infilled structure elastic vibration period.

(v) Generally, the trends described above are confirmed when passing from the buildings designed and analyzed for PGA equal to 0.05 g to those designed and analyzed for PGA equal to 0.35 g. However, the peak PSA/PFA associated with the first vibration mode of the structure is noticeably reduced due to the effect of structural nonlinearity; in addition, the resonance period is higher than \( T_1 \) for the period elongation due to nonlinearity. Remember that, as previously highlighted, this effect is due to elements’/infills’ cracking. Both effects of structural nonlinearity are expected also by applying the more refined literature proposals discussed in Section 2. Note also that, even if with a lower impact, also the peaks associated with higher vibration modes may be reduced due to structural nonlinearity. This was observed also by Surana et al.\textsuperscript{13}

Based on the above considerations, which are corroborated by the discussion of literature and code proposals presented in Section 2, in can be concluded that, with reference to the set of case-study buildings analyzed for this study, the significant parameters influencing the values of the maximum PSA/PFA ratios associated with the resonance with the \( i \)th vibration mode at a certain floor are the PFA/PGA value at that floor, the \( S_a(T_a = T_1) \) value, potentially substituted directly by \( T_1 \), and the PGA value (which is representative of the level of expected nonlinearity in the structure). On the other hand, the shape of the PSA/PFA profile is characterized by a peak associated with the resonance of the secondary element with the structure first vibration mode and with a group of close peaks associated with the resonance with the structure higher modes.
4.2 Comparison with code and literature formulations

In the following, comparisons between the results of the numerical analyses and the proposals by code and literature discussed in Section 2 are shown. SL and WL models are neglected, since we refer hereafter to the case-study structures in which a “complete” modeling of infills has been performed, that is, in which the out-of-plane response of infills is considered as well as the in-plane/out-of-plane interaction effects. When applying proposals in which structural nonlinearity is considered, the necessary parameters are determined by means of a nonlinear static analysis of the case-study buildings; when applying Circolare9 “rigorous” approach, the \( S_\alpha(T_i) \) is calculated by dividing the elastic spectral acceleration by the behavior factor adopted during the design process, \( q \), equal to 4.68. This may yield to a significant underestimation of PFA/PGA and PSA/PFA values, since dividing \( S_\alpha(T_i) \) by the behavior factor is equivalent to assuming that the pseudo-spectral acceleration at global yielding of the structure is exactly equal to the design pseudo-spectral acceleration: this is not true, since different overstrength sources make the case-study structures able to withstand a pseudo-spectral acceleration higher than \( S_\alpha(T_i)/q \) without yielding as shown, for the case-study buildings analyzed in this study, by Ricci et al.38 and in section 3.4.

The PFA/PGA profiles obtained along the longitudinal direction for a selection of case-study buildings are compared with those predicted by code and literature formulations in Figure 6.

As already discussed in Section 2, some approaches based on the combination of modal contributes may yield to the prediction of a PFA value lower than PGA at bottom stories, especially in tall buildings, a circumstance never observed from the numerical time-history analyses presented in this study (and rarely observed in past studies). According to Hadjian,39 this is due to the fact that the “real” response of bottom stories is strongly influenced by the “stiff” motion of the dynamic system that is included both in higher modes (which, being typically characterized by very low participating mass are often neglected in common-practice modal combination rules) and in lower modes (which partially contribute to the stiff motion, too). Only elaborated modal combination rules can account for these effects. So, some authors, such as Calvi and Sullivan14 and Vukobratović and Ruggieri,17 suggest assuming a PFA value never lower than PGA at bottom stories of buildings. On the contrary, this is not suggested explicitly by Circolare9 for the “rigorous” approach.

