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SEISMIC ASSESSMENT OF SUSPENDED CEILINGS THROUGH DYNAMIC NONLINEAR ANALYSES

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Abstract: *The detrimental effects of irregularities in plan and elevation on the seismic performance of building structures are well documented in the literature. Conversely, researchers and practitioners have paid little attention to the effects of structural irregularities on the seismic response of non-structural components (NSCs). This aspect deserves more detailed investigations, as the poor seismic behaviour of NSCs can put people's life at risk and cause significant direct and indirect economic losses. Unsurprisingly then, current design methods to demonstrate the seismic safety of NSCs have many shortcomings. In this paper, nonlinear dynamic analyses are used to rigorously quantify the seismic performance of suspended metal ceilings, a particularly important type of NSCs due to their diffuse presence in both public and private buildings. The analyses are carried out within the PEER's framework of performance-based earthquake engineering. Nonlinear time-history analyses are executed on a case-study building for increasing levels of the seismic intensity measure and varying degrees of structural irregularity in plan and in elevation, while linear elastic and nonlinear constant-ductility floor spectra allow evaluating the seismic demand on the suspend ceilings, whose mechanical parameters have been calibrated against the results of an extensive experimental campaign.*

Introduction

Suspended ceilings are light ceiling systems normally utilised in commercial or office buildings. They typically consist of ceiling tiles supported by a metal grid, which in turn is suspended from the load-bearing floor through metal wires or ties. The void created underneath the floors of the building can be used for wiring, piping, light fixtures and HVAC (heating, ventilation, and air-conditioning) systems.

During earthquakes, different failure mechanisms can occur in the metal ceilings, including dislodging of panels, fall of panels, perimeter failure, connection failure, pounding with other building components, etcetera (e.g. Dhakal *et al*, 2011; Dhakal and MacRae, 2013). These mechanisms can result in injuries and even deaths, seldomly; more frequently, they affect the functionality of the building in the aftermath of a seismic event, which is particularly worrying in the case of hospitals and other facilities key to civil protection; furthermore, the cost for repairing or replacing damaged metal ceilings can be significant.

In earthquake engineering, suspended ceilings are an important example of non-structural components (NSCs), whose performance should be properly accounted for when quantifying the expected losses due to a certain seismic event (e.g. NIST GCR 17-917-44). Approximate expressions are available in design codes to estimate the seismic forces on NSCs, e.g. ASCE 7-16 and BS EN 1998-1, but several shortcomings affects their application in the context of a proper performance-based design (see e.g. Filiatrault and Sullivan, 2014; Filiatrault *et al*, 2018), including: *i*) the empirical assumption that the peak floor acceleration increases linearly along the height of the building, thus neglecting the effects of higher modes of vibration on the seismic response of the building; *ii*) the characteristics of the nonlinear response of both the supporting

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structure and NSC are not accounted for; *iii*) the adoption of a force reduction factor (i.e. the behaviour factor q_a in BS EN 1998-1) to NSCs is difficult to justify and potentially misleading.

A rigorous approach to the seismic analysis and design of NSCs would require the numerical integration of the nonlinear equations of motion for both the primary (load-bearing) structure and the NSCs. As suspended ceilings are relatively light in comparison to the floors which they are attached to, their feedback force on the primary structure can be neglected (see e.g. Chen and Soong, 1988; Villaverde, 1997; Singh *et al*, 2006a and 2006b; Muscolino and Palmeri, 2007; Kasinos *et al*, 2015). In this "cascade approach", the seismic response of primary structure and secondary system are computed in sequence: that is, first the primary structure, neglecting the presence of any NSC; then the secondary system, which is excited by the absolute motion of the primary structure at the contact points with the secondary system. If the latter can be modelled as a SDoF (single degree of freedom) oscillator singly connected to the primary structure, the cascade approach becomes equivalent to the application of the so-called floor spectra, i.e. response spectra in which the seismic input is the absolute acceleration at a given contact point in the primary structure.

It is well known that irregularities in a building structure can severely affected its modes of vibration and, ultimately, its seismic response (e.g. Shahrooz and Moehle, 1990; Wood, 1992; Valmundsson EV and Nau JM, 1997; De Stefano and Pintucchi, 2008; Anagnostopoulos *et al*, 2015). Architectural and functional requirements, as well as accidental variations, are all potential sources of irregularity, either in plan (i.e. asymmetric distributions of mass, stiffness and strength) or in elevation (e.g. due to discontinuities in structural elements or variations in occupancy), which can result in large torsional effects of deformations concentrated in certain soft storeys.

