SEISMIC RELIABILITY OF ACCELERATION-SENSITIVE ANCILLARY ELEMENTS IN THE NEW GENERATION OF EUROCODES

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Abstract: *The seismic performance of nonstructural/ancillary elements plays a decisive role in the seismic resilience of both ordinary buildings and critical industrial and infrastructure facilities, since damages that are likely to be sustained by such components can undermine the functionality and safety of an otherwise structurally intact structure. Owing to the above, the new generation of Eurocodes invests a great deal of effort towards prescribing appropriate provisions for delivering anchoring systems of acceleration-sensitive nonstructural components that can withstand the (often greatly) amplified floor accelerations with respect to the ground ones. In particular, prEN 1998-1-2:2022 offers a very detailed methodology for estimating the acceleration that is eventually imposed at the component level, accounting for several dynamic attributes of the primary system and the nonstructural component, which are yet not always trivial to determine with an appropriate level of confidence. This paper investigates to what extent the reliability of a code-conforming nonstructural component can be affected by the uncertainties associated with the assumptions made during design. The focus is on the relation of the period of the component to the period of the supporting building using a typical industrial building-type structure as a case-study. On account of the findings that revealed a rather significant sensitivity of the final design product to these uncertainties, the study extended its scope towards reviewing two alternative design methodologies that are offered in prEN 1998-4:2022 for the design of ancillary elements. These two latter approaches are shown to be less sensitive to the designer's input and can offer more robust designs for the anchorage systems of nonstructural components that are close to tuning with the frequencies of the primary structure.*

1. Introduction

Nonstructural elements can be discretised into two main categories, according to the failure mode to which they are prone (FEMA, 2020): (a) drift-sensitive ones, referring to those components/anchorages that are likely to sustain damage due to excessive interstorey drift demands imposed to the supporting structure (e.g., piping spanning across the building height) or (b) acceleration-sensitive ones, which are prone to sustaining damage due to excessive acceleration demands (e.g., vessels, heat exchangers, server racks) developed in response to the floor/ground motion imposed at their base. A number of components can be also classified into both categories (Taghavi and Miranda, 2003).

Safeguarding the seismic integrity of drift-sensitive equipment and their attachment points requires accounting for the deformation response of the supporting structure, while the equipment itself is assumed to conform to supporting-structure deformations. Contrarily, the design of acceleration-sensitive components and their anchorage system is more complex. This is due to the acceleration demands that are imposed at the component level, and eventually at the anchorage points, being highly dependent on (i) the dynamic characteristics (i.e., period, damping, mode shapes) and the response (linear or nonlinear) of the supporting structure, (ii) the dynamic characteristics (i.e., natural period and damping) and the ductility of the nonstructural component, and (iii) the location/height where the component is attached (e.g., Igusa and Der Kiureghian, 1985; Adam and Fotiu, 2000; Taghavi and Miranda, 2003; Sankaranarayanan and Medina, 2007; Vukobratović and Fajfar, 2016; Di Domenico *et al*., 2021; Ding *et al.*, 2024).

The heavily overdesigned industrial buildings located in critical infrastructure facilities (e.g., oil refineries) are rife with ancillary elements, such as heat exchangers, vessels, mechanical and electrical equipment, which are special nonstructural components largely governing the overall operational and seismic performance of an industrial plant. To this end, over the past few years, significant effort has been put towards developing a design framework for ancillary elements that are either supported or nested in buildings, so as to appropriately treat the vast uncertainties associated with their capacity and demand evaluation. In view of the above, the New European Bauhaus for verifying the satisfactory seismic performance of nonstructural/ancillary elements (CEN, 2022a; CEN, 2022b):

• Exploits past evidence for the seismic acceleration demands imparted on acceleration-sensitive equipment, suggesting that these could be significantly amplified from floor to component level, especially at the upper building floor levels and for components that are either tuned or almost tuned to one of the predominant periods of the supporting building—which is often the case for the short period components that are nested in stiff industrial structures.

• Offers three alternative design routes; a detailed component/structure-specific design route that accounts for all pertinent component and building characteristics, a conservative approach in which the nonstructural component is designed as always being at resonance with the supporting structure, and a ductile design route that involves the utilisation of a sacrificial fuse of verified ductility and strength in the component-building load path to limit the imposed seismic demands.

Hence, the engineer is asked to select one of the aforementioned methods to design the anchorage system of an ancillary element. This decision should be based upon three aspects: (a) the level of knowledge and the quality of the available data for the structure-ancillary element system dynamic properties, (b) the type of the element's anchorage system and in particular whether this will remain elastic or its ductility and overstrength can be certified, allowing for a fully dissipative design to limit the acceleration demands that are imparted at the component level, and (c) the existence of any manufacturer acceleration limits that should not be exceeded for a vibration-sensitive electromechanical equipment to remain functional.

