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DESIGN OF ACCELERATION-SENSITIVE ANCILLARY ELEMENTS UNDER UNCERTAINTIES IN THE NEW EUROCODE

Athanasia KAZANTZI[1,2], Nikolaos KARAFERIS[3], Vasileios MELISSIANOS[4], Dimitrios VAMVATSIKOS[2,5]

Abstract: *The overall seismic safety and operability of industrial building-type structures located in critical infrastructure facilities, largely depend on the seismic performance of their nested and/or supported ancillary elements, namely mechanical and electrical equipment, machinery, vessels, etc. Hence, on account that (a) no or minimal direct structural damages are anticipated in the equipment-supporting structures per se during moderate or even strong earthquake events since such structures are typically overdesigned and (b) the sustained structural damages are mostly due to the inferior seismic performance of the nested ancillary elements that could trigger a series of adverse cascading incidents (e.g., uncontrolled fires, explosions), significant effort has been invested towards developing a design framework that could deliver safe designs for the latter. Despite the significant advancements in the relevant field, the development of a robust design framework is often undermined by several uncertainties that come into play in the evaluation of the capacity and demand of such nonstructural components. In particular, critical information that is needed for the design of the ancillary elements, such as the dynamic characteristics of the component and the supporting structure, are often abstract and/or require substantial effort for being retrieved with certain confidence. On that basis, the new Eurocode 8, offers three distinct design options that allow for adjustments in the conservatism that is induced in the design of the acceleration-sensitive ancillary elements according to the availability and reliability of information on the overall system. This study investigates, by means of a case study industrial structure, the extent to which the seismic reliability of an otherwise code-compatible component designed to comply with each one of the three alternative Eurocode design routes, is likely to be undermined for small discrepancies of the assumed properties from their actual values.*

Introduction

The heavily overdesigned building-type industrial structures located in critical infrastructure facilities are rife with ancillary elements, such as heat exchangers, vessels, mechanical and electrical equipment. The latter are those governing to a great deal the overall operational and structural performance of an industrial plant in case of an earthquake event. To this end, over the past few years, significant effort has been put towards developing a design framework for ancillary elements, which are either supported or nested in such buildings, so as to appropriately treat the vast uncertainties associated with their capacity and seismic demand evaluation. In view that the information available for such secondary nonstructural systems is often abstract, the new Eurocode 8 (CEN, 2022a, CEN, 2022b) offers a spectrum of design approaches, allowing design engineers to explicitly adjust the level of conservatism induced to the design of such acceleration-sensitive components to the level of the available information and to any specific requirements for limiting the induced accelerations.

The developed design roadmap, that will become in the coming years the New European Bauhaus for verifying the satisfactory seismic performance in ancillary elements, was developed based on past evidence for the acceleration demands imparted on acceleration-sensitive equipment. This evidence showcases that the acceleration demands could be amplified from floor to component level by several orders of magnitude, especially at the upper floors 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. The new roadmap offers three alternative design routes, including one

[1] Adjunct Lecturer, International Hellenic University, Serres, Greece, kazantzi@ihu.gr
[2] Member, Societal Resilience & Climate Change (SoReCC) Center of Excellence, Diegem, Belgium
[3] PhD Candidate, National Technical University of Athens, Athens, Greece
[4] Research Associate, National Technical University of Athens, Athens, Greece
[5] Associate Professor, National Technical University of Athens, Athens, Greece

that involves insetting a sacrificial fuse of verified ductility in the component-building load path to limit the imposed accelerations.

Eurocode 8 design methodologies

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

• **Method 1** 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 period of vibration 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.

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

The design horizontal seismic force F_{an} 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_{ap}' is the period-dependent behaviour factor of the ancillary element estimated after Annex C of prEN 1998-1-2:2022, but limited to a maximum value of 1.5 per prEN 1998-4:2022, and $S_{\mathrm{ap},j}$ 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):

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

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

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

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

$$q_{\mathrm{D}}' = \begin{cases} 1.0 & T_{\mathrm{p},1} \leq T_{\mathrm{A}} \\ \text{linear between 1.0 and } q_{\mathrm{D}} & T_{\mathrm{A}} \leq T_{\mathrm{p},1} \leq T_{\mathrm{C}} \\ q_{\mathrm{D}} & T_{\mathrm{p},1} \geq T_{\mathrm{C}} \end{cases} \quad (5)$$

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.

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:

$$S_{\mathrm{ap}} = AMP \cdot PFA \quad (6)$$

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:

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

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 spectra acceleration at the fundamental period $T_{\mathrm{p},1}$ of the supporting structure in the considered direction and the corresponding damping ratio $\xi_{\mathrm{p},1}$ that is subject to a lower bound equal to the elastic response spectra acceleration corresponding to 0.5sec, 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. 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.