It is observed from Figure 6 that among code and literature models able to provide different PFA/PGA profiles for the different models (ie, for models in which the PFA/PGA profiles do not depend only on geometric parameters), Calvi and Sullivan14 approach works quite well for 2-story buildings but, as most of the other proposals, significantly underestimates the PFA/PGA profiles for the 8-story buildings. Regarding the remaining code proposals according to which the PFA/PGA profile only depends on geometric parameters, it seems that Eurocode 85 formulation works quite well for the 2-story building, while NZSEE20177 proposal have a better performance on the 8-story building. In general, the underestimation of the numerical results appears quite significant. However, it should also be considered that the capacity of providing good estimates of floor spectra should be assessed on the absolute PSA values: in this sense, the underestimation of PFA/PGA provided by literature proposals could be equilibrated by an overestimation of PSA/PFA values. In addition, it should be remembered that some proposals existing in the literature have been validated against the results obtained on numerical models in which members’ force-deformation response was modeled by adopting a typical elastic-plastic bilinear curve. Such proposals should be always applied with caution for comparison with numerical outcomes obtained by adopting different modeling approaches for reinforced concrete members, as in the present case.

The PSA/PFA spectral shapes obtained along the longitudinal direction for a selection of case-study buildings are compared with those predicted by code and literature formulations in Figure 7. The comparison with ASCE-SEI 7/106 formulation is not reported, since it does not define a spectral shape, as discussed in Section 2. Note that the application of code and literature formulation was performed by normalizing \( T_{a1} \) with respect to the appropriate effective \( T_1 \) period. However, considering that the normalization period adopted for different formulations may be different in the graphs reported in Figure 7 and only for representation purposes, \( T_{a1} \) is always normalized with respect to the elastic period of the bare frame.

It is observed from Figure 7 that the approach proposed by Eurocode 85 fails in predicting the PSA/PFA peaks due to higher vibration modes, while it works quite well, in terms of “envelope” of different floor response spectra, for buildings in which a significant level of nonlinearity is expected (ie, those designed and analyzed for PGA equal to 0.35 g). This occurs also for NZSEE20177 approach. The approach by Vukobratović and Fajfar15,16 shows a good performance, especially when dealing with buildings designed and analyzed for low PGA, while it appears slightly conservative for buildings designed and analyzed for high PGA. However, it should be noted that, as already pointed out, the capacity of providing good estimates of floor spectra should be assessed on the absolute PSA values: in this sense, the overestimation of PSA/PFA provided by literature proposals is equilibrated by the underestimation of PFA/PGA values. Circolare9 “rigorous” approach
appears significantly conservative (for the reasons discussed in Section 2). A better (even if still conservative, especially at bottom floors) performance is provided by the simplified model proposed by Circolare.9 In summary, the best performance seems to be obtained by applying the approach by Calvi and Sullivan,14 both in terms of localization of maximum peaks and in terms of maximum PSA/PFA values corresponding to resonance periods, even if with a slight conservativeness for high values of $T_a/T_1$.

5 | CODE-ORIENTED PROPOSAL FOR THE ASSESSMENT OF FLOOR RESPONSE SPECTRA AT DESIGN LEVEL OF SEISMIC DEMAND

Based on the outcomes of the numerical analyses described in the previous section, a new proposal for the assessment of floor response spectra at the design level of seismic demand for bare and infilled reinforced concrete framed structures is herein presented. More specifically, a formulation is proposed for the assessment of the PFA/PGA profile as well as a formulation for the assessment of the PSA/PFA spectral shape. The proposal is based on the basic principles listed below.
(i) The formulations proposed should depend on predictor parameters which can be simply calculated by the practitioner, that is, only by means of a linear analysis of the elastic uncracked structure. For this reason, the effect of nonlinearity is considered, when necessary, through the value of the demand PGA at Life Safety Limit State, which is equal, for a practitioner and for the structural analyses carried out in this study, to the design PGA at Life Safety Limit State. This simplification, which improves the ease-of-use of the proposal, can be deemed acceptable since the higher the design PGA, the higher is the observed nonlinearity demand in the case-study buildings (ie, design PGA and nonlinearity demand level are positively correlated). Remember that the nonlinearity demand in the case-study buildings herein analyzed is due to cracking of members (and infills, if present). This nonlinearity demand level at the design PGA is not equal for all the case-study buildings (ie, it is not independent on the design PGA) since it is not controlled during the design process. At PGA higher than the design one (which is typically outside the scope of a
code-oriented safety assessment of nonstructural components), an even higher nonlinearity demand level is expected due to members’ yielding and, potentially, softening. This could qualitatively and significantly change the observed results in terms of both PFA/PGA profiles and PSA/PFA spectral shapes.