2. Eurocode 8 design methodologies

The seismic design of nonstructural components requires a great deal of knowledge on the vibration characteristics of the primary (supporting) structure and the secondary system (nonstructural/ancillary element) in order to attain an acceptable level of risk. In particular, the seismic ground acceleration that is imposed to the building base undergoes two modulations, which involve selective amplitude amplification (see Figure 1) due to: (a) its dynamic filtering by the vibration modes of the supporting structure and (b) the flexibility of the component. Both potential amplifications can result in resonance if the period of the underlying soil matches the fundamental periods of the structure, or if the component is tuned or nearly tuned to the period of the supporting structure (Goel, 2018); the latter is of primary interest here. Knowledge is also required on the component damping level (Kazantzi *et al*., 2020a), as well as on the position of the component along the building height, since floor and component acceleration demands generally increase with the floor height (NIST, 2017).

In view of the above, the provisions of Eurocode 8 (current version under public inquiry) offer three different design methods for ancillary elements and their attachment to the supporting structure. These methods require various levels of data for the supporting structure and the ancillary element. In particular:

• Method 1 (Vukobratović and Fajfar, 2023) is presented in section 7 and Annex C of prEN 1998-1-2:2022 (CEN, 2022a) and requires for its implementation a high level of knowledge regarding the modal characteristics of the supporting structure and its nested/supported equipment.

• Method 2 is the non-dissipative design approach presented in section 9 of prEN 1998-4:2022 (CEN, 2022b). The designer is considered to have imperfect knowledge of the modal characteristics of the supporting structure and/or the ancillary element, with the latter conservatively assumed to be tuned to the vibration period of the supporting structure.

• Method 3 is the dissipative design approach presented in section 9 of prEN 1998-4:2022 (CEN, 2022b), where, similarly to Method 2, limited knowledge of the modal characteristics of the structure-element system is considered. In this method, certain components of the element's anchorage system are allowed to yield in a ductile manner for energy dissipation.

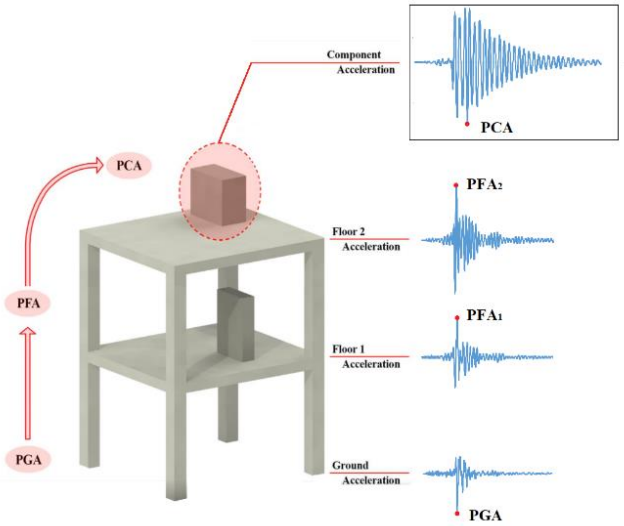

Figure 1. Amplification of the ground acceleration at the floor and component level (adopted from Kazantzi et al., 2024).

2.1. Method 1: Design approach per prEN 1998-1-2:2022

The design horizontal seismic force F_{ap} of an ancillary element residing at floor j of a structure may be determined after prEN 1998-1-2:2022 (CEN, 2022a) as adapted for use in prEN 1998-4:2022 (CEN, 2022b):

$$F_{\mathrm{ap}} = \frac{\gamma_{\mathrm{ap}} \cdot m_{\mathrm{ap}} \cdot S_{\mathrm{ap},j}}{q_{\mathrm{ap}}{'}} \tag{1}$$

where γ_{ap} is the performance factor of the element, taking values equal to 1.0 or 1.5 for components non-participating or participating in safety-critical systems, respectively, unless otherwise instructed by a relevant authority or National Annex; m_{ap} is the mass of the ancillary element; $q_{\mathrm{ap}}{'}$ is the period-dependent behaviour factor of the ancillary element estimated after Annex C of prEN 1998-1-2:2022 (CEN, 2022a), but limited to a maximum value of 1.5 per prEN 1998-4:2022 (CEN, 2022b); $S_{\mathrm{ap},j}$ is the value of the floor acceleration spectrum in the considered horizontal direction at floor j at the natural period of the ancillary element T_{ap}, and for a critical damping ratio for the ancillary component of ξ_{ap}.

If the floor response spectra are not available (e.g., response-history analysis has not been conducted) and the ancillary element cannot be considered as rigid, the floor acceleration spectrum $S_{\mathrm{ap},j}$ is evaluated according to the provisions of Annex C as:

$$S_{\mathrm{ap},ij} = \frac{\Gamma_i \cdot \varphi_{ij}}{\left|\left(\frac{T_{\mathrm{ap}}}{T_{\mathrm{p},i}}\right)^2 - 1\right|} \sqrt{\left(\frac{S_{\mathrm{ep},i}}{q_{\mathrm{D}}{'}}\right)^2 + \left[\left(\frac{T_{\mathrm{ap}}}{T_{\mathrm{p},i}}\right)^2 \cdot S_{\mathrm{eap}}\right]^2} \leq AMP_i \cdot \left|PFA_{ij}\right| \tag{2}$$