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 acceleration demands should a yielding element be inserted between a nonstructural component and the supporting system is provided in Kazantzi *et al.* (2020a; 2022a) 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 $\mu_D \geq 1.50$. 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 $\mu_D \cdot \gamma_{ap}$.

Case study

An equipment-supporting reinforced concrete (RC) moment resisting frame (Figure 1), adapted from Kazantzi *et al.* (2022b), is considered as the case study. This is a typical refinery building, designed for Zone 3 according to the new seismic hazard zonation proposed for Greece 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 as per prEN 1998-4:2022 (CEN, 2022b) for Consequence Class 3a 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 fire proofing) 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. In particular, the building linear elements were modelled using elastic beam-column elements and a rigid diaphragm was assigned at the floor levels, implying that sufficient in-plane rigidity is guaranteed by the RC floor slabs.

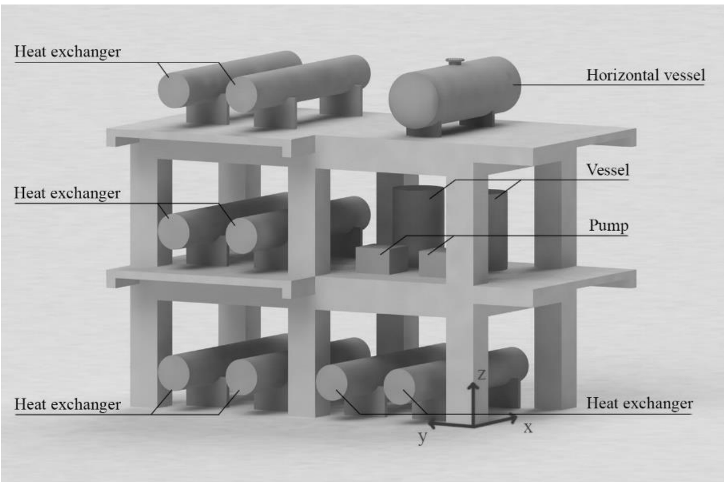


Figure 1. 3D photorealistic representation of the examined RC building with indicative nested equipment that can be found in an oil refinery facility (from Kazantzi et al, 2023).

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 an 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 on a linear (for Methods 1 and 2) and an elastic-perfectly-plastic (for Method 3) single-degree-of-freedom (SDOF) oscillator (see Figure 2). The 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 that are offered in the new generation of Eurocodes. A performance factor γ_{ap} equal to 1.5 has been assumed, since the considered components are part of a safety-critical system.

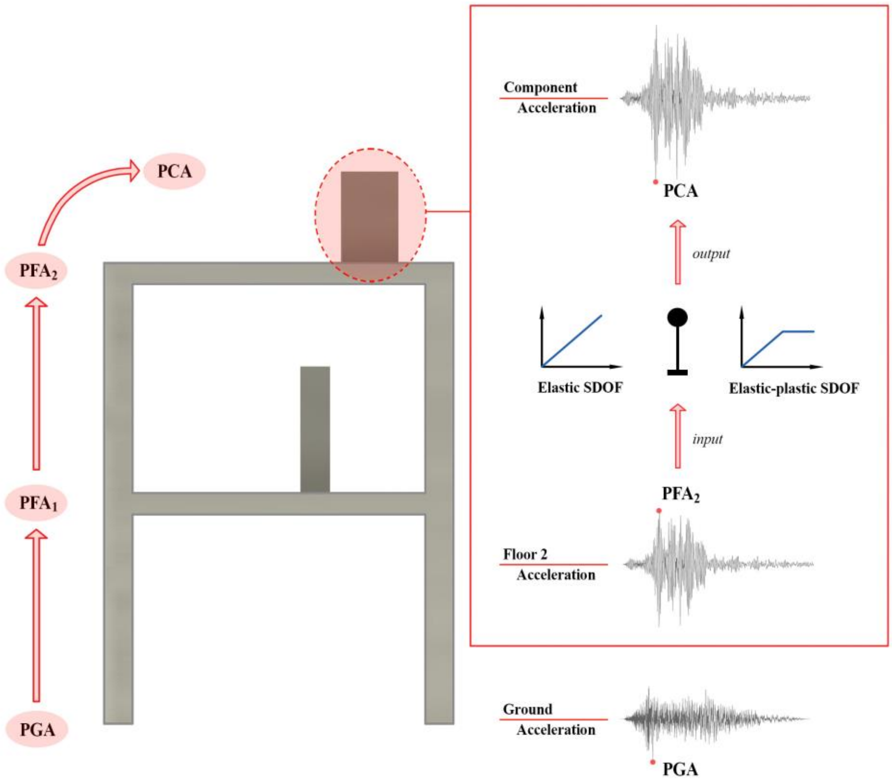


Figure 2. Graphical outline explaining how the component seismic demands have been evaluated in the present study for the dissipative and non-dissipative ancillary elements. PGA denotes the Peak Ground Acceleration, PFA denotes the Peak Floor Acceleration and PCA denotes the Peak Component Acceleration.