Comparatively, less attention has been paid to the effects that irregularities in the primary structure can have on the seismic performance of NSCs. Aldeka and his co-authors (Aldeka *et al*, 2014; Aldeka *et al*, 2015) have shown that Eurocode 8 predictions (BS EN 1998-1) tend to underestimate the seismic response of NSCs mounted on the flexible sides of RC (reinforced concrete) MRFs (moment resisting frames) with irregularities in plan. Kasinos *et al* (2015) have investigated the seismic response of secondary MDoF (multiple degrees of freedom) systems multi-connected to primary multi-storey MRFs with various degrees of irregularity; the results of their parametric study show that the effects of irregularities in the primary structure tend to affect more acceleration-sensitive NSCs (such as suspended ceilings) than drift-sensitive NSCs. More recently, Surana *et al* (2018) have proposed some new spectral amplification functions for the seismic design of acceleration-sensitive NSCs in hill-side buildings, affected by a combination of irregularities in plan and elevation.

In this paper, the preliminary results of an extensive parametric study are presented, aimed at quantifying in a rigorous manner the effects that irregularities in the load-bearing structure can have on the seismic response of metal ceilings installed at different locations in a multi-storey case-study building. The analysis of the results supports the claim that simplified design expressions for NSCs, such as those provided by ASCE 7-16 and BS EN 1998-1, do not provide reliable indications on the actual performance of these elements. Instead, seismic damage occurring in metal ceilings can be better analysed through a probabilistic characterisation of a relevant EDP (engineering demand parameter). The latter has been taken in this study as the ordinate of the floor spectrum in terms of pseudo-accelerations for the fundamental period of vibration of the metal ceiling, calculated for increasing values of the IM (intensity measure) of the seismic input and at the locations of interest in the building.

Methods

Performance-based earthquake engineering (PBEE)

Performance-based engineering (PBE) is a general methodology that enables the rigorous probabilistic assessment of the performance of structural systems and infrastructural facilities when subject to natural and man-made hazards (Barbato *et al*, 2014). One of the main reasons underpinning the engineers' quest to overcome prescriptive seismic design codes is that, although they generally provide adequate protection against the no-collapse requirement (NCR), direct and indirect economic losses caused by seismic events of relatively low-to-moderate intensity proved historically to be excessively high (e.g. Ghobarah, 2001; de Luca di Roseto *et al*, 2018), particularly because of damage occurred to NSCs. The most popular framework of performance-based earthquake engineering (PBEE) is by far the formulation developed by the

PEER (Pacific Earthquake Engineering Research) centre since the early 2000s (Porter, 2003), which consists of four successive stages, namely: 1) hazard analysis, 2) structural analysis, 3) damage analysis and 4) loss analysis (see Figure 1).

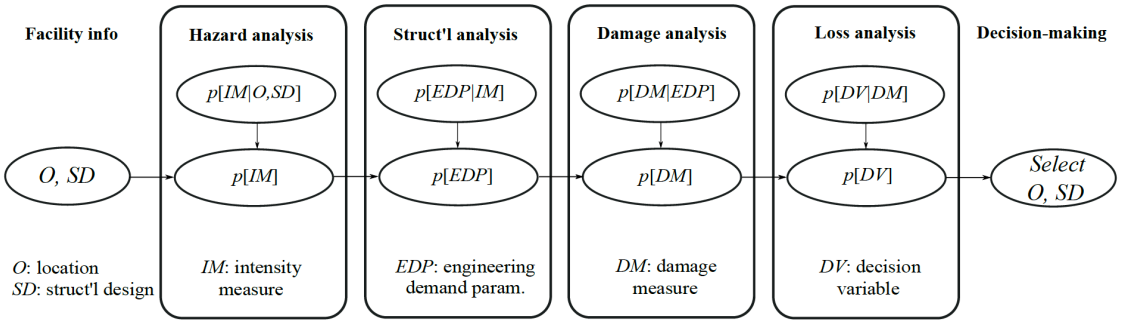

Figure 1. PBEE framework, as conceived by the PEER centre (adapted from Moehle and Deierlein, 2004).