where Γ_i is the modal participation factor for the i^{th} mode of the supporting structure in the direction of interest; φ_{ij} is the i^{th} mode shape value of the supporting structure at the j^{th} floor; $T_{\mathrm{p},i}$ is the natural period of the i^{th} mode of the supporting (primary) structure; $S_{\mathrm{ep},i}$ is the elastic spectral acceleration S_{e} evaluated for the supporting structure at $T_{\mathrm{p},i}$ and $\xi_{\mathrm{p},i}$ that is obtained from the elastic response spectrum after prEN 1998-1-1:2021 (CEN, 2022c); $\xi_{\mathrm{p},i}$ is the critical damping ratio (in %) of the i^{th} mode of the supporting (primary) structure that is equal to 5% (regardless of the lateral-load resisting system) for a building structure; S_{eap} is the elastic spectral acceleration S_{e} evaluated for the ancillary element at T_{ap} and ξ_{ap} that is obtained from the elastic (ground) response spectrum after prEN 1998-1-1:2021 (CEN, 2022c); AMP_i is the amplification factor that is evaluated via Eq.(3):

$$AMP_i = \begin{cases} 2.5 \cdot \sqrt{\dfrac{10}{(5+\xi_{\mathrm{ap}})}}\ , & \dfrac{T_{\mathrm{p},i}}{T_{\mathrm{C}}} = 0 \\ \text{linear between } AMP_i\left(\dfrac{T_{\mathrm{p},i}}{T_{\mathrm{C}}} = 0\right) \text{ and } AMP_i\left(\dfrac{T_{\mathrm{p},i}}{T_{\mathrm{C}}} = 0.2\right), & 0 \le \dfrac{T_{\mathrm{p},i}}{T_{\mathrm{C}}} \le 0.2 \\ \dfrac{10}{\sqrt{\xi_{\mathrm{ap}}}}\ , & \dfrac{T_{\mathrm{p},i}}{T_{\mathrm{C}}} \ge 0.2 \end{cases} \tag{3}$$

and PFA_{ij} is the peak floor acceleration in the considered horizontal direction at floor j and for mode i, which is evaluated as:

$$PFA_{ij} = \Gamma_i \cdot \varphi_{ij} \cdot \frac{S_{\mathrm{ep},i}}{q_{\mathrm{D}}'} \tag{4}$$

where q_{D}' is a period-dependent behaviour factor that characterises the primary structure, being defined as:

$$q_{\mathrm{D}}' = \begin{cases} 1.0 & T_{\mathrm{p},1} \le T_{\mathrm{A}} \\ \text{linear between } 1.0 \text{ and } q_{\mathrm{D}} & T_{\mathrm{A}} \le T_{\mathrm{p},1} \le T_{\mathrm{C}} \\ q_{\mathrm{D}} & T_{\mathrm{p},1} \ge T_{\mathrm{C}} \end{cases} \tag{5}$$

with T_{A} being the short period cut-off associated to the zero-period spectral acceleration, T_{C} being the upper corner period of the constant spectral acceleration range of the elastic response spectrum of prEN 1988-1-1:2021 (CEN, 2022c), and q_{D} is the building behaviour factor accounting for deformation capacity and energy dissipation capacity, as determined by the ductility class considered during the design of the structure. For use in industrial structures, a stricter approach is employed to determine q_{D}', typically limiting it to 1.0 if no verification of overstrength is undertaken.

2.2. Method 2: Non-dissipative design approach per prEN 1998-4:2022

Apparently, Method 1 requires a high level of knowledge with regards to the properties of the supporting structure and the nonstructural component, which are often not readily available to the engineer undertaking the design of the ancillary elements. To work around this actual problem, a non-dissipative design method (denoted as Method 2 hereinafter) has been adopted in prEN 1998-4:2022 (CEN, 2022b) in which the acceleration applied at the component level, S_{ap}, is defined as:

$$S_{\mathrm{ap}} = AMP \cdot PFA \tag{6}$$

where AMP is an amplification factor that takes a constant value equal to 7, essentially implying a resonance condition between the component and the supporting structure, and PFA is the peak floor acceleration corresponding to the fundamental mode of vibration, computed as:

$$PFA = \Gamma_1 \cdot \varphi_{1,\mathrm{ap}} \cdot \frac{S_{\mathrm{e}}(T_{\mathrm{p},1}, \xi_{\mathrm{p},1})}{q_{\mathrm{D}}'} \ge \frac{S_{\alpha}}{F_{\mathrm{A}}} \tag{7}$$