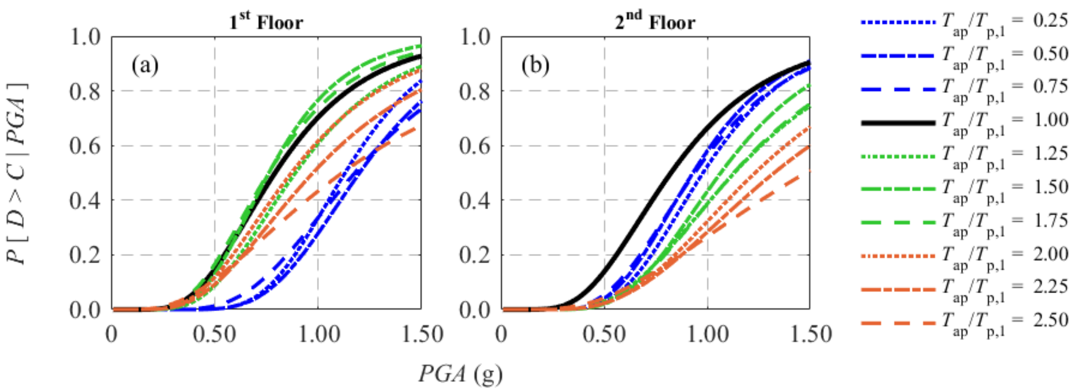
Seismic fragility study

A comparison of the three different design methodologies that were outlined in the preceding sections was undertaken by means of analytically evaluated fragility curves for several nested components of varying periods at both floor levels of the two-storey building. The comparison of the fragilities allows a probabilistic overview on 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 being used as the intensity measure (IM). In particular, the component fragility curves were obtained under the typical lognormality assumption (Cornell *et al.*, 2002):

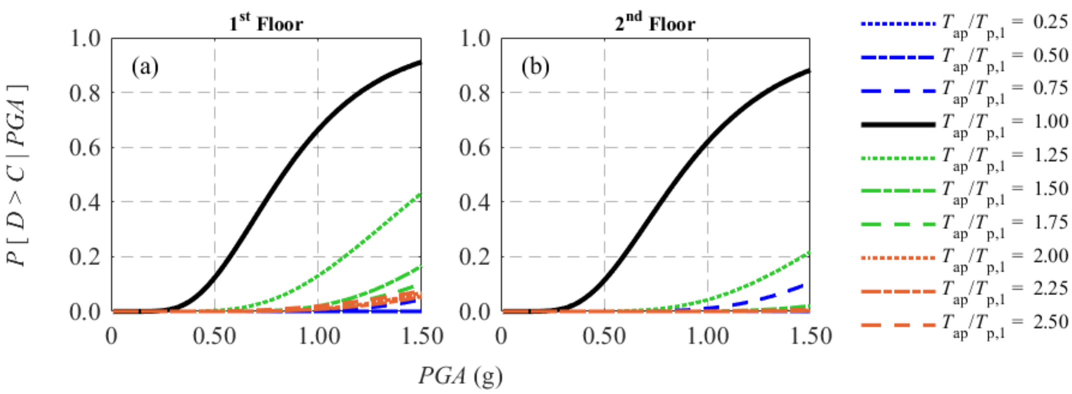
$$P(D > C|PGA = pga) = \Phi\left(\frac{ln(\widehat{D}(pga)) - ln(\hat{C})}{\beta_{\text{tot}}}\right) \quad (10)$$

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.

The obtained fragility curves having the component capacities evaluated on the basis of Method 1 and Method 2 (that essentially correspond to the non-dissipative design approaches of Eurocode 8) are presented in Figure 3. Indicatively, the results are shown for the Y direction of the building (see Figure 1) whereas the fragilities were computed for several components of variable period and for both floors levels of the supporting structure. Evidently, the most fragile components (irrespectively of the adopted non-dissipative design method) are those tuned to the predominant vibration period of the supporting building ($T_{\text{ap}}/T_{\text{p,1}} = 1.00$) in which case both methods achieve a nearly identical seismic performance. By contrast, the results between the two design methodologies significantly differ for the case of the detuned components. On account that 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), 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 to practical design: 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.