(ii) The formulations proposed for infilled buildings are calibrated based on the results obtained for infilled models with “complete” modeling of infill walls, that is, by accounting for their out-of-plane response and for the in-plane/out-of-plane interaction.

(iii) The formulations proposed should account for the effect of higher vibration modes and of structural nonlinearity. In addition, the formulation proposed for the assessment of PSA/PFA spectral shape is referred to an elastic secondary element with damping ratio equal to 5%. However, a correction coefficient accounting for the damping ratio of the secondary element is provided in Section 5.3.

5.1 Assessment of PFA/PGA profiles

Based on the outcomes of the numerical analyses, the shape function suggested for the assessment of PFA/PGA profiles is reported in Equation 13 and shown in Figure 8.

\[
\frac{\text{PFA}}{\text{PGA}} = \alpha \left( \frac{z}{H} \right)^\beta \geq 1. 
\]

(13)

In Equation 13, \( \alpha \) is the value of PFA/PGA at the top floor of the building, while \( \beta \) is a shape factor ranging from 0 (“bulged” shape of PFA/PGA profile) to 1 (linear shape of PFA/PGA profile).

As discussed in the previous section, the PFA/PGA value at the top floor, \( \alpha \), is strictly related to the \( S_a \) acting on the considered structure; in addition, it may be limited due to the effect of nonlinearity, which is higher at higher PGA demand. For these reasons, the predictor parameter \( s \) and the upper and lower bound values of \( \alpha \), \( \alpha_{min} \), and \( \alpha_{max} \), are defined in Equations 14-16.

\[
s = \frac{1}{2} \sqrt{\left( \Gamma_1 S_a (T_1) \right)^2 + \left( \Gamma_2 S_a (T_2) \right)^2} / \text{PGA},
\]

(14)

\[
\alpha_{min} = 2.50,
\]

(15)

\[
\alpha_{max} = 3.20 + 4 (0.35 - \text{PGA}) \geq 3.20.
\]

(16)

In Equations 14 and 16, accelerations are expressed in g units; \( \Gamma_i \) is calculated by normalizing the mode shape with respect to the top floor modal displacement. The \( \alpha \) value can be calculated by means of Equation 17.

\[
\alpha = \alpha_{min} + (\alpha_{max} - \alpha_{min}) (s - 1) \leq \alpha_{max}.
\]

(17)
The shape factor $\beta$ should be calculated by accounting, as shown in Section 4.1, for the fact that the PFA/PGA profile tends to be linear ($\beta$ tends to 1) for low-rise buildings, for bare buildings and for low level of nonlinear demand; on the other hand, the same profile tends to “bulge” ($\beta$ tends to 0) for high-rise buildings, for infilled buildings and for high level of nonlinearity demand. For these reasons, Equations 18–21 are suggested for the calculation of this coefficient. Also in this case, PGA is expressed in g units.

\[
\beta = \frac{T_1}{T_{1L}} \beta_{\text{min}} + \left(1 - \frac{T_1}{T_{1L}}\right) \beta_{\text{max}} \geq \beta_{\text{min}},
\]

(18)

\[
\beta_{\text{min}} = 0.20,
\]

(19)

\[
\beta_{\text{max}} = 0.80 - \text{PGA} \geq 0.45,
\]

(20)

\[
\begin{cases} 
T_{1L} = 0.80 \text{ s} & \text{for bare buildings} \\
T_{1L} = 0.40 \text{ s} & \text{for infilled buildings}
\end{cases}
\]

(21)

The comparison between the numerical and predicted PFA/PGA profiles for the longitudinal direction of a selection of case-study structures is shown in Figure 9.