where Γ_1 is the participation factor of the fundamental mode in the direction of interest, which, in the absence of more accurate data, can take a value of 1.5 for the majority of the supporting structures, except for tanks and silos where a value of 1.8 is recommended; $\varphi_{1,\mathrm{ap}}$ is the fundamental mode shape amplitude at the height z of the supporting structure where the component is attached. If a linear distribution is assumed over the total height H of the supporting structure, then it may be evaluated as $\varphi_{1,\mathrm{ap}} = \left(\frac{z}{H}\right)$, with z measured from the ground level. Then, $S_{\mathrm{e}}(T_{\mathrm{p},1}, \xi_{\mathrm{p},1})$ is the elastic response spectral acceleration at the fundamental period $T_{\mathrm{p},1}$ of the supporting structure in the considered direction and the corresponding damping ratio $\xi_{\mathrm{p},1}$. The acceleration $S_{\mathrm{e}}(T_{\mathrm{p},1},\xi_{\mathrm{p},1})$ is subject to a lower bound equal to the elastic response spectral acceleration corresponding to 0.5sec. Also, q_{D}' is a period-dependent primary-structure behaviour factor [see Eq. (5)], that for structures where there is uncertainty about the q_D value or no verification of the actual overstrength has been undertaken may be taken equal to 1.0. Finally, S_α is the maximum response spectral acceleration (5% damping) corresponding to the constant acceleration range of the horizontal elastic response spectrum and F_{A} is the ratio of the maximum response spectral acceleration (for 5% damping) corresponding to the constant acceleration range of the elastic response spectrum over the zero-period spectral acceleration, often taken equal to 2.5, unless otherwise set by the National Authorities.

2.3. Method 3: Dissipative design approach per prEN 1998-4:2022

The code provisions of prEN 1998-4:2022 (CEN, 2022b) allow, also, for a dissipative design approach. Sufficient evidence for the relaxation in the imposed component acceleration demands should a yielding element be inserted between a nonstructural component and the supporting system is provided in Kazantzi *et al.* (2020b; 2020c; 2022a; 2023) and Elkady *et al*. (2022). In that case, the design horizontal seismic force, F_{ap}, of the fuse may be determined as:

$$F_{ap} = m_{ap} \cdot S_{ap} \tag{8}$$

with S_{ap} being computed after Eq. (6). All other elements within the load path from the component to the supporting structure should have at least a 25% overstrength with respect to the fuse strength. In addition, the maximum force (and acceleration) transmitted to the component per Eq. (8), including any fuse overstrength, should not exceed the respective component capacity. The amplification factor AMP in Eq. (6) is now evaluated as:

$$AMP = \max\left\{1.30;\ 0.60 + \frac{1.40}{(\mu_D - 1.0)}\right\} \tag{9}$$

where μ_D is the certified fuse ductility with $1.50 \le \mu_D \le 3.00$. The cyclic ductility capacity of the fuse should be verified either experimentally by means of cyclic tests or otherwise, and it should be at least equal to $\mu_D \cdot \gamma_{ap}$.

3. Case-study building

A typical industrial equipment-supporting building is considered to present and evaluate the alternative design methods for ancillary elements that were detailed in Section 2. The aim is to shed light onto these methods by revealing the impact of the underlying assumptions, as well as of the uncertainties emerging from the engineer's choices on the properties of the nonstructural elements and the supporting structure. The case-study building is an open-frame reinforced concrete (RC) moment resisting frame (Figure 2), which was adopted from Kazantzi *et al.* (2022b).

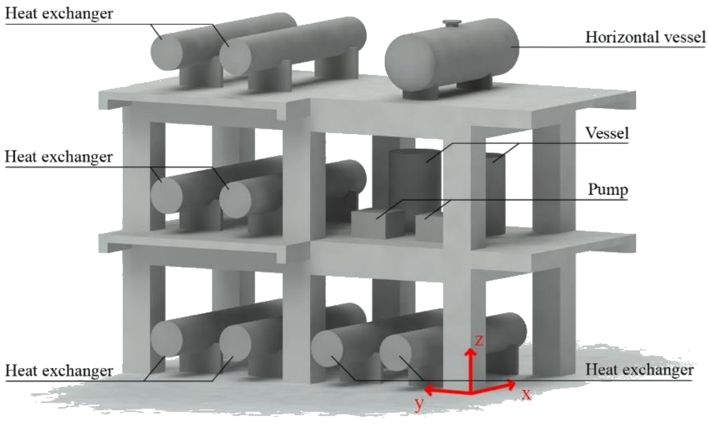

Figure 2. Photorealistic representation of the examined RC building with indicative nested equipment that can be found in an oil refinery (adapted from Kazantzi et al., 2024).

This typical refinery building was designed for Zone 3 of Greece according to the new seismic hazard zonation proposed by Pitilakis *et al*. (2022). Zone 3 corresponds to $S_{\alpha,\mathrm{ref}} = 0.71\mathrm{g}$ for a return period of 475 years. For the case at hand the acceleration is amplified by a performance factor of 1.75 per prEN 1998-4:2022 (CEN, 2022b) for Consequence Class 3a (i.e., buildings whose seismic resistance is of importance in view of the consequences associated with collapse) and the Near Collapse (NC) damage state, resulting to $S_{\alpha,\mathrm{ref}} = 1.24\mathrm{g}$ for 2,500 years. Detailing compatible with a Ductility Class 2 structure has been assumed. Note that compliance with non-seismic design provisions (especially fireproofing) means that such industrial structures are heavily overdesigned, well beyond what seismic loading would require. Hence, no or at worse minor structural damage is anticipated even during strong ground motions. Owing to the above, a 3D elastic model has been adopted for the supporting structure.