The PEER's framework can be also expressed mathematically through the following triple integral (Lee and Mosalam, 2006):

$$p\left[DV\middle|\left\{O,SD\right\}\right]=\int\int\int p\left[DV\middle|DM\right]p\left[DM\middle|EDP\right]p\left[EDP\middle|IM\right]p\left[IM\middle|\left\{O,SD\right\}\right]\mathrm{d}IM\,\mathrm{d}EDP\,\mathrm{d}DM, \quad (1)$$

where the symbols $p[X]$ and $p[X\,|\,Y]$ are to the probability density function (PDF) of X and the conditional PDF (CPDF) of X given Y; *IM*, *EDP*, *DM* and *DV* are the relevant intensity measure, engineering demand parameter, damage measure and decision variable in the design problem; while *O* and *SD* are the set of parameters characterising location and structural design. In this paper, only the first two stage of the PBEE will be considered, leading to the probabilistic characterisation of *IM* and *EDP*, as detailed in the following subsections.

Case-study structure

A 3-by-2-bay 5-storey steel MRF adapted from the work by Araújo and Castro (2017) has been used as case-study structure. The inter-storey height is 3.50 m, with the only exception of the ground level, whose height is 4.50 m, for an overall building height of 18.5 m; all the bays are 6×6 m, for an overall plan area of 216 m^2, doubly symmetrical in its original configuration. Dead load of 4.0 kN/m^2 and live load of 3.0 kN/m^2 have been assumed. Columns' cross sections are HE 450B for the ground level, HE 400 B for the intermediate levels and HE 300 B for the top two levels; beams' cross sections are IPE 330 for the bottom two storeys and IPE 300 for the other ones; the structural steel grade was specified as S275. The first three modal period of vibration were $T_1 = 0.46\,\mathrm{s}$ in the transverse direction, $T_2 = 0.38\,\mathrm{s}$ for the first torsional mode and $T_3 = 0.33\,\mathrm{s}$ in the longitudinal direction.

In the following, the frame detailed above will be referred to as layout L0 (see Figure 2(a)), without any irregularity. Layouts L1 to L5 have also been considered, with different levels of irregularity, namely:

- L1, characterised by an irregularity in elevation due to the suppression of 2 bays in each of the top two levels (see Figure 2(b));
- L2, with increased irregularity in elevation, the suppression of 2 bays in each of the top four levels (see Figure 2(c));
- L3, with an irregularity in plan, due to enlarged cross sections for the columns located on one of the long sides of the building (resulting in an eccentricity between centre of mass and centre of rigidity of about 1.0 m, see Figure 2(d));
- L4, with an increased irregularity in plan, due to a further enlargement of column's cross sections (with the eccentricity increased to about 1.5 m, see Figure 2(d));
- L5, a combination of irregularities in plan and elevation (namely L2 and L4, see Figure 2(e)).