(a) Method 1



(b) Method 2

Figure 3. Component fragility curves computed in the Y direction at both building floor levels, obtained for ancillary elements designed to (a) Method 1 and (b) Method 2 and having ten different period ratios of $T_{ap}/T_{p,1}$.

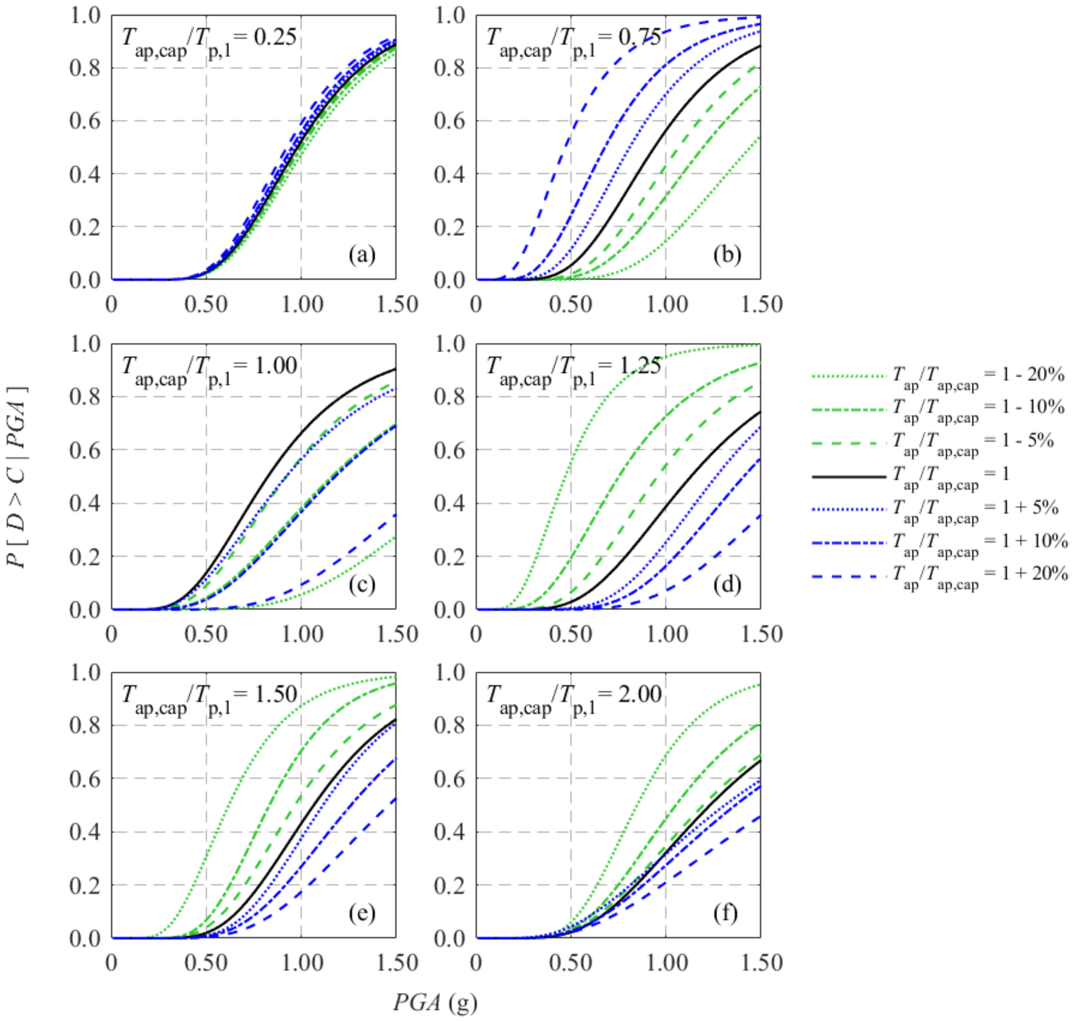


Figure 4. Fragility-based sensitivity analysis for components designed per Method 1 (indicatively presented for the 2nd Floor).