The developed 3D elastic model of the RC building was subjected to 30 "ordinary" (i.e., non-pulse-like, non-long-duration) natural ground motion records, which were selected by Bakalis *et al*. (2018). The floor acceleration histories were recorded at the anchorage points of the nested equipment at both the 1st and the 2nd floor. The equipment was accounted for in the 3D model only via point masses, essentially disregarding any component-structure interaction. This assumption is valid only for components with mass that is not substantial compared to the mass of the supporting structure. A more elaborate discussion with regards to this issue may be found in Kazantzi *et al*. (2022b).

The computed floor acceleration histories at the anchorage points were used as input to eventually estimate the maximum seismic demands that are induced at several components with different dynamic characteristics. The demands were computed on the basis of time-history analyses of a linear (for Methods 1 and 2) and an elastic-perfectly-plastic (for Method 3) single-degree-of-freedom (SDOF) oscillator (see Figure 3). The component demands were then compared with the component capacities (in fact the capacities of their anchorage system) having the latter evaluated following the provisions of the three design methods of Section 2. A performance factor γ_{ap} equal to 1.5 has been assumed, since the considered components are part of a safety-critical system. No additional overstrength in the evaluated capacities, other than the overstrength that is recommended by the provisions of Eurocode 8, was accounted; a condition that renders the findings of this study somewhat conservative, yet uniformly so among the different design methods.

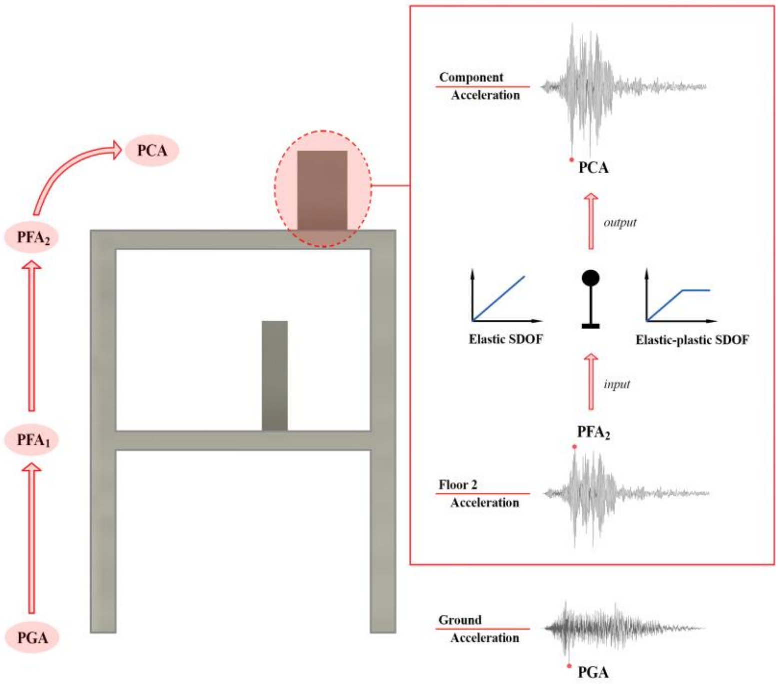

Figure 3. Graphical outline explaining how the component seismic demands were evaluated in the present study for the dissipative and the non-dissipative ancillary elements (PGA: Peak Ground Acceleration, PFA: Peak Floor Acceleration, PCA: Peak Component Acceleration).

4. Seismic fragility study

To allow a meaningful comparison of the three different design methodologies that were outlined in Section 2, analytical fragility curves for several nested components of varying periods at both floor levels of the case-study building were computed. The comparison of the fragilities essentially allows viewing from a probabilistic standpoint how code-conforming ancillary elements perform if designed on the basis of the three available Eurocode 8 methodologies. The fragility curves have been expressed in terms of the geometric mean PGA as the intensity measure (IM). In particular, the component fragility curves were obtained under the typical lognormality assumption (Cornell *et al.*, 2002):

$$P(D > C|PGA = pga) = \Phi\left(\frac{ln(\widehat{D}(pga)) - ln(\hat{C})}{\beta_{\text{tot}}}\right) \tag{10}$$

where $\widehat{D}(pga)$ is the median component acceleration demand evaluated for a given $PGA = pga$ level, $\hat{C}$ is the median design acceleration capacity of the component evaluated via one of the three design methodologies, and β_{tot} is the total lognormal dispersion for the PGA level considered. Herein, only demand dispersion was considered, essentially discarding any capacity variability across all methods.

4.1. Component fragilities for ancillary elements designed to Method 1

The design procedure of Method 1 for estimating the design capacity of a component utilises as input several dynamic characteristics (i.e., periods, mode shapes, participation factor, behaviour factors) of both the ancillary elements and the supporting building, as outlined in Section 2.1. The most important input elements are the period of the component T_{ap} and the periods of the supporting primary structure $T_{\text{p},i}$. The former matching any of the latter essentially defines whether the component will be a tuned or untuned one.

The obtained fragility curves having the component capacities evaluated via Method 1 are presented in Figure 4. Indicatively, the results are shown for the Y direction of the building (see Figure 2) whereas the fragilities were computed for several components of variable period and for both floors of the supporting structure. Specifically, ten (virtual) components with ratios of $T_{\text{ap}}/T_{\text{p},1}$ within the range of 0.25 to 2.50 were investigated. Evidently, the most fragile components are those tuned to the predominant vibration period of the supporting building ($T_{\text{ap}}/T_{\text{p},1} = 1.00$).