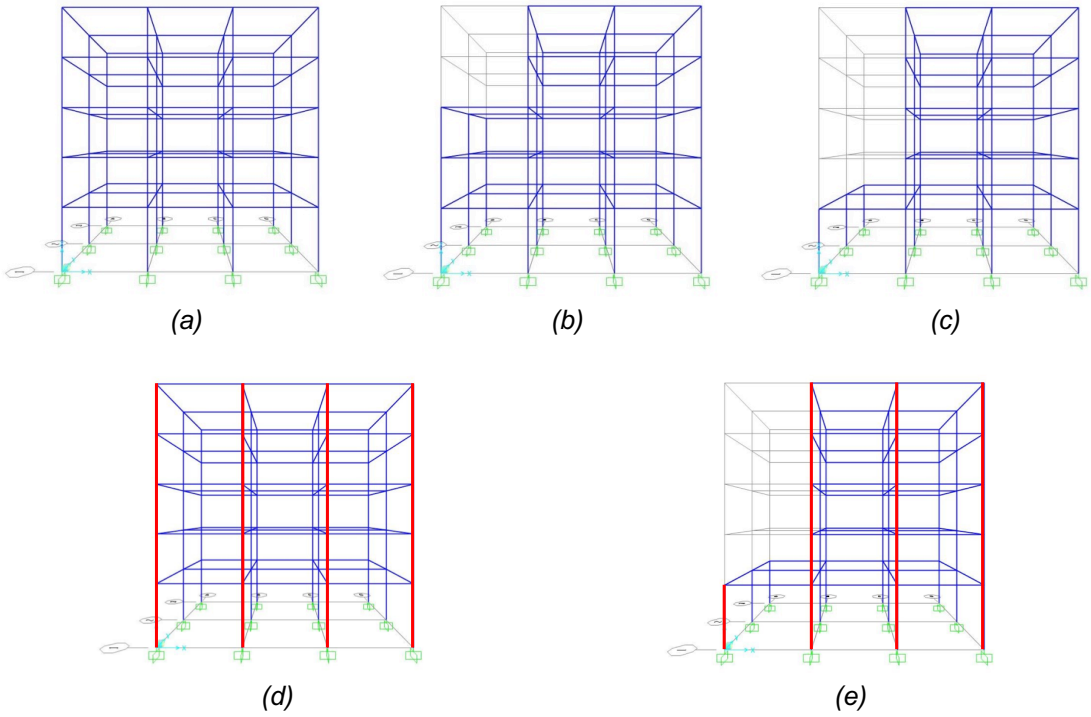

Figure 2. Various structural layouts considered as part of the parametric study.

Hazard analysis

The frame by Araújo and Castro (2017) was designed for a location in Lisbon, Portugal. The PGA (peak ground acceleration) for this original site is approximately 0.2 g for a seismic event with PoE (probability of exceedance) of 10% in 50 years (corresponding to a return period $T_R = 475\,\text{yr}$, i.e. the NCR). A site with the same PGA for the NCR has been identified in the USA, near Charleston, South Caroline (latitude= 33.212° N, longitude= 79.980° W).

For this new design site, the Unified Hazard Tool (UHT) developed by the NSHMP (National Seismic Hazard Mapping Project) within the Earthquake Hazards Program (EHP) of the USGS (US Geological Survey) has been used, assuming: site class “B/C boundary”, corresponding to an average shear velocity in the upper 30m of $V_S^{(30)} = 760\,\text{m/s}$; spectral period $\overline{T} = 0.20\,\text{s}$, which was the value closest to the fundamental period of vibration T_1 among the available options; earthquake hazard model “Dynamic: Conterminous US 2014 (v4.1.1)” (Earthquake.USGS.gov, 2019). $N_{IM} = 3$ return periods have been considered, namely $T_R = 475$, 975 and 2475 yr (or, equivalently, $N_{IM} = 3$ annual frequencies of exceedance, i.e. $\lambda_{IM} = T_R^{-1} = 0.00211$, 0.00103 and 0.00040), for which the IM takes the values $S_{pa}\left(\overline{T}\right) = 0.344$, 0.750 and 1.612g. The hazard curve for the site under consideration is plotted within Figure 3, where the red circles indicate the chosen levels of the IM.

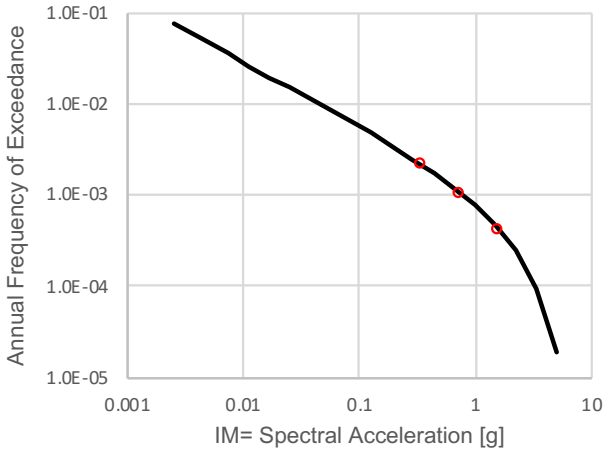

Figure 3. Hazard curve for the sire under investigation in South Coraline.

The disaggregation of the seismic hazard has been performed with the UHT, returning mean magnitude $M_W = 6.5$ to 7.1 and mean rupture distance $r_{Rup} = 23$ to 13km when increasing the IM. For each value of the IM, a suite of $N_{EQ} = 120$ earthquake records have been found in the PEER's ground motion database (NGAwest2.Berkeley.edu, 2019), divided in four quadrants $\{M_W, r_{Rup}\}$, depending on whether they are characterised by magnitude and rupture distance smaller or larger than the corresponding mean values. Each accelerogram has then been scaled to match the value of the IM.