To apply Equations 14 and 18, the elastic vibration periods and modal participation factor of the uncracked structure must be known. In case of infilled structure, if its elastic model has not been built, Equations 14 and 18 could be applied by adopting the modal participation factor of the bare structure with no significant error; the first vibration period of the infilled structure can be calculated by applying Equation 22 by Ricci et al.\textsuperscript{40} in which $H$ is the height of the building and $L$ is the length of the building plan dimension in the direction of interest, both expressed in meters. This formulation predicts the observed elastic periods of the infilled buildings analyzed in this study with observed-to-predicted ratios equal to 1.02, median equal to 1.01, and coefficient of variation equal to 7\%. The second vibration period of the infilled structure can be calculated by applying Equation 23, which has been derived based on the second elastic vibration period of the buildings analyzed for this study (when applying Equation 23, observed-to-predicted ratios have mean and median equal to 1 and coefficient of variation equal to 25\%).

\[
T_1 = 0.063 \frac{H}{\sqrt{L}},
\]

(22)

\[
T_2 = 0.026 \frac{H}{\sqrt{L}}.
\]

(23)

### 5.2 Assessment of PSA/PFA spectral shape

Regarding the assessment of PSA/PFA at resonance and of the corresponding periods, it is assumed that two “resonance regions” exist, as shown in Figure 10: the first is associated with the first vibration mode of the structure; the second is associated with higher modes. In the first “resonance region,” PSA/PFA maximum value is named AMP1; in the second “resonance region,” PSA/PFA maximum value is named AMP2. Note that AMP2 is not necessarily associated with the second vibration mode: as shown in Section 4.1, at bottom stories of high-rise buildings the maximum amplification of PSA/PFA may occur due to the third (or higher) vibration mode. This choice (ie, adopting a unique “plateau” enveloping the PSA/PFA peaks due to higher vibration modes) may be not appropriate for tall buildings, for which two (or more) peaks associated with higher vibration modes can be clearly distinct and visible.\textsuperscript{17} However, it has been adopted as a reasonable compromise between accuracy and ease-of-use of the proposed approach.

First, AMP1 and AMP2 values, together with the resonance periods at which they are registered, $T_{R1}$ and $T_{R2}$, respectively, were determined for each floor of each case-study structure. These observed variables are related to predictor parameters by means of a least square regression analysis. More specifically, the regressions were performed by correlating the natural logarithm of each observed variable with a set constituted by $i$ ($i = 1, \ldots, n$) candidate parameters. First, the full model ($i = n$) was selected as the “reference model”. Then, $F$-tests were performed comparing the reference full model
**FIGURE 9** PFA/PGA profile along the longitudinal direction of a selection of case-study buildings (continuous lines) compared with the outcomes of the proposal presented in Section 5 (dashed lines).

**FIGURE 10** Proposed PSA/PFA spectral shape. [Correction added on 26 Jul 2021 after first online publication: the ‘!’ symbol should be replaced with ‘1’ along y axis.]
TABLE 3  Statistics of the observed-to-predicted ratios for AMP1, AMP2, $T_{R1}/T_{1,\text{BARE FRAME}}$, and $T_{R2}/T_{1,\text{BARE FRAME}}$

| Observed                  | Observed-to-predicted ratios (bare buildings) | Observed-to-predicted ratios (infilled buildings) |
|---------------------------|----------------------------------------------|---------------------------------------------------|
|                           | Mean | Median | Coefficient of variation | Mean | Median | Coefficient of variation |
| AMP1                      | 1.03 | 0.99   | 22%                       | 1.03 | 1.01   | 20%                     |
| AMP2                      | 1.00 | 0.99   | 10%                       | 1.01 | 0.99   | 15%                     |
| $T_{R1}/T_{1,\text{BARE FRAME}}$ | 1.00 | 0.99   | 8%                        | 1.01 | 1.05   | 15%                     |
| $T_{R2}/T_{1,\text{BARE FRAME}}$ | 1.06 | 1.02   | 34%                       | 1.04 | 1.03   | 28%                     |