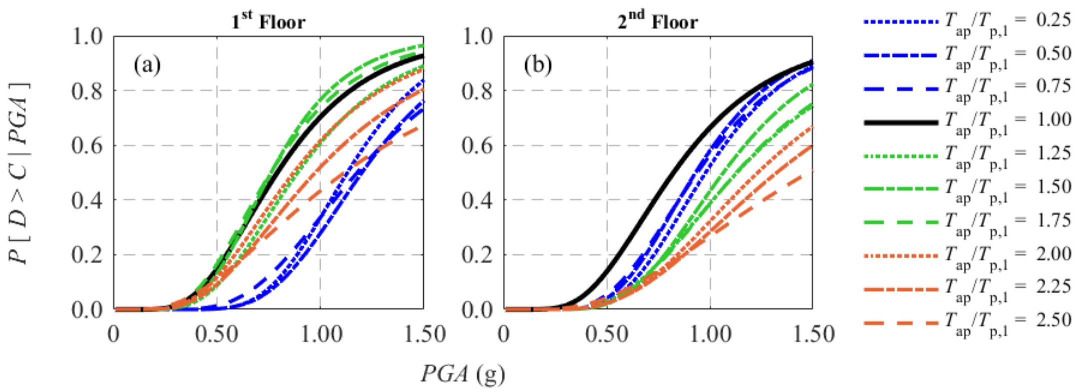

Figure 4. Component fragility curves computed in the Y direction at the (a) 1st and (b) 2nd floor of the case-study building, obtained for ancillary elements designed to Method 1 and having ten different period ratios of $T_{ap}/T_{p,1}$ *(adapted from Kazantzi et al., 2024).*

To obtain the fragilities presented in Figure 4, a perfect knowledge level was assumed with respect to the periods of structure and component. Hence, in each case, the component was designed per Method 1 and its capacity was compared to the elastic floor spectral acceleration derived via (linear) response history analysis. Always, the exact same component period T_{ap} was used to assess demands as well as to determine its capacity, implying a wealth of information on the facility to be constructed and the components to be installed. Thus, any pertinent uncertainties were neglected. However, such level of information is rarely readily available to the designer of the component anchorage, since the design of the latter is usually performed by engineering firms that are different from those that were involved in the design of the supporting structure and in some cases, this means that also limited information may be available on the dynamic characteristics of the structure itself.

To investigate the robustness of Method 1 against inaccurate assumptions that could be made during the design process, a sensitivity analysis was undertaken. For simplicity, only the uncertainty due to the period of the component (T_{ap}) is accounted for. Yet, the findings hold for the case where the uncertainty is associated with the period of the building or both, since we are mostly interested in their relative values rather than their absolute ones. The procedure that was followed involved taking six component $T_{ap,cap}$ values—i.e., assumed component periods for evaluating the design component acceleration capacity by means of Method 1—over building $T_{p,1}$ period ratios. The component acceleration demands were then evaluated assuming that the actual component period (T_{ap}) is different to the one employed at the design stage ($T_{ap,cap}$), being 5/10/20% higher or lower.