Structural analysis

Each of the $N_{SD} = 6$ variants of the case-study frame (i.e. layouts L0 to L5) was analysed with the commercial structural analysis program SAP2000 (CSiamerica.com, 2019). Standard plastic hinges were defined at the ends of the steel beams (simple flexure "M_3") and columns (axial-flexure interaction "P-M_2-M_3"), according to FEMA-356 (2000), and Rayleigh's damping has been assumed for the load-bearing frame, with an equivalent viscous damping ratio of 5% at $T = 0.1$ and 1.0s. The gravitational actions were initially applied to each frame (100% of the dead load and 30% of the live load) through a nonlinear static case, before the N_{EQ} ground accelerations could be applied. The nonlinear time-history analyses have been performed with the classical Hilber-Hughes-Taylor integration scheme (Hilber *et al*, 1977). Overall, $N_{sym} = N_{SD} \times N_{IM} \times N_{EQ} = 2160$ nonlinear time-history analyses have been carried out.

Nonlinear floor spectra

For each simulation performed with SAP2000, the time history of the absolute acceleration in various control points on the frame have been recorded, including the corner nodes at the top storey. According to the cascade approach discussed in the Introduction, the latter have been used as base motion exciting a series of nonlinear SDoF oscillators, conveniently calibrated against the data collected through an extensive experimental campaign conducted on full-scale metal ceilings with different configurations and technologies (preliminary results have been presented in the work by Fiorin *et al* (2019)). Figure 4 shows an example of hysteretic loops obtained through quasi-static cyclic testing.

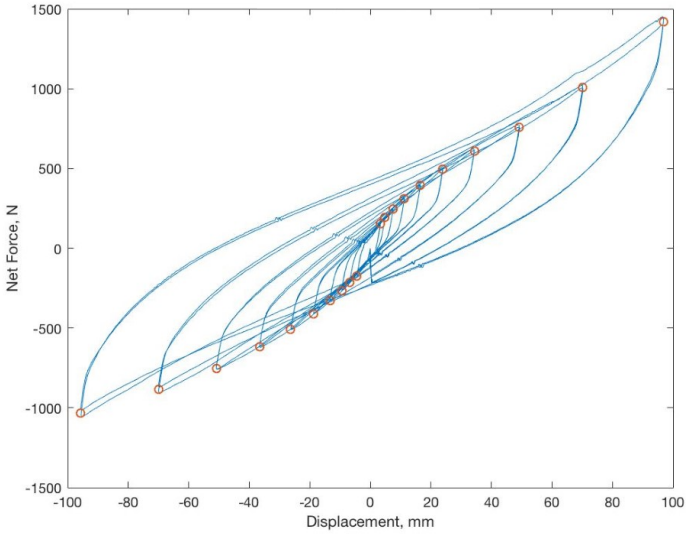

Figure 4. Typical hysteretic loops of metal ceilings obtained through quasi-static cyclic tests.

Results and Discussion

For the sake of conciseness, only selected results are presented in what follows.

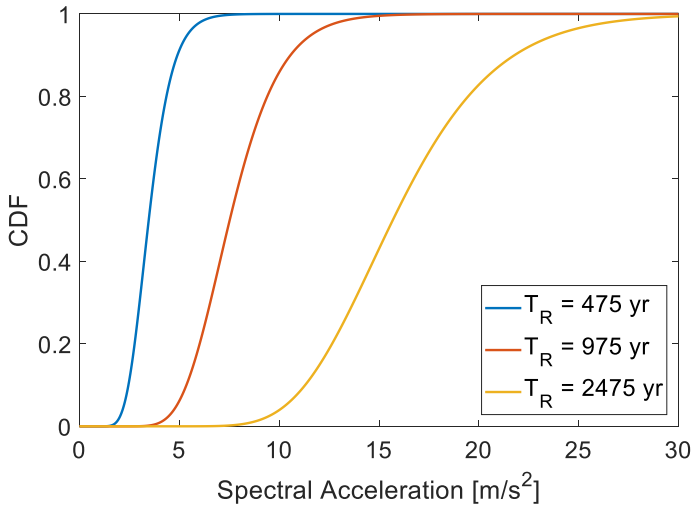

Figure 5. Probability distribution for the EDP chosen to characterise the performance of the metal ceiling.