with all reduced models ($i = 1, \ldots, n-1$) to associate with each reduced model a p-value related to the null hypothesis of statistical equivalence between the considered reduced model and the reference model. Models with a p-value lower than the significance level were immediately rejected, because in this case the null hypothesis itself should be rejected. Among all possible reduced models, the one with a minimum number of parameters and a higher p-value was accepted and is proposed in the paper. Note that the significance level was set to 0.10 to conservatively reduce the risk of a Type II error (i.e., not rejecting a false null hypothesis—in this case, accepting a reduced model with a statistically significant difference from the reference model). The selected predictors resulting from this procedure are those individuated within the discussion of the results of numerical analyses presented in Section 4.1.

Namely, AMP1 is related to the PFA/PGA of the specific floor, to the PGA value (to account for nonlinearity effects) expressed in g units and to the elastic uncracked vibration period of the bare structure $T_1$. AMP2 is related to $z/H$ (given that different higher modes may be predominant at different floors) and to the PGA value (to account for nonlinearity effects). Regarding the resonance periods, $T_{R1}$ is related to $T_1$ of the elastic uncracked bare frame and to the PGA value (to account for the effect of nonlinearity). $T_{R2}$ is related to $T_1$ of the bare frame, to the PGA value (to account for the effect of nonlinearity) but also to the height of the building $H$ and to $z/H$ of the specific floor (given that different higher modes may be predominant at different floors). Different predictive formulations are provided for bare and infilled buildings, as reported in Equations 24–27. The statistics of the observed-to-predicted ratios for the predicted variables are reported in Table 3.

\[
AMP1 = \begin{cases} 
      \min \left( 5.5, 0.99 \cdot \frac{\text{PFA}}{\text{PGA}}^{1.57} \cdot \text{PGA}^{-0.27} \cdot 0.11 T_1 \right) & \text{for bare buildings} \\
      \min \left( 5.5, 1.25 \cdot \frac{\text{PFA}}{\text{PGA}}^{1.27} \cdot \text{PGA}^{-0.29} \cdot 0.17 T_1 \right) & \text{for infilled buildings}
\end{cases},
\]

\[
AMP2 = \begin{cases} 
      \min \left( 5.5, 2.61 \cdot \text{PGA}^{-0.13} \cdot 0.89 \frac{z}{H} \right) & \text{for bare buildings} \\
      \min \left( 5.5, 2.52 \cdot \text{PGA}^{-0.13} \cdot 0.86 \frac{z}{H} \right) & \text{for infilled buildings}
\end{cases},
\]

\[
\frac{T_{R1}}{T_{1,\text{BARE FRAME}}} = \begin{cases} 
      1.08 \cdot 1.05^{\text{PGA}} & \text{for bare buildings} \\
      0.75 \cdot 1.97^{\text{PGA}} & \text{for infilled buildings}
\end{cases},
\]

\[
\frac{T_{R2}}{T_{1,\text{BARE FRAME}}} = \begin{cases} 
      2.21 \cdot H^{-0.52} \cdot \left( \frac{z}{H} \right)^{0.37} \cdot 1.20^{\text{PGA}} & \text{for bare buildings} \\
      0.82 \cdot H^{-0.17} \cdot \left( \frac{z}{H} \right)^{0.42} \cdot 0.84^{\text{PGA}} & \text{for infilled buildings}
\end{cases}.
\]

Although not perfectly rigorous, as explained in section 2, the formulation proposed by Circolare is adopted to express the variation of PSA/PFA in the two “resonance regions” as reported in Equations 28 and 29.

\[
\left( \frac{\text{PSA}}{\text{PFA}} \right)_1 = \min \left( \sqrt{\frac{1}{\text{AMP1} \frac{T_{R1}}{T_{R1}}}} + \left( 1 - \frac{T_{R1}}{T_{R1}} \right)^{-1} ; \text{AMP1} \right),
\]
\[ \left( \frac{\text{PSA}}{\text{PFA}} \right)_2 = \min \left( \left[ \sqrt{\left( \frac{1}{\text{AMP2}} \frac{T_a}{T_{R2}} \right)^2 + \left( 1 - \frac{T_a}{T_{R2}} \right)^2} \right] ; \text{AMP2} \right). \] (29)