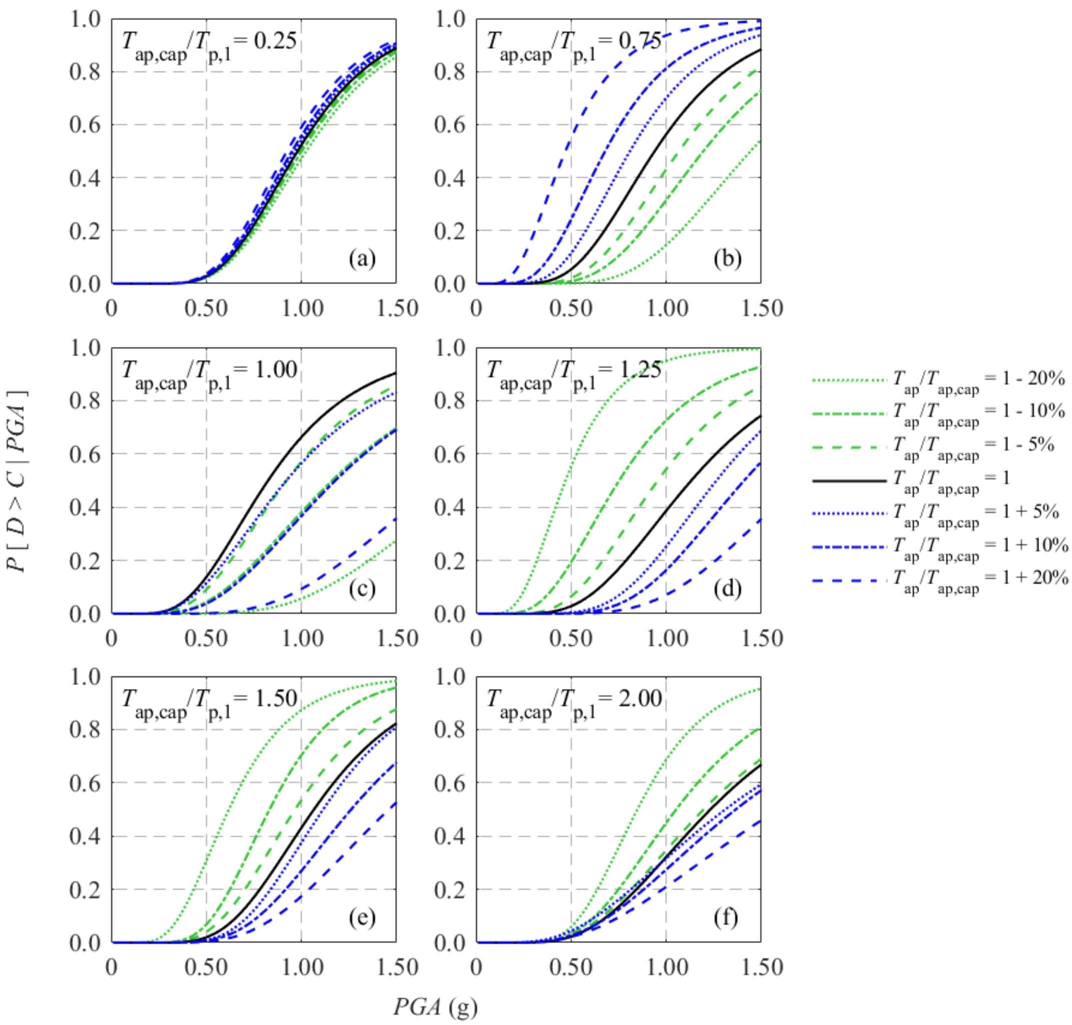

Figure 5. Fragility-based sensitivity analysis for components designed per Method 1 (indicatively presented for the 2nd Floor of the case-study building; adapted from Kazantzi et al., 2024).

Figure 5 presents the fragilities that were computed under the aforementioned assumptions. As can be inferred by inspecting the fragilities illustrated in Figure 5, even small deviations from the period assumed during the design of the component could undermine the reliability of an otherwise code-conforming nonstructural element. The only exception to this conclusion is the case in which the component was designed as being tuned to the period of the primary structure (see Figure 5c). Therefore, one could claim that Method 1 performs consistently well when the designer has a good level of knowledge about the actual periods of the component and the supporting structure. Contrarily, it is likely to render unconservative designs in several cases, if the periods of the component and/or the structure deviate from the actual values in a way that brings them closer to tuning, when originally no resonance was assumed.

4.2. Component fragilities for ancillary elements designed to Method 2

Method 2 is essentially a simpler version of Method 1 for non-dissipative design, where the design component acceleration (design PCA) is always computed on the basis of resonance—where a maximum amplification factor of $AMP = 7$ is adopted. As can be inferred from the component fragilities that are presented in Figure 6, Method 2 yields for the detuned components consistently conservative designs, regardless of their period. If one considers that the cost of even a heavily overdesigned anchorage system is trivial compared to the overall value of a critical facility, its functionality, and safety, then Method 2 offers some considerable advantages over Method 1 for practical design applications: By virtue of being period-agnostic, it nullifies by default any bias associated with the period estimation for both the component and the supporting building.

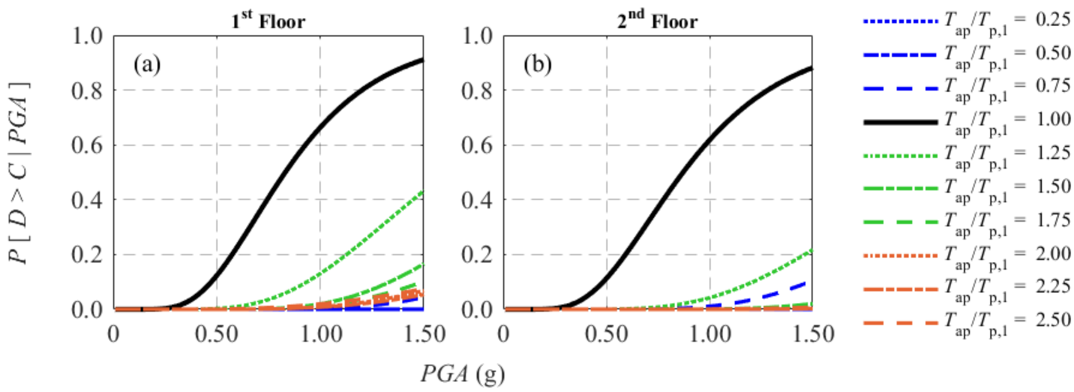

Figure 6. Component fragility curves computed in the Y direction at the (a) 1st and (b) 2nd floor of the case-study building, obtained for ancillary elements designed to Method 2 and having ten different period ratios of $T_{ap}/T_{p,1}$ (adapted from Kazantzi et al., 2024).