Figure 5 shows the CDF (cumulative distribution function) of the EDP taken as the spectral acceleration of the metal ceiling installed in the frame of layout L0, for the three value of the IM. As expected, the higher the IM, the higher the EDP. Due to the combined nonlinearity of structural frame and metal ceiling, the EPDs are not directly proportional to the value of IM.

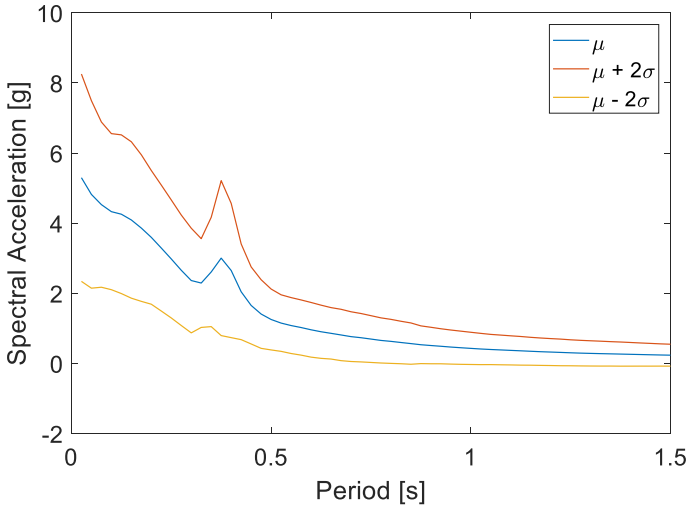

Figure 6. Variability in the nonlinear floor spectra.

Figure 6 shows the aleatory variability affecting the nonlinear floor spectra, assuming that the ductility of the secondary oscillator is $\mu_S = 6$ and its period of vibration T_S varies between 0.025 and 1.50 s. The plots of Figure 6 have been obtained for the lowest value of the intensity measure, namely for $T_R = 475$ yr, and layout L0. It is evident the dynamic amplification occurring when the metal ceiling is in tune with the fundamental mode of vibration of the primary structure (in this case, $T_S = T_2$).

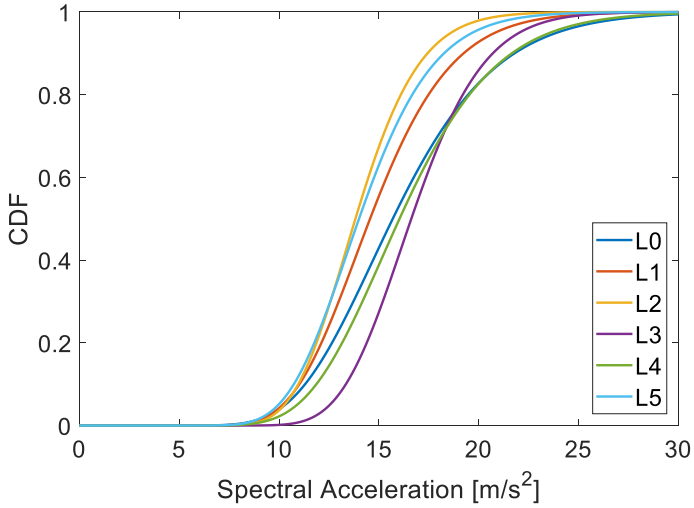

Figure 7. Probability distribution for the EDP for different structural irregularities.

Finally, Figure 7 presents a comparison of the CDFs of the EPD calculated for the six different structural layouts, from L0 (regular frame) to L5 (combined irregularities in plan and in elevation). Although no clear trend can be observed from this figure, it is evident the impact that irregularities can have on acceleration-sensitive NSCs, and in particular metal ceilings. For instance, the variability on the median (CDF= 0.50) can be as large as 20%.

Conclusions

In this paper, the results of an extensive simulation campaign have been presented, aimed at quantifying the effects of irregularities in the load-bearing structure on the seismic performance of suspended metal ceilings. While no definite trends have been identified, the case-study structure has clearly shown the importance to account for the presence of structural irregularities when analysing non-structural components (NSCs), with variations in the order of 20%. A similar level of variability has been observed for the aleatory nature of the seismic action.

The numerical results confirm the inadequacy of force-based design provisions for NSCs in current design codes, which do not consider the impact of soft storeys, eccentricities in plan and other forms of structural irregularity. Further investigations are needed to establish clear correlations between structural irregularities and the seismic performance of NSCs, which in turn can guide designer and specifiers.

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