The spectral shape is defined by the envelope of the spectral shapes determined in the two “resonance regions,” as reported in Equation 30 and shown in Figure 10.

\[ \frac{\text{PSA}}{\text{PFA}} = \max \left( \left( \frac{\text{PSA}}{\text{PFA}} \right)_1 ; \left( \frac{\text{PSA}}{\text{PFA}} \right)_2 \right) \] (30)

This is consistent with the fact that AMP1 and AMP2 were not determined by “separating” the contributions of different modes. In other words, the empirical values of AMP1 and AMP2 determined from the numerical results already account for the contribution of different modes; hence, combining (PSA/PFA)1 and (PSA/PFA)2 by means of a modal combination rule (eg, the SRSS rule) would not be correct.

The comparison between the numerical and predicted PSA/PFA spectral shapes for the longitudinal direction of a selection of case-study structures is shown in Figure 11.

It is observed that the proposed approach works quite well on a wide range of case-study structures, even if it may result overconservative or unconservative in some “extreme” cases (eg, for very low- or very high-rise buildings designed and analyzed for very low or very high PGA). Of course, this is expected since an empirical approach has been adopted when evaluating the maximum PSA/PFA ratio in the “resonance regions,” so, a certain prediction error is unavoidable. Within a code-based approach, it could be appropriate to assume a safety factor to amplify the predicted maximum PSA/PFA value, in order to have a prediction error always on the side of safety. Future studies may address this issue, also considering record-to-record variability and the uncertainty related to mechanical properties and modeling assumptions, especially regarding infill walls.

5.3 | Effect of damping and of potential nonlinear behavior of the secondary element

As above stated, the proposed formulations are given by calculating PSA/PFA spectral shapes with damping ratio of the secondary element, $\xi_a$, equal to 5%. However, nonstructural components may be characterized by different values of $\xi_a$. The specific value of $\xi_a$ could significantly affect the expected value of PSA/PFA: clearly, secondary elements with low $\xi_a$ are expected to experience higher values of PSA/PFA and vice-versa; no effect of $\xi_a$ is expected on PFA, of course.

For each case-study building, for each floor and for each direction, the observed values of AMP1, AMP2, $T_{R1}$, and $T_{R2}$ were re-evaluated for $\xi_a$ equal to 1%, 2%, 10%, 15%, and 20%. Then, the observed values were normalized with respect to those predicted by means of Equations 24–27 (ie, those predicted for $\xi_a = 5\%$). This has been done in order to define correction coefficients able to account for the effect of damping ratio on the PSA/PFA spectral shape. The trends of the average values of AMP($\xi_a$)/AMP($\xi_a = 5\%$) and of $T_{R1}(\xi_a)/T_{R1}(\xi_a = 5\%)$ with $\xi_a$ are shown in Figure 12.

It is observed that a clear decreasing trend relates both AMP1 and AMP2 to $\xi_a$, nearly independently on the infill layout. The correction coefficient $\eta_a$ reported in Equation 31 is proposed. It should be calculated with $\xi_a$ expressed as a percentage and used to increase/reduce both AMP1 and AMP2. The proposed formulation is analogous to the one proposed by Calvi and Sullivan14 and by Vukobratović and Fajfar15,16

\[ \eta_a = \sqrt{\frac{5}{\xi_a}}. \] (31)

On the other hand, the effect of the damping of the secondary element on $T_{R1}$ and $T_{R2}$ appears quite negligible. Hence, no correction is proposed. Also, the PSA/PFA spectral shape could be significantly affected by the nonlinear response of the secondary element as shown, among others, by Vukobratović and Fajfar15,16 This effect has not been considered in this study. It is suggested that the PSA values obtained from the proposed formulations are reduced when a nonlinear response of the secondary element is expected. To select appropriate values of the PSA reduction factor, the reader could refer, for example, to the values of the behavior factor proposed for nonstructural components by codes5 or by the literature26, for example, for the out-of-plane assessment of unreinforced masonry infill walls.
Applicability range of the proposed formulation. Potential application to existing reinforced concrete framed buildings