4.3. Component fragilities for ancillary elements designed to Method 3

Method 3 goes one step beyond Method 2 for alleviating its conservatism associated with designing a nonstructural component as potentially tuned to the period of the supporting building. This is achieved by introducing a fuse of guaranteed ductility and strength in the load path, thus removing the effect of resonance and tying the amplification factor of the peak floor acceleration to the yielding fuse ductility [see Eq. (9)]. This sacrificial fuse is essentially an element of the anchorage system, explicitly designed and verified to develop a controlled yielding mechanism should the seismic force (or acceleration) exceed a predetermined level. The end effect of allowing the fuse to undergo inelastic deformation is the substantial reduction of the accelerations that are imparted to the component, even under the persistent design condition that the component is tuned. In fact, as it was showcased both analytically and experimentally (e.g., Kazantzi *et al*., 2020c; Elkady *et al*., 2022; Kazantzi *et al*., 2023), if nonlinearity is permitted at the component level, the strong narrow-band amplification effect of the floor spectra is substantially limited, even in the vicinity of the tuning range and even for small inelastic displacements.

Figure 7 illustrates the component fragility curves that were obtained by having the component capacities evaluated via Method 3, considering two fuse ductility levels, i.e., $\mu_{\mathrm{D}} = \{1.5; 2.5\}$. Note that such values are only nominal, meant to be used for determining AMP per Eq. (9), with actual ductilities being $\gamma_{\mathrm{ap}} = 1.5$ times higher per the design requirements of the case-study. As can be inferred by inspecting Figure 7, Method 3 yields component fragilities that are slightly safer than those of Methods 1 and 2 at resonance, yet of considerably more reasonable (i.e., lesser) conservatism for detuned components when compared to the ultra-conservative Method 2. Moreover, it offers one less obvious but equally important advantage: Components designed by Method 3 eventually sustain considerably lower accelerations, limited by the fuse yield strength. Currently, the only missing link for the widespread application of Method 3 is the limited availability of anchoring products with verified ductility and strength. Nevertheless, this is an issue to be resolved by manufactures who wish to offer products of superior and guaranteed seismic performance. Otherwise, Methods 1 and 2 are the only alternatives.

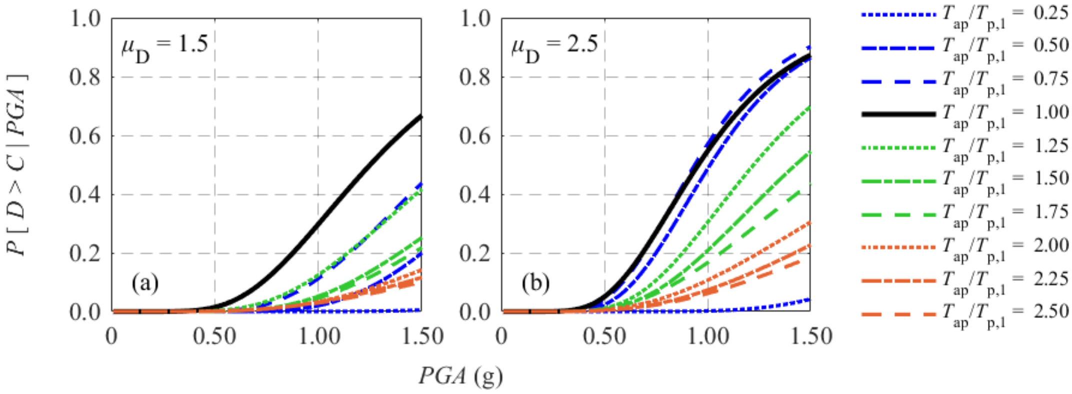

Figure 7. Component fragility curves computed for the 1st Floor in the Y direction, obtained for ancillary elements designed to Method 3 for two indicative fuse ductility levels. Note that the nominal ductility capacity μ_D is reported in the figures, whereas the actual ductility capacity is $1.5 \cdot \mu_D$ (adapted from Kazantzi et al., 2024).

5. Conclusions

In the new generation of Eurocode 8, three different methodologies are offered for the seismic design of acceleration-sensitive ancillary elements in industrial facilities. For applying each of these methods, different levels of knowledge with regards to the dynamic properties of the supporting structure and the nested ancillary elements are needed. In view that the required input for the most detailed design route (i.e., Method 1) is often neither readily available nor easily acquired for practical design applications, the present study reviews analytically computed component fragilities for ancillary elements designed to the different Eurocode 8 methods and investigates how the uncertainties associated with the required input could propagate and affect the final design products. It was showcased by means of an analytical seismic fragility assessment that the nonstructural component design method in Eurocode 8 – Part 1-2 can deliver robust designs in those cases where the designer has a high level of knowledge with regards to the dynamic properties of the supporting structure and the nonstructural component. However, if such a high level of knowledge is not the case, it was demonstrated that even small discrepancies of the assumed properties from their actual values can severely undermine the seismic reliability of an otherwise well-designed code-conforming nonstructural element. By contrast, the two additional methods that are offered in Eurocode 8 – Part 4, namely the non-dissipative and the dissipative approach, are less sensitive to the uncertainties associated with the needed input, since conservative assumptions are made, with the most important being that the nonstructural component is always designed as being tuned to the supporting structure. It was also demonstrated that the method allowing for certain fuses in the anchorage system—of verified ductility and strength—to go inelastic, could provide consistently reliable and less conservative final designs that are also subjected to substantially lower accelerations compared to those that result from the other design routes.

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