The aforementioned advantage of Method 2 becomes more apparent by means of undertaking a sensitivity analysis for Method 1 to investigate how the component fragility of nonstructural elements designed to the latter method is likely to be affected by the variations in the assumed T_{ap} and $T_{\mathrm{p},1}$ values. In fact, Method 1 requires a great deal of information for designing a component (i.e. estimating its capacity), with the most important ones being the assumptions made with regards to the period of the component T_{ap} and the periods of the supporting structure $T_{\mathrm{p},i}$ (or to put it otherwise how these two dynamic properties compare to each other). However, such kind of information is rarely readily available to the person designing the anchorage of the component, since the latter task 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. Figure 4 presents the fragilities that were computed both for the case where the actual (as-built) values of T_{ap} are equal to those considered for designing the component ($T_{\mathrm{ap,cap}}$), as well as for those cases where the actual period deviates from that assumed in the design (by ± 5,10 and 20%). As can be inferred by inspecting the fragilities in Figure 4, 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 4c). Therefore, one could claim that Method 1 works 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.

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 analytically by Kazantzi *et al*. (2020a; 2020b; 2022a) and experimentally by Elkady *et al*. (2022), 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 5 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.

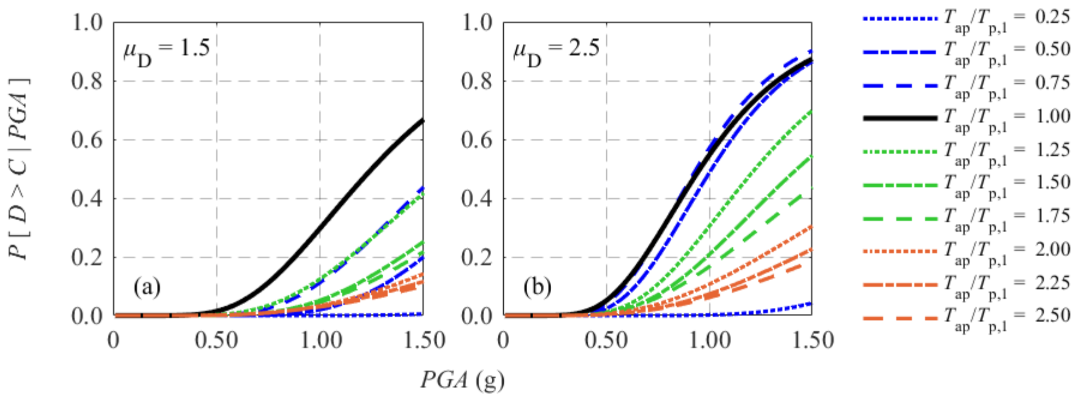


Figure 5. Component fragility curves computed for the critical 1st Floor in the Y direction, obtained for ancillary elements designed to Method 3. Note that the nominal ductility capacity μ_D is reported in the legend, whereas the actual ductility capacity is $1.5 \cdot \mu_D$.

As can be inferred by inspecting Figure 5, 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, Method 3 offers one less obvious but essentially important advantage. Ancillary elements designed by Method 3 will sustain considerably lower accelerations, limited by the fuse yield strength. Hence, by virtue of exploiting the detuning effect of hysteresis, even nominally resonant components receive PFA amplification factors much lower than 7. Thus, not only component safety but also functionality can be secured. The only aspect that currently hampers the applicability of Method 3, is the limited availability of anchoring (commercial) products with verified ductility and strength. Yet, this is an issue that could be addressed by the manufactures of anchoring systems in the coming years.

Conclusions

The new generation of Eurocode 8 offers to the design engineers three alternative methodologies for designing the anchorage system in the case of acceleration-sensitive ancillary elements. In this study, a detailed comparison was undertaken between those seismic design methods, considering a wide range of ancillary components, being tuned to, almost tuned to or away from the predominant period of the supporting structure. On account that the compared design approaches require substantially different levels of knowledge for the dynamic properties of the entire system (nonstructural component and supporting structure) to determine the required input, we have explicitly addressed if and to what extent the said methods and the final design products are likely to be affected and their reliability undermined, should the assumptions made during the design phase deviate from reality. It was showcased that the design method provided in Eurocode 8 – Part 1-2 is robust under the condition that the level of knowledge with regards to the dynamic properties of the nonstructural component and the supporting structure is high. Yet, if the latter is not the case, the reliability of the final design product may substantially downgrade even for small deviations of the actual properties from those assumed at the design phase. By contrast, the two methods that are provided in Eurocode 8 – Part 4, namely the non-dissipative and dissipative approaches, 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 tuned to the supporting structure. It was also demonstrated that the dissipative approach, i.e., the one that allows for certain fuses of verified ductility in the anchorage system to go inelastic, could provide consistently reliable and less conservative final designs in which also the accelerations are substantially lower compared to the elastic design approaches. This latter property could be of interest when designing the anchorage system for a vibration-sensitive equipment.

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