The proposed approach for the assessment of floor response spectra is empirical and based on the results of numerical analyses. Hence, it is fully applicable on reinforced concrete buildings similar to those analyzed in this study, while every
application on buildings significantly different from those analyzed in this study should be performed with caution (e.g., for irregular structures, for structures with different structural systems—for example, reinforced concrete wall structures or dual structures—for tall structures with more than eight stories, for infilled reinforced concrete structures with irregular distribution of infill walls—e.g., pilotis structures).

The proposed approach has been calibrated based on the assessment of reinforced concrete structures designed according to Eurocodes. It has been shown, among other effects, that the higher the PGA demand at Life Safety Limit State, the higher the reduction of maximum PFA and PSA demand due to nonlinearity, mainly due to elements’/infills’ cracking. Most likely, in an existing building not designed according to current seismic provisions (or not designed to seismic action at all), at equal PGA demand currently associated with the assessment at Life Safety Limit State, an even higher nonlinearity is expected with respect to a new building, not only due to elements’ cracking, but also due to their yielding. This could further limit PFA and PSA demand with respect to that predicted according to the approach herein proposed. In other words, the approach described in this section is expected to be conservative (and then, potentially, applicable) for existing reinforced concrete framed buildings. However, further investigation is needed regarding this issue.

6 | CONCLUSIONS

Floor response spectra are a useful tool for the assessment of the seismic demand on nonstructural components, whose safety check is a paramount issue in performance-based earthquake engineering. This study was dedicated to the assessment via nonlinear time-history analyses of floor response spectra of 2-, 4-, 6-, and 8-story case-study reinforced concrete framed structures designed according to Eurocodes for four different levels of design peak ground acceleration at Life Safety Limit State (0.05, 0.15, 0.25, and 0.35 g). The 16 case-study bare structures have also been analyzed by including in the model two different kinds of uniformly-distributed infill walls. The main novelty of this study is not only the assessment of floor spectra for infilled reinforced concrete buildings, which has been rarely addressed in the past, but above all the fact that infill walls are modeled by considering their in-plane response, as usual, and their out-of-plane response, as well as the in-plane/out-of-plane interaction, that is, the degradation of the in-plane response due to the out-of-plane seismic demand and vice-versa.

First, after an introduction on the concept of floor spectrum, the main code prescriptions and literature proposals have been presented and discussed. Based on these formulations, the main parameters influencing floor response spectra have been identified, namely the floor height normalized with respect to the building height, the pseudo-spectral accelerations acting on the considered building, the vibration periods of the considered building and the shape of its vibration modes, the building height, the expected PGA demand. Moreover, floor spectra shape is strongly influenced by higher vibration modes and by the nonlinearity demand experienced by the structure. Also, it was observed that the in-plane/out-of-plane interaction in infill walls makes the floor response spectra of infilled buildings intermediate between those predicted for bare structures and those predicted for infilled structures when only the in-plane response of infills is modeled.

The expected influence of these parameters on floor spectra has been confirmed by the results of the numerical analyses and by comparing these outcomes with the prediction of code prescriptions and literature formulations. In addition, it has
been observed that among the available approaches, the one by New Zealand code NZSEE2017 is the most appropriate for the assessment of the peak floor acceleration distribution along the building height, while the one by Calvi and Sullivan is the most appropriate for the assessment of the floor spectra shapes at different floors, together with the simplified approach proposed by the commentary to the Italian technical regulation (Circolare), which may be, however, quite conservative.

Based on the above discussion, formulations for the assessment of floor response spectra at design level of seismic demand have been proposed for both bare and infilled buildings. The proposed formulations, which account for the effect of higher vibration modes, of structural nonlinearity and of the damping ratio of the secondary element, are characterized by ease-of-use, since they are applicable by knowing basic geometric and dynamic properties of the bare structures, and by a satisfactory matching with the outcomes of the numerical analyses